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Laboratory Testing of Cohesive Subgrades: Results and Implications Relative to Structural Pavement Design and Distress Models

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A thorough investigation of the distress and deformation properties of subgrade materials is under way at the National Danish Road Laboratory. Thus far, the program has concentrated on establishing sound laboratory relationships for cohesive subgrades with respect to resilient and permanent strain characteristics through the use of triaxial testing equipment. In the dynamic testing phase alone, more than 75 million repeated loads have been applied on a series of intact samples representing 12 test sites from 6 countries, and 4 new sites are being added. Although the results of the program thus far must be regarded as inconclusive from a predictive performance point of view, some basic relationships have emerged that are both sound and useful. First, the analysis of the data collected has indicated that the concept of equivalent axle loads is both misleading and irrelevant with respect to directly related subgrade-distress causes. A criterion based on a realistic requirement aimed at limiting the amount of permanent deformation in the soil itself has an extremely high axle-load exponent (mean value greater than 15). Second, the use of a constitutive relationship that can describe the progress of permanent subgrade strain is possible, but because of other uncertainties associated with natural variations, inhomogeneities, and difficulties inherent in the determination of material-characterization constants, the use of a permissible subgrade deviator stress is preferred; the suggested one is a function of a reference resilient modulus (M_0). As a result of the high load exponent hereby implied, this permissible stress is only slightly dependent on the number of load repetitions. On the other hand, the concept of a permissible resilient strain is shown to be poorly correlated with permanent strain and the number of repeated loads because of the various forms and degrees of nonlinear elastic response observed among the investigated subgrades. A fictive strain can, however, be used in lieu of the permissible-stress approach, although this will result in overly conservative designs if linear-elastic material behavior is assumed in the design technique.

The wealth of information and experience gained from the AASHO Road Test and subsequent smaller scale projects has, in a more or less direct manner, exerted a great influence on flexible pavement design practice the world over. Based on this background, distress models have been empirically derived in accordance with various measures of serviceability. The question will now be raised as to the degree of validity of these models as applied to cohesive subgrades on a global scale, that is in general terms apart from the specific AASHO Road Test materials and structural systems, imposed traffic, climatic conditions, and subsequent interpretations. The following is intended to elucidate the results of research (which has thus far been limited to cohesive subgrades) that is intended to answer this question.

TERMINOLOGY AND NOMENCLATURE

The following notation and specific definitions are used in this paper.

N = number of loads,
σ = stress and is positive in compression,
ϵ = strain and is positive in compression,
$\sigma_{1,0}$ = vertical preload (static) stress,
$\sigma_{3,0}$ = horizontal preload (static) stress,
σ_{dyn} = superimposed dynamic deviator stress,
$\sigma_{dyn,f}$ = σ_{dyn} at triaxial failure for $N = 100\,000$,
$\sigma_{dyn,p}$ = σ_{dyn} at ϵ_p permissible,
ϵ_r = resilient axial strain,
$\epsilon_{r,p}$ = ϵ_r permissible,
ϵ_p = permanent axial strain,
M_r = resilient modulus of elasticity (σ_{dyn}/ϵ_r),
M_0 = reference value of M_r, and
β = degree of failure ($\sigma_{dyn}/\sigma_{dyn,f}$).

Permanent strain hardening (often called time hardening) is defined as the rate of permanent deformation with respect to N and σ_{dyn} and is governed by the previous amount of permanent strain the soil element has experienced because of dynamic loading ($<\sigma_{dyn,f}$) (see Figure 1a).

Fatigue hardening (often called strain hardening) is defined as the rate of permanent deformation with respect to N and σ_{dyn} and is governed by the number of previous dynamic load repetitions (N) regardless of their magnitude ($<\sigma_{dyn,f}$) (see Figure 1b).

APPROACH

What happens to a pavement subgrade under the influence of time, traffic, and environment? From a design-performance point of view, the potential modes of distress directly or indirectly attributable to a subgrade can be limited to three:

1. Elastic (resilient) deformations in the soil caused by traffic loadings;
2. Plastic (permanent) deformations in the soil caused by traffic loadings; and
3. Plastic deformations in the soil caused by climatic or environmental effects such as changes in soil suction, freeze-thaw cycles, or differential settlements.

Mode 1 can be considered an indirect cause of distress inasmuch as it only affects potential points of distress in the overlying structural layers. Modes 1 and 2 are often complicated by mode 3, which can result in varying degrees of distress for the same traffic. Mode 3 can also work independently and is manifested by such effects as swelling and frost heave.

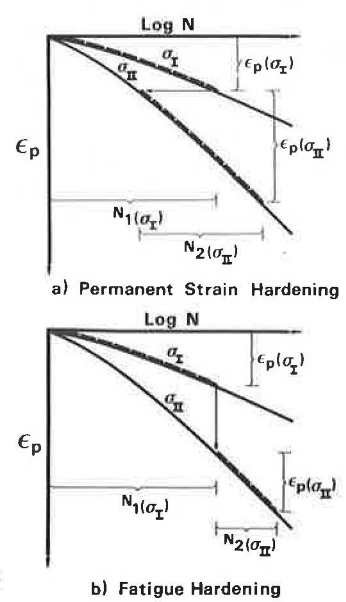
Mode 1 (elastic behavior) is thus considered nonapplicable as a direct measure of subgrade distress, although E-moduli and resilient behavior have been appropriately considered in the research program to facilitate the structural analysis as a whole. Distress that arises partly or wholly from resilient movements in the subgrade is important insofar as it affects other pavement components, but this effect is beyond the scope of the present analysis. Non-traffic-associated distress (mode 3) that is caused by such effects as freeze-thaw cycles is also beyond the current scope because it can and obviously should be handled in a way other than to relate it in a design technique or distress structural subsystem to traffic loadings.

The main objective here, then, is to ascertain how and to what extent loss in serviceability can be attributed to the subgrade distress that results from wheel loads. Loss in serviceability must be related to differential changes in subgrade surface level if it is to be directly attributable to the subgrade itself. Such changes in subgrade surface level then manifest themselves as surface irregularities (e.g., bumps or ruts). If these differential changes arise directly as a result of subgrade movements, it follows that the movements must be plastic in nature. From a design point of view, then, the question is, "How are these (potential) plastic movements related to traffic loadings?"

The first step in elucidating the problem was to conduct a series of triaxial tests to establish some basic relationships between the elastic (or resilient) and the plastic (or permanent) deformation behavior of subgrades. This phase of the research has been carried out over the past 3 years at the National Danish Road Laboratory.

This investigation is essentially different from most of the research conducted elsewhere, but uses similar triaxial procedures. Here, we are not concerned with basic research as such, where such effects as preconsolidation, structure, density, and water content are artificially created and their effects studied in the laboratory. We are concerned, instead, with soil behavior that more closely approximates actual field conditions. Our results are intended to be directly related to practical design techniques in which, on the basis of experience and certain conventional approaches, the soil is directly or indirectly assigned a set of parameters and distress properties.

Figure 1. Models of permanent deformation for cohesive subgrades.



Note: For the same numbers of repeated loads $N_1 + N_2$, the total permanent strain = $\epsilon_{p(\sigma_I)} + \epsilon_{p(\sigma_{II})}$.

PROCEDURE

Sampling and Specimen Preparation

Intact sampling techniques were considered to be an unconditional prerequisite to obtaining meaningful results from the triaxial tests because it is not possible to satisfactorily reproduce the soil fabric of natural deposits of cohesive soils through laboratory recompaction procedures. It is even doubtful whether embankment subgrades can be consistently reproduced by using laboratory compaction techniques. Thus, intact samples of cohesive subgrades were obtained from various field locations by using 12.5 × 74-cm (5 × 29-in) cylindrical samplers (1) and, at the same time, field measurements of various types were conducted and other pertinent data collected.

From each core, two test specimens, each having an area of 100 cm^2 (15.5 in^2) and a height-to-diameter ratio of approximately 2, were trimmed for triaxial testing (see Figure 2).

Triaxial Apparatus and Loading Procedure

Because we are interested in the properties of the subgrade in situ, unsaturated samples corresponding to in situ conditions were used and the triaxial tests carried out undrained. Thus, total stresses (σ) were used throughout; the magnitudes of the neutral stresses in the water and air phases must necessarily remain unknown.

A constant confining pressure and a vertical static preload corresponding to realistic subgrade conditions were used. Typically, $\sigma_{3,0}$ was kept between 5 and 10 kPa (0.7 and 1.4 lbf/in^2) and $\sigma_{1,0}$ at about 2 × σ_3.

This static stress state was subsequently superimposed by simulating the vertical dynamic stresses introduced by traffic (i.e., σ_{dyn}); σ_{dyn} was continously monitored and adjusted in direct proportion to the changing maximum cross-sectional area. The vertical deformation could be measured as a piston movement outside the triaxial cell or on the sample itself by using linear variable differential transformers (LVDTs) (see Figure 2).

The deviator stresses were applied to the triaxial sample by using an electrohydraulic loading system, as shown with the triaxial cell in Figure 3. For the dynamic tests, a loading time of 0.1 s (haversine formed) was used, and various rest periods between pulses from continuous loading (0-s rest period) to several-minute periods were investigated. There were no significant differences in either permanent or resilient deformation performance for rest periods of 0.33 s or more, although shorter ones caused changes in specimen behavior, e.g., the delayed elastic response phenomenon. Thus, typically, a loading frequency of 2 Hz (i.e., a 0.1-s load and a 0.4-s rest) was selected.

Figure 2. Intact soil sampling tube, specimen in trimming apparatus, and specimen in triaxial cell with mounted LVDTs for vertical strain measurements.

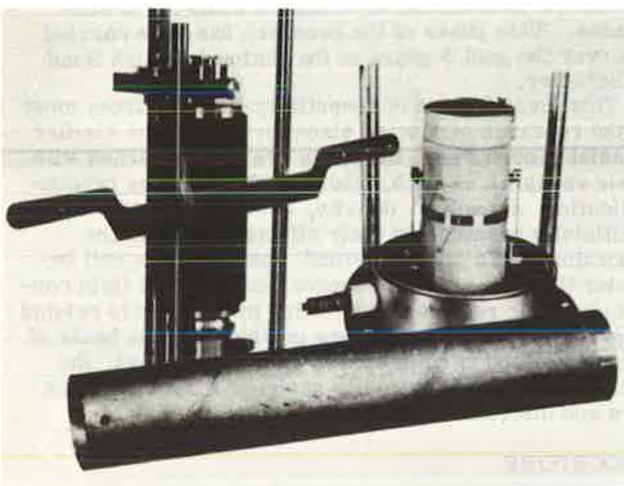
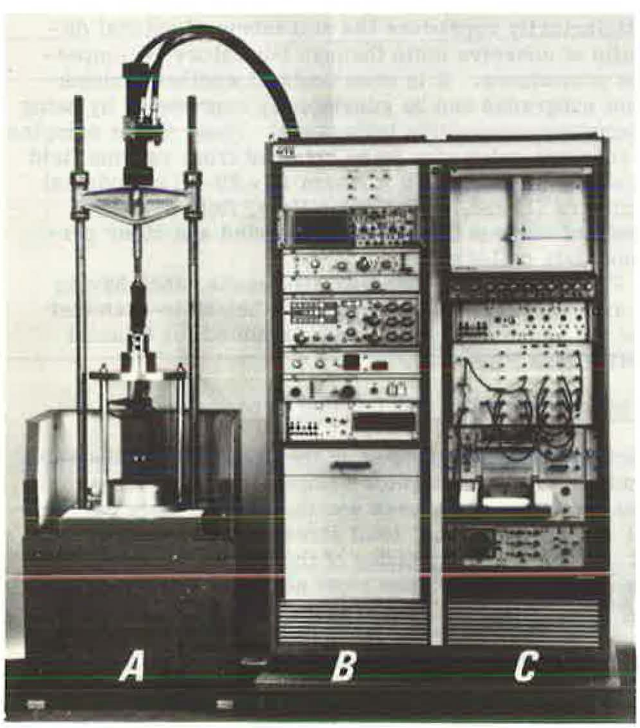


Figure 3. Dynamic triaxial equipment: (a) in-place cell, (b) control panel, and (c) signal conditioning and registration system.



Registration of Loads and Deformations

One of the problems associated with dynamic testing of soils is the separate registration of the resilient and permanent components of the transducer signals. To this end, a signal conditioning system was designed and developed in 1973 at the National Road Laboratory that, in effect, continuously separates the deformation signal—or any similar electronic signal—into three components: the permanent, the resilient, and the peak-to-peak value of the resilient. The procedure is illustrated schematically in Figure 4. The time-dependent values of the various components can then be recorded as desired. A similar signal conditioner was used to continuously monitor the stresses.

MATERIALS AND PARAMETERS

Types of Soil Studied

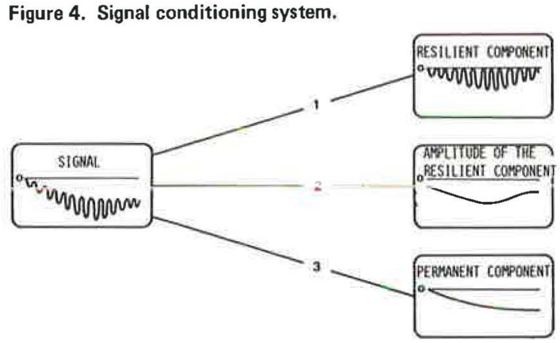
This research has been limited to cohesive materials, both natural deposits and embankments. From a Danish point of view, there is a high motivation for this because the greater part of the country (~70 percent in terms of kilometer tons of highways) is covered by moraine deposits of boulder clay.

Thus far, 16 test sites from 6 different countries have been investigated. The foreign road-test sampling sites included the AASHO (United States), Alconbury Hill (England), and Hilpoltstein (West Germany). Some of the types of soil exhibited inhomogeneities with respect to depth (e.g., the AASHO embankment clay). Several of the sites have been studied over a period of more than 1 year, which has allowed several samplings and the observation of some seasonal variations of subgrade properties.

Parameters Measured

For each type of soil, a series of routine tests was conducted. These included grain-size analyses, Atterberg limits, calcimeter tests, specific gravity measurements, and standard Proctor compaction tests. The materials investigated varied in plasticity index from 6 to 50 percent, in clay content from 12 to 60 percent, and in silt content up to 65 percent. For each sampling, the void ratio (e) and several water contents (ω) were determined, and one or more static triaxial compressive tests and two or more dynamic triaxial tests were car-

Figure 4. Signal conditioning system.



ried out. Because the dynamic triaxial test was the basis of the research viewed as a whole, most of the time and effort spent on laboratory testing has been directed toward this particular test—the triaxial apparatus has undergone some 75 000 000 repeated loads since the inception of the project!

A limited number of California bearing ratio (CBR) tests were conducted, both of the recompacted type and the intact type (from intact specimens). The values obtained by these two approaches were oftentimes widely divergent; the latter type appears to be the only one that has any relation to actual field conditions, especially in the case of natural deposits (see Figure 5).

RESULTS

Resilient Moduli

One of the primary parameters available from triaxial testing is the resilient modulus (M_r). Some typical M_r versus σ_{dyn} relationships are shown in Figure 6.

Figure 6a shows the mean, the ±1 standard-deviation, and the spring-thaw curves for one of the Danish test sites. It must be remembered, however, that the spring thaw of 1976 in Denmark was an unusually mild one; thus the observed decrease in values of M_r was not particularily pronounced.

Figure 6b shows a typical example of the M_r versus σ_{dyn} characteristics of cohesive subgrades. It is obvious that the assumption of a constant, linear-elastic E-value is hardly justifiable. It can also be seen that the commonly used stress-dependent relationship used for cohesive materials

$$M_r = C \times \sigma_{dyn}^{\alpha} \tag{1}$$

where C and α are material constants, can and often is in error in principle because of (a) the oftentimes increased values of M_r at higher values of σ_{dyn} (which is true for about half of the soils investigated) and (b) the fact that, for very low values of σ_{dyn}, M_r will not approach ∞ in accordance with Equation 1 and the negative value of α associated with cohesive materials, but rather have a finite limit. Nevertheless, if this relationship is used for values of M_r to the left of the point at which the curve of M_r versus σ_{dyn} bends upward, an acceptable coefficient of determination (r^2) is found, which in all cases is better than 0.90 when all test values in this range are used as input [i.e., from σ_{dyn} = 5 kPa (0.7 lbf/in^2) to the bending point].

It was also found that M_r tended to increase with increasing N for lower values and decrease for higher values of the degree of failure (β). These deviations were typically less than 15 percent, however, and will not be considered as important at this point.

Other abnormalities in measured values of M_r were also observed under multiple-stage testing, especially as a result of a decrease in σ_{dyn}, but these tended to stabilize if the length of time of the dynamic testing at the new stage (or, alternatively, the rest-period length before resuming testing) was sufficient. This phenomenon is apparently associated with the undrained nature of the triaxial test, and it remains to be established to what extent field conditions can be represented as such. Again, however, it would appear that misgivings about this type of imperfection in behavior are rather academic because of the relatively short-lived changes observed and the fact that a sufficient rest period tends to cancel such effects.

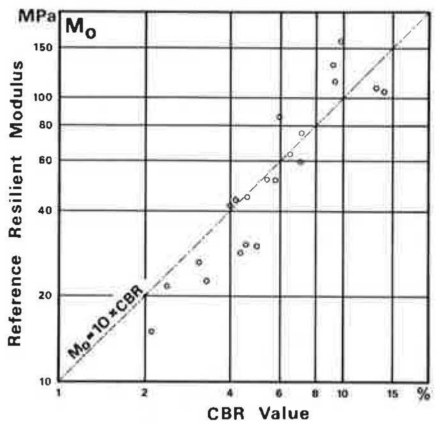


Figure 5. California bearing ratios of intact (unsoaked) samples versus reference resilient modulus.

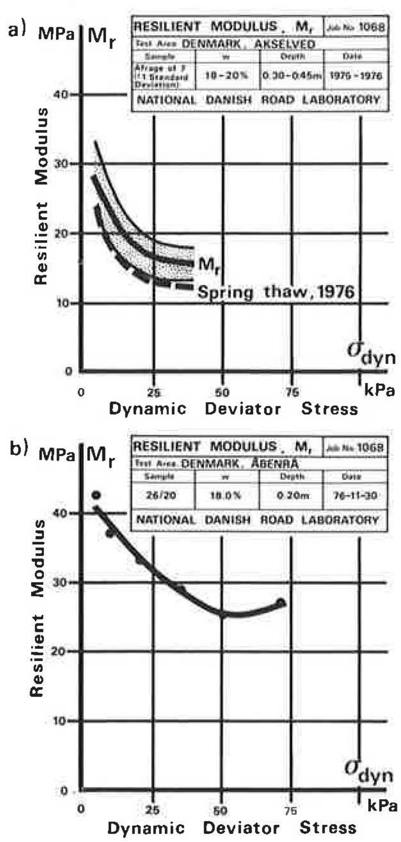


Figure 6. M_r versus σ_{dyn} relationships for two cohesive subgrades based on multiple-stage triaxial tests of intact samples.

Concept of M_0

For various reasons, a reference resilient modulus (M_0) was defined as the M_r-value at a dynamic deviator stress of 15 kPa (2.2 lbf/in^2).

In the first place, this particular modulus corresponds well empirically with the surface modulus (E_0) determined by the standard field procedures used at present in Denmark, i.e., static plate loading and the falling-weight deflectometer. E_0 is of course always measured at a higher surface stress than 15 kPa, but the stress-dependent nature of cohesive subgrades results in an effective composite resilient modulus within the elastic half space under the loading plate that appears to be satisfactorily close to the chosen value of M_0. The relationship between E_0 and M_r for cohesive materials has been discussed by Ullidtz (2).

Second, CBR values of intact samples could be positively correlated with M_0; the observations made thus far are shown in Figure 5. This would tend to corroborate the old rule-of-the-thumb relationship that 10 × CBR (percent) = E-modulus (MPa). Subsequent field investigations, however, have indicated a tendency to overestimate M_0 by using the 10 × CBR rule. In addition, it can be seen that the spread in results shown in Figure 5 is appreciable, so that a cautious attitude towards the CBR approach is deemed advisable.

Permanent Deformation Models

Monismith, Ogawa, and Freeme (3) have used two distress models, shown schematically in Figure 1. (The terms "time hardening" and "strain hardening" used by Monismith and others are equivalent to the terms "permanent strain hardening" and "fatigue hardening" respectively used here.)

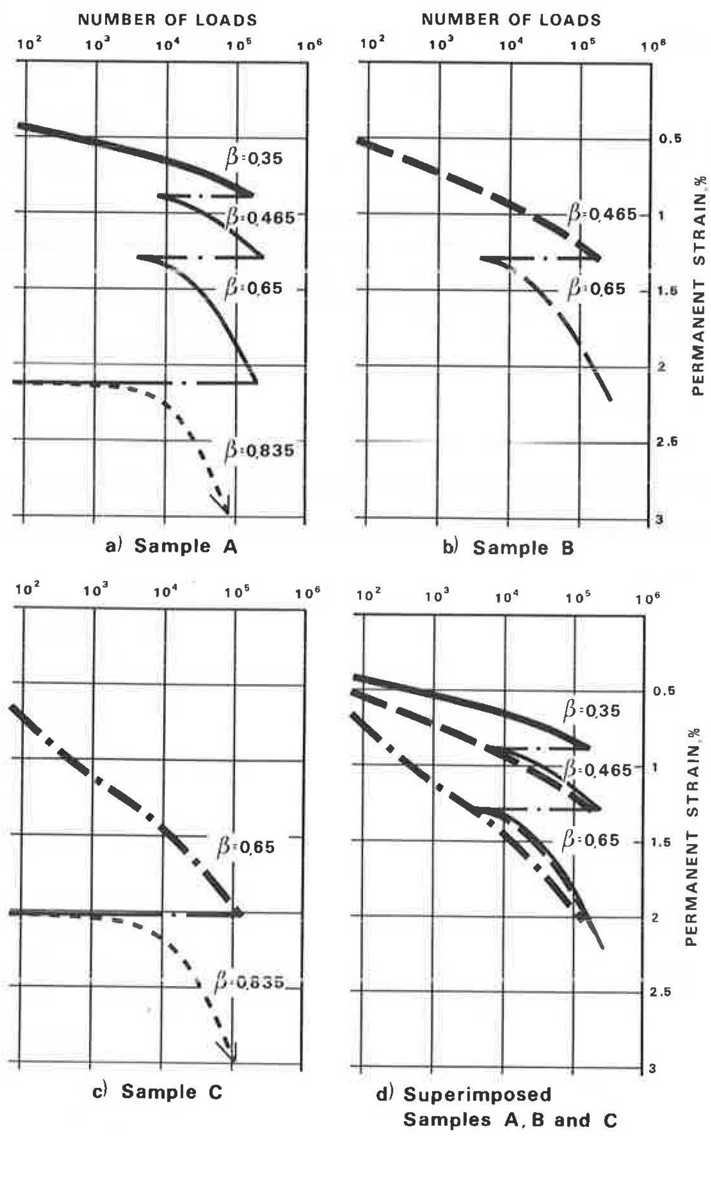
Several attempts were made to ascertain whether one of these models could satisfactorily describe the behavior of the investigated soils. In agreement with the findings of Monismith and others, it appears that neither represents permanent deformation behavior perfectly.

The permanent-strain-hardening model, however, appears to fit the available data well enough that it can be used with a reasonable degree of confidence. This was first established on the basis of pilot tests conducted in 1974 and formed the basis of the multiple-stage testing and interpretation techniques used in this investigation. The reasonableness of this model has now been verified for several of the types of materials used.

The results of one of these attempts are shown in Figure 7. The specimens were trimmed from intact cores of a natural deposit of calcareous boulder clay from the Jutland peninsula of Denmark. They were as close to identical as possible in terms of depth, location, dry density, moisture content, void ratio, and M_0. The thickest lines represent the single-stage deformation paths for each respective sample (A, B, and C), for stress stages 1, 2, and 3 (β = 0.35, 0.465, and 0.65, respectively). Stage 4 (β = 0.835) was run on samples A and C as an additional check to support the similarity of the soil samples' homogeneity in terms of plastic behavior. The thinner lines represent the subsequent permanent deformation behavior of samples A and B and use the starting points on the corresponding single-stage paths in accordance with the permanent-strain-hardening model.

When these curves are superimposed on one another (Figure 7d), the correspondence with the single-stage paths under multiple-stage loading is striking. At first, the rate of plastic deformation is somewhat lower,

Figure 7. Permanent deformation behavior of three identical intact samples of a cohesive subgrade (natural deposit) based on the permanent-strain-hardening model.



although by no means low enough to even approximate that of the fatigue-hardening model. After a fairly large number of dynamic loads, however, the deformation rate exceeds even that of the single-stage curves, until they once again almost coincide (but this requires at least 100 000 loading cycles). Similar tendencies were also observed for decreasing stress stages.

It thus follows that the use of the permanent-strain-hardening model will give satisfactory results for sufficiently high numbers of repetitive loads at the same magnitude. It is not obvious, however, whether an actual traffic-loading spectrum will result in less permanent deformation than the model specifies, because such large numbers of high loads never take place continuously in practice. Nevertheless, at this point, it appears that an assumption of permanent strain hardening under triaxial testing gives results that are on the safe side with respect to structural evaluation of pavement distress.

On this basis, then, if the permanent-strain-hardening model described by

$$\delta\epsilon_p/\delta N = f(\sigma_{dyn}, \epsilon_p) \tag{2}$$

where ϵ_p is the result of previous dynamic loadings, is used, it follows that the material-characterization constants of a satisfactory constitutive relationship that describes the progress of permanent deformation of a specimen subjected to multiple-stage loadings can be derived through a relatively simple iterative procedure.

Application of the Model

To establish a true picture of the progress of permanent strain for a given σ_{dyn}, it was necessary to operate with a large number of loading cycles, especially at significantly high levels of β. At lower stress stages, 10 000 to 100 000 repeated loads were normally used and at higher stages, 100 000 to 1 000 000. After attempting to fit the data to several equations designed to describe permanent deformation behavior, it was found that the basically simple expression

$$\epsilon_p = e_1 N^{e_2} \beta^{e_3} \tag{3}$$

where e_1, e_2, and e_3 are material-characterization constants and N is the number of loads corresponding to a single-stage test, used with an iterative procedure that agrees with the assumed model (Equation 2) gives a satisfactory correlation ($r^2 > 0.98$ for all soil samples) with each data set. The parameters (e_i) were not determined through a conventional multiple regression analysis, but rather by minimizing the sum of the squares of the absolute values of the deviations. Equation 3 is straightforward and simple, but has the disadvantages that it is in error for small values of N and diverges from the raw data at very low values of β. Nor does it describe the failure state. It can be argued, however, that small values of β give rise to only very small amounts of permanent strain (typically underestimated by Equation 3), and its valid range can be limited, for example, to $N > 100$ and $\beta < 1$.

For significant magnitudes of permanent strain relevant to the concept of road serviceability, Equation 3 thus provides a valid expression that represents the behavior of any given triaxial sample. When the additional data are available from the other test sites, a more refined expression can and will be developed, but for the present purposes, Equation 3 will suffice.

Unfortunately, however, whether Equation 3 or a more refined (and complex) representation is used, the derived values of the constants, especially e_i, were significantly median dependent and, as far as could be ascertained, have either a very weak or no correlation at all with any of the general soil parameters available from either routine or special laboratory tests. It might in fact be that any satisfactory mathematical representation of the development of permanent strains will require the use of some type of accelerated fatigue test, because such an expression varies not only with type of soil (test site), depth, natural variations, or other properties measurable with conventional classification methods, but also with the geological origin and history of the soil.

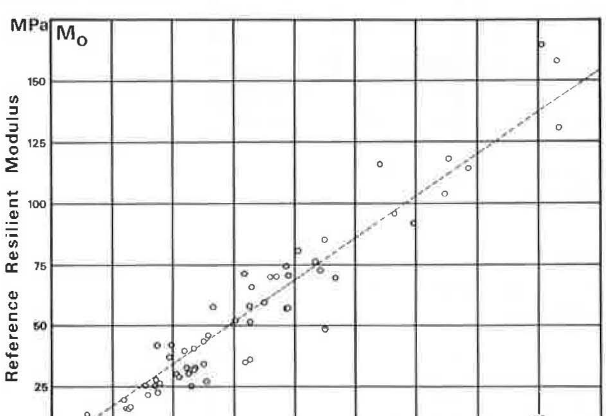
Thus, at least as far as practical road design and construction are concerned, it appears that the most useful approach at present is not to try to predict the development of permanent strains, although this appears theoretically possible, providing a great deal of information is available. (This is being pursued in the continuing research program at the National Danish Road Laboratory.) Rather, an alternative approach that ensures that a limiting amount of permanent strain will not be exceeded is suggested as one that appears to be both more direct and more useful.

Criteria Concept

After the exclusion of a few samples that showed abnormal failure behavior because of sample damage, procedural errors, or loading-system failures, the M_0-values of all the multiple-stage samples tested were plotted against the permissible stress ($\sigma_{dyn,p}$) as shown in Figure 8, where $\sigma_{dyn,p}$ is defined as the dynamic deviator stress associated with ϵ_p = 2 percent for N = 100 000.

The correlation is surprisingly good, and it is of course also possible to plot similar curves corresponding to alternative values of ϵ_p or N in the same manner. But on doing this, one finds, perhaps surprisingly, that the curves for other values of N are almost identical with that shown in Figure 8, the difference ($\Delta\sigma_{dyn,p} < \pm 15$ percent) being less than the scatter in results. In other words, when a reasonable value of ϵ_p is used as the criterion, the magnitude of the repeated loads is far more important than the number of repetitions.

Figure 8. Permissible dynamic deviator stress relationship for cohesive subgrades.



Notes: N = 100 000, ϵ_p = 2 percent, and r^2 = 0.92, based on laboratory tests of 59 intact samples from 12 test sites.

Thus, by using the available test results and information about natural subgrade variations along a design-relevant length of roadway, it should be possible to develop a criterion that can be used in actual pavement design or subsequent evaluation. The final criterion will naturally be based on changes in the serviceability of the pavement as introduced by the traffic-related permanent deformations in the subgrade itself.

This analysis, which is at present under way, includes the use of nonlinear stress-strain calculations and probabilistic considerations based on both field and laboratory tests.

In the dynamic triaxial test, no consideration is taken of the favorable horizontal stress redistributions that would occur under a pavement as a function of time, permanent deformation, and activated earth pressures. In agreement with normal geotechnical practice, a safety factor of more than 2 against total failure is present. As a first approach, it is therefore deemed reasonable to assume that the average-value regression curve shown in Figure 8 reflects a conservative relationship if the critical deviator stress at the subgrade surface is substituted for $\sigma_{dyn,p}$ and can thus represent a useful tool to the discerning engineer. The criterion has the advantage of expressing sound relationships between parameters directly related to distress in the subgrade itself, rather than indirect empirical evidence (such as the AASHO) of surface deterioration relative to a subgrade criterion that might be only indirectly (because of distress mode 1) or even not at all related to subgrade distress.

Strain Criteria

Although there was a good correlation between $\sigma_{dyn,p}$ and M_0, the actual resilient strain values corresponding to $\sigma_{dyn,p}$ (e.g., at ϵ_p = 2 percent and N = 10 000 to 1 000 000) were very poorly correlated. Again, these values were highly median dependent; the tests shown in Figure 8 are represented in Figure 9 in terms of the actual resilient strain corresponding to $\sigma_{dyn,p}$ and have an unacceptably high standard deviation. Only if one particular soil type is plotted does the relationship appear reasonable.

Figure 9 also shows a fictive representation of $\epsilon_{r,p}$, where $\epsilon_{r,p}$ (fictive) is defined as $\sigma_{dyn,p}/M_0$. This relationship is equivalent to the $\sigma_{dyn,p}$ versus M_0 concept and naturally shows an equally good correlation. It is fully recognized that the use of the fictive strain criterion in conjuction with M_0 in design practice, where assumptions of linear-elastic materials are used, will result in overconservative designs because of the effect of the so-called stress-softening elastic behavior of cohesive subgrades. This is the result of the spreading out of the stresses on the subgrade itself for the actually lower resilient moduli experienced at dynamic deviator stress levels higher than 15 kPa (at M_0), which virtually all subgrades can withstand. At present, it thus appears that the stress criterion should be preferred in connection with some provision for stress-dependent E-values in design calculations. The best-fit line representing $\epsilon_{r,p}$ (fictive) is shown together with some of the current strain criteria (4, 5, 6) in Figure 10.

The important thing to notice here is that Poulsen's slope agrees with the findings of Witczak, whose analysis was based on airfield pavement performance. This is perhaps not surprising, inasmuch as distress in such types of pavement is normally dominated by vertical distortion and not fatigue cracking in the asphalt-bound layer.

Numerically, the slope is $\approx$0.06. By assuming that the vertical wheel load is proportional to the vertical deviator stress and taking the reciprocal of the slope, one can calculate the effective load-equivalency exponent, which in this case is about 17. If equivalent 80-kN (18 000-lbf) axles were actually used for representing the permanent-deformation behavior of a cohesive subgrade, then this means that axles having an overload of 50 percent, for example, will be equivalent to $(1.5)^{17} \approx 1000$ times as many 80-kN loads!

However, this exponent of 17 varied significantly from subgrade to subgrade, being as low as 10 or so and as high as 25 to 30. The effective exponent can also diverge somewhat because the wheel load is not necessarily proportional to the laboratory-suited deviator stress concept, but all in all the useful exponent, for significant magnitudes of permanent strain, is disproportionately high compared with the standard values used in design and analysis procedures the world over.

It should now be clear that the use of the concept of equivalent axle loads is misleading and irrelevant when applied to cohesive materials. A more appropriate approach would be to relate the permanent deformation behavior to the (very approximate) number of loads for the heaviest axle-load class expected in the design period, or alternatively to fix that class so that N $\approx$ 100 000 in accordance with Figure 8.

This research indicates, in fact, that the general fatigue type of approach implied in the equivalent-axle-load concept might be unsuitable for other nonbituminous road-building materials as well.

Other General Observations

Unfortunately, none of the other, more easily accessible parameters measured thus far could contribute any appreciable improvement to the above relationships. Tests are now under way to determine which type of field tests can be used and how (e.g., static plate loading, falling-weight deflectometer, lightweight deflectometer, hand shear vane, or CBR). Preliminary results have indicated that the first two procedures and, to a lesser extent, the intact CBR may be useful in determining a

Figure 9. Permissible actual and permissible fictive resilient strains (±1 standard deviation) based on laboratory triaxial tests of intact cohesive subgrades.

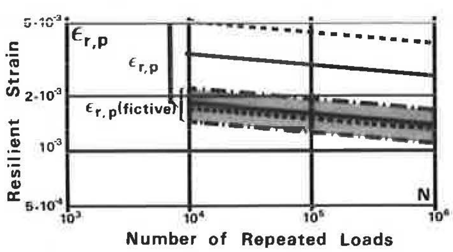
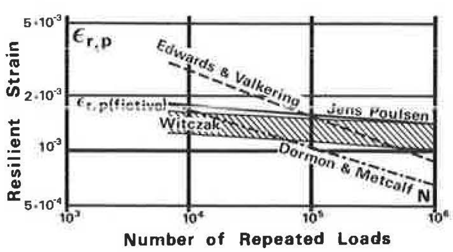


Figure 10. Resilient vertical strain criteria for subgrades.



reasonably close approximation to M_0. The moisture content—for the same type of soil—was obviously related to M_0, all other things being equal, and therefore to the bearing capacity of the soil, but the variations observed at test locations at which several samples were taken over a year's time span were too small to make any recommendations.

$\sigma_{dyn,f}$ was always (by definition) equal to or greater than $\sigma_{dyn,p}$ but, in many cases, it was more than twice as large. This phenomenon was mainly associated with embankment subgrades (e.g., AASHO) that demanded much higher deformations to mobilize their full strength. Nevertheless, the performances of these materials were similar to those of natural deposits in terms of permanent strains up to $\epsilon_p = 2$ percent relative to M_0, so that the general criteria derived apply equally well to both soil conditions.

The materials investigated included natural glacial deposits of both calcareous boulder clay and carbonate leached clay of the same origin. The latter type of soil dominates the upper ~1.5 m (5 ft) of soil in Denmark. Ahrentzen (7) has correctly maintained that the leached surface deposit is much weaker, and this investigation has reinforced his generally accepted assumption, which is considered in the planning, design, and construction of Danish highways. The low M_r-values seen in Figure 6 can be explained in this way. The M_0-values for natural deposits of calcareous clays were usually more than 50 MPa (7500 lbf/in^2), sometimes even more than 100 MPa (15 000 lbf/in^2 (with correspondingly higher values of $\sigma_{dyn,p}$, of course). Similar situations probably exist in many other places on a global scale and taking this into account can be very important from a structural pavement point of view, especially for secondary roads where the centerline profile lies close to the original terrain.

CONCLUSIONS

More research along these lines is necessary, especially because the relationships found thus far are quite different from those normally used in practice. The National Danish Road Laboratory intends to pursue these investigations to the point where the results and their implications can both be useful to the designer and describe reality in a sound and correct manner. The curves and relationships are, to be sure, somewhat premature although it appears that the fundamental concepts are basically sound.

Because of the appreciably higher bearing-capacity values implied in the approach discussed above, it is of utmost importance that we obtain a better understanding of the frost-susceptibility and soil-suction variations associated with cohesive materials so that we can evaluate more rationally the magnitude of the criterion input parameter(s) (e.g., M_0, a surface modulus, or related quantities) by reasonably simple field surveys. Although the design-criterion approach itself should in principle be applicable on a global scale, the methodology and safety factors incorporated in the specification of the input quantities will vary regionally according to, e.g., climatic conditions, and within a region according to such factors as road category. Investigations with these objectives in mind are thus included in the National Road Laboratory's current research and development plans.

Some general implications based on the above discussion are obvious. A design or analysis procedure in which all modes of pavement distress are based on the conventional "number of equivalent 8- or 10-ton axle loads" ultimately results in a misunderstanding because it begs the question. Especially in regard to cohesive materials, an ultimate-strength design procedure would be much more sound than a fatigue-oriented one. In light of the present state of the art, it would be overly optimistic to attempt to predict actual magnitudes of vertical distortion without access to a disproportionately large volume of information about each type of subgrade encountered along a roadway. Caution is thus urged when using, e.g., the highly refined VESYS performance-prediction systems. It is even doubtful whether a serviceability prediction approach is at all suitable for or desirable in practical pavement design and evaluation.

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