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Permanent Deformation of Subgrade Soils

(Phase I: A Test Protocol)

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13. ABSTRACT (Maximum 200 words)

Based on comprehensive literature review and repeated load testing on an Arkansas subgrade soil, a test protocol for permanent deformation of subgrade soils was established. This protocol is to be used in Phase II of this study to evaluate permanent deformation behavior of subgrade soils.

The basic configuration and test program proposed in this protocol include: a confining pressure of 21 kPa (3 psi), a repeated load frequency of 1 Hz, a rest period of 0.9 second, a load duration of 0.1 second, a limitation of load application of 10,000 repetitions. Three moisture levels: 105%, 110%, 120% of optimum moisture content, and three density levels: 90%, 95%, 100% of maximum dry density, will be used in fabrication of specimens with a kneading compactor. The proposed protocol will be used throughout the next phase research to test seven additional subgrade soils.

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1. Introduction

The major function of subgrade soils is to provide support to pavement structures. Under heavy traffic loads, subgrade soils may deform and contribute to distress in the overlying pavement structure. In asphalt pavements this distress normally takes the form of cracking and rutting. It has been well documented that the subgrade soil plays a critical role in the initiation and propagation of permanent deformation of pavement structures and directly influences pavement performance (Huang, 1993).

Deformation of subgrade soils can be divided into two parts: recoverable elastic deformation that is a measure of the resilient behavior and non-recoverable plastic deformation that indicates the absorbing behavior. Current pavement design procedures consider soil support characteristics in terms of its resilient behavior. These procedures ignore permanent deformation behavior even though it may be a very important component in pavement performance.

Conventional wisdom suggests that, under normally encountered pavement design conditions, permanent deformation is relatively insignificant and is adequately accounted for by consideration of the resilient behavior. This philosophy has resulted in a great deal of research over the past 20 years devoted to the definition of the resilient behavior of subgrade soils (Robnett and Thompson, 1976, Elliott et. al, 1988). As a result of this work the resilient modulus (M_R), an indicator of the resilient behavior of soils, has been successfully incorporated into the AASHTO design procedure for asphalt pavements [AASHTO 1986]. The focus of more recent research efforts has been on providing better interpretation of M_R and the incorporation of M_R into mechanistic design models.

On the other hand, only limited research has been conducted on the permanent deformation of subgrade soils. As a consequence, very little knowledge of permanent deformation of subgrade soils has been incorporated into the design of asphalt pavements. There is evidence to suggest that, in some instances, permanent deformation may have a much greater role in the life and performance of flexible pavements than designer currently recognize (Elliot and Thompson, 1985). The importance of permanent deformation in predicting performance of pavements seems to be more critical in thin pavements such as those encountered in rural or low volume roads. However, extensive experimental and theoretical work remains to be done before a potential parameter directly governing permanent deformation can be introduced for practical design consideration.

All pavement layers contribute to permanent deformation of the pavement structure. Yet subgrade does not attract as much attention as do the asphalt surface and granular base. The last two are man-made products in some sense and their properties are relatively well known and have been extensively studied for several decades. Actually, with the implementation of Superpave Level I Mixture Design method, the rutting of AC can be minimized through the proper selection of materials. As a result, subgrade becomes a weak point in the pavement structure.

Although the deforming behavior of subgrade soils is relatively difficult to identify and define, it is an essential factor in determining pavement structural performance. A scientific hypothesis supported by some researchers' work [Elliott and Thompson 1985] suggests that subgrade deformation not only directly governs rutting, but may have a strong relationship with cracking of the pavement's surface as well. Thus, subgrade soils contribute to the two main pavement distresses: rutting and cracking.

There are several reasons that permanent deformation has not received much attention. (1) Tests are tedious, time-consuming and expensive; (2) Most research efforts and expenses have gone to resilient behavior of subgrade soils during the last decade and at the same time people have a high expectation that flexible pavement design will be improved by better understanding of resilient modulus of soils; (3) Pavement distresses are thought to be highly dependently on resilient behavior rather than plastic behavior of subgrade soils; (4) Highway research has been mostly devoted to high-volume road ways where thick AC layer and/or base layer are generally used in design and subgrade permanent deformation is believed to be insignificant.

While there is still a need for further research in the area of resilient behavior, a great deal of significant research has already been conducted in this area. The testing protocol for determining M_R is fairly well established. Now it is important to quantify the impact of permanent deformation of subgrade soil on pavement performance so that both portions of a soils deformation can be properly incorporated into pavement design procedures. However, before any significant effort can be mounted to define the contribution of permanent deformation on pavement performance a testing protocol to predict permanent deformation of subgrade soil under typical traffic loading conditions must be developed.

Repeated loading or cyclic loading has been the well accepted test method in analyzing elastic and plastic deformation behavior of subgrade soils (Behzadi and Yandell, 1996). This testing procedure will serve as a starting point in this project.

2. Literature Review

2.1 Early Research

The research on deformation behavior of soils under repeated loading dates back to 1950s. Seed (Seed et. al, 1955, 1956, 1958, 1960) studied the effects of repeated loading on the strength and deformation of compacted clay. A silty clay (LL=37, PL=23) from Vicksburg, Mississippi was used for testing. After mixing, the soil specimens were cured for 24 hours before they were compacted using the Triaxial Institute Kneading Compactor. Compacted specimens were trimmed for testing to a diameter of 35.6 mm (1.4 inches) and a height of 71.2 mm(2.8 inches). The degree of saturation ranged from 92% to 97%. A dial indicator was used to measure the deformation of specimens. Water was used to provide confining pressure of 100 kPa(14.2 psi). The deviator stress was provided by an air pot and ranged from 200 kPa (28 psi) to 800 kPa (114psi). The original plan was to create a 0.1 second load duration which would represent the time of loading for a moving vehicle at 88 km/h (60 mph) (Seed et al 1955). However, the actual duration of deviator stress was set to 1 second due to limitations of the self-designed test apparatus (Seed et al 1955). The load duration was reduced to 0.2~0.33 second in later work when equipment modifications made that possible (Seed et al 1958). Specimens were subjected to around 100,000 load applications.

These pioneering and comprehensive research efforts produced some significant conclusions:

- For soils without thixotropic properties, deformation under repeated loads was observed to be independent of load frequency, provided that the applied stress is small enough not to change the soil structure and/or density, and the loading rate is within the range of 3 to 20 applications per minute.

- Repeated loading produced a gain of strength. The number of applications required to cause a strength increase was greater than 1,000, probably in the range of 10,000 to 100,000. This suggests that "a roadway grows with traffic"(Seed, Chan, 1958).
- High deviator stress could cause a specimen to fail relatively suddenly without previous excessive deformation.
- The resilient modulus increased as the stress increased, except when the applied stress was near the failure stress.
- For thixotropic soils, appreciable difference in deformation was observed for different load frequencies, but the difference was not consistent.

Early research by Larew and Leonards (Larew and Leonards, 1962) suggested the existence of a critical level of repeated deviator stress, σ_{rc}. This critical level is defined as the threshold stress state at which the slope of the deformation vs. load cycles curve remains constant after the first few load applications. For levels of deviator stress less than this critical level, the curve of deformation vs. load cycles would approach a horizontal asymptote. On the other hand, failure would be expected if specimens were subjected to a deviator stress higher than the critical level. Three soils were tested: micaceous silt, a mixture of limestone fragments and residual clay, and sandy clay. Static compaction was used to fabricate a sample cake 254 mm (10 inches) in diameter and 89 mm (3.50 inches) in high. Cylindrical specimens 71.2 mm (2.80 inches) high by 35.6 mm (1.40 inches) in diameter were then cut from the soil cake. Load frequency was controlled to be between 20 to 22 cpm. Most samples were subjected to 60,000 to 80,000 cycles of repeated load with a few of them subjected to over 400,000 repetitions. Water was used to provide confining pressure. The critical stress level was normalized by expressing it as a critical stress ratio, σ_{rc} over σ_{ult}. Larew and Leonards found that the critical stress ratio approached a minimum at or near optimum moisture content. The critical stress ratio for soils tested in this study was found to be above 70%. Similar reports by Ahmed and Larew (Ahmed and Larew, 1962) confirmed the existence of a critical stress ratio.

The research done during this period was very labor-intensive. The early prototypes of "in-house" loading devices mechanically alternated load, had highly compliant proving rings, and analog dial gages. These devices required constant monitoring to insure the load frequency and magnitude were correct and to manually record data. The versatility of these devices was limited and did not allow researchers to easily change test parameters or to accurately measure all of the data that was generated during a test. Due to these technical limitations, most deformation studies reported total deformation. Although some efforts were made to measure resilient deformation, in order to estimate the resilient, no attempt was made to separate permanent deformation from total deformation. Most analyses were able to establish only qualitative relationships between cyclic loading and deformation. No attempts were made to develop explicit constitutive equations to predict the performance of soil under repeated loading conditions.

Modern equipment and advanced technology make it possible to upgrade the research efforts for the permanent deformation of subgrade soils. The two major features enhancing modern research are servo-hydraulic or servo pneumatic loading devices and the use of strain gages and LVDTs for measuring load and deformation. Computer technology also make it possible to develop data acquisition equipment that will automatically capture and record load and deformation data as well as control the rate and intensity of loading. These advances in laboratory testing equipment have changed civil engineering research from a labor-intensive effort to a technology-intensive adventure.

2.2 Modern Research

In evaluating the rutting potential of base course materials Barksdale (Barksdale, 1972), tested silty sands and crushed stones. Specimens 71.2 mm(2.8 inches) in diameter and 153 mm(6 inches) high were subjected to an average of 100,000 load applications at confining pressures of 21 kPa (3 psi), 35 kPa(5 psi), 69 kPa(10 psi). Load frequency was set to 30 cycles per minute with a triangular shaped load function. The load ramped up to the peak value and then back to the trough value in period of 0.1 second, a 1.9 second period of no load separated the load applications. The deviator stress ranged from 1 to 6 times the confining pressure. Pneumatic pressure was used to load the specimens and an LVDT was used to measure deformation. The author suggested a reasonable range of repetition varying form 100,000 to 1,000,000 or more. From this work a qualitative rutting index was defined to evaluate pavement performance. Although no specific constitutive relationship between deformation and load applications was derived, the author did suggest the extrapolation of strain versus logarithm of load repetitions to save machine-hours.

The pioneering research work in the area of permanent deformation has always been credited to the efforts of Monismith et al (Monismith et. al, 1975). The power model proposed by Monismith is well accepted in predicting the amount of rutting contributed by the subgrade.

$$\varepsilon_p = AN^b \qquad (2\text{-}1)$$

where:

ε_p = permanent or plastic deformation

N = number of load repetitions

A, b = material parameters (regression coefficients from test data)

A silty clay with a LL=35 and PI=15 was used to develop this model. Static compaction was used to manufacture test specimens with a diameter of 2.8 in. and height of 6 in. The dry unit weight of these specimens was 90 to 95% of the maximum dry unit weight defined by ASHTO T99. Most of the specimens were subjected to 10,000 load repetitions with several receiving up to 100,000 applications. The applied load was supposed to be representative of traffic travelling at 32 km/h (20 mph) to 64 km/h(40 mph). A load duration of 0.1 sec., followed by a rest period created a cyclic frequency of 20 repetitions per minute.

One major finding of the Monismith study that other researchers have confirmed is that the exponent b depends only on soil type which also indicates parameter A plays the primary role in the introduction and development of subgrade permanent deformation. The tested soils had a b parameter between 0.154 to 0.332 and an A parameter between 0.0467 and 39.5. Obviously, the effect of factors such as applied stress history and moisture content had to be included into parameter A.

This power model has been referenced by most of the later researchers. Actually, Barksdale's (Barksdale, 1972) data could be represented using Equation (2-2).

$$\frac{\varepsilon_p}{\overline{\sigma}} = \frac{1/K\sigma_3^n}{1 - \dfrac{\overline{\sigma} R_f (1-\sin\phi)}{2(c \bullet \cos\phi + \sigma_3 \sin\phi)}} \bullet \left(\frac{N}{N_0}\right)^m \qquad (2\text{-}2)$$

where

$K\sigma_3$ = relationship defining the initial tangent modulus as a function of confining pressure

c, ϕ = cohesion, and angle of internal friction of granular materials

$\overline{\sigma} = equivalent - stresses,$
which is defined as $\{[(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_2)^2]/2\}^{1/2}$

R_f = Constant relating compressive strength to an asymptotic stress difference in which $0.75 \leq R_f \leq 1$

m = testing parameter

All these parameters are estimated from test data at a N_0 stress repetitions. The Coefficient m determines the rate of deformation accumulation while the magnitude of deformation was related to soil structure, strength, and stress condition which are characterized using parameters such as c, ϕ, R_f, and σ.

Hyde and Brown (Hyde and Brown, 1976) tested a Keuper Marl which is classified as a silty clay with a liquid limit of 32 and a plastic limit of 18. The soil was tested under creep loading and repeated loading. They found that the accumulation of plastic strain could be predicted using a relationship between strain rate and time:

(2-3)

$$\log \dot{\varepsilon} = \alpha - \lambda \log T$$

Where

λ = "decay constant", from regression of test data;

$\dot{\epsilon}$ = strain rate;

T = time in seconds ;

α = log(strain rate at unit time).

Hyde and Brown conducted a series of tests with rest periods of 1 sec and 10 sec to determine the effects of short rest periods between load applications. It was found that no significant change in strain rate could be expected from the inclusion of rest periods of different lengths. By comparing test data form repeated load and creep load, it was concluded that permanent deformation could be predicted with confidence from creep test data. The creep testing was done by sustaining a constant deviator stress over time.

Another popular rutting model has been termed the Ohio Model (Majidzadeh, 1978): Five soils were tested in the development of this model(three silty sands and two low-plasticity clays) The test specimens were created using a drop-hammer compaction technique that yielded specimens with a constant diameter of 71 mm(2.78 in.), but with a variable height of 145-152 mm(5.7-6.0 in.). Specimens were subjected to loading using a uniaxial dynamic testing technique. The accumulation rate of permanent deformation was related to the number of load applications using power model, Equation (2-4). The material properties were characterized by dynamic modulus of E^* (Actually, this is the same thing as M_R). E^* was supposed to account for the combined effects of moisture contents, density, and soil structure.

$$\varepsilon^p/N = AN^{-m}. \tag{2-4}$$

$$A = RE^{*-C}\exp(\sigma_{apl}/\sigma_{ult}) \tag{2-4a}$$

Where

A, m = rutting parameters

R, C = material constants

E^* = dynamic modulus of resilience

Actually, this model was updated from previous efforts where a similar equation was used to predict permanent deformation (Guirguis, 1974).

$$\varepsilon^p/N = A(D, W)N^{-m}. \tag{2-5}$$

Where:

m = absolute value of the slope of log ε_p/N versus log N

A(D, W) = rutting parameter depending on D(deviator stress) and W (water content), later related to dynamic modulus $|E^*|$ (Mojidzadeh, 1976).

$$A = K|E^*|^S \tag{2-5a}$$

K, S = parameters that depend on applied dynamic stress

It was found that the exponent m was nearly constant between 0.85 and 0.90. Basically, parameter A was interrelated to dynamic modulus E^* which would take care of such material characteristics as dry density, moisture content, and soil structure. Majidzadeh found this model to be valid for asphalt concrete surfaces and base courses as well as subgrade soils.

Poulsen conducted cyclic loading tests on undisturbed samples obtained from 16 sites located in six different countries, including the AASHO test site, (Poulsen, 1978).

Poulsen, tested at various loading frequencies and found that no significant differences in either permanent deformation or resilient deformation occurred when rest periods were greater than 0.33 seconds. A loading frequency of 2 Hz was selected for testing, with a load duration of 0.1 s and rest period of 0.4 seconds. Based on this testing Poulsen et al. proposed a model similar to equation (2-1),

$$\varepsilon^p = e_1 N^{e2} \beta^{e3} \tag{2-6}$$

where

e_1, e_2, and e_3 = material-characterization constants (These parameters were not based on a conventional regression method, but by minimizing the sum of the square of the absolute values of the deviations.

β = degree of failure , σ_{dyn} (applied deviator stress) / $\sigma_{dyn,f}$ (σ_{dyn} at triaxial failure for N=100,000)

During his PhD work, Lentz, tested a Michigan subgrade sand (C_u=1.8, G_s=2.62, γ_{dmax}=105pcf, OMC = 14%), (Lentz, 1979). All specimens were subjected to 10,000 cycles of deviator stress with a magnitude of 25% to 90% of static strength. One specimen was subjected to 100, 000 load applications. Load frequency was set to 1 Hz. Confining pressures of 34.5, 172, 345 kPa(5, 25, 50 psi) were used for testing. Lentz found that stress history had a significant effect on permanent deformation while moisture content had only a slight effect. Lentz related the permanent strain to the log of cycles of loading using Equation 2-7. Lentz found that Equation (2-7) fit his data better than Equation (2-1).

$$\varepsilon_p = a + b\,(\log N) \tag{2-7}$$

where

a, b = material parameters

$a = \varepsilon_{0.95Sd} \ln (1 - \sigma_d/S_d)^{-0.15}$

S_d = static strength of soils

$\varepsilon_{0.95Sd}$ = strain at 95% of soil strength

Diyaljee and Raymond (Diyaljee and Raymond, 1982) established a protocol to predict the permanent deformation under long term repeated loading using the static stress-strain data and a minimum number of cycles of repetitive load test data. Based upon their results from testing Conteau Dolomite railroad ballast under repeated load triaxial compression testing and data from other researchers, a series of formulas were developed for cohesionless soils that have the following general form:

$$\varepsilon^p = Be^{nX}N^m \qquad (2\text{-}8)$$

Where

$B = \varepsilon^p(N1, X_0)$, value of strain at X=0 for the first cycle

X = the ratio of the repeated deviator stress to the failure deviator stress under static loading.

n, m = regression parameter.

$$m = 0.0006\sigma_3 - 0.054 \quad \text{for Ottawa sand} \qquad (2\text{-}8a)$$

An example expression for subgrade sand (35 kPa of confining pressure) would be $\varepsilon^p = 0.004e^{4.07X}N^{0.12}$.

Diyaljee and Raymond found that different confining pressures do not cause appreciable change in strains for X below 60%. To get deformation of one-cycle repeated load under different deviator stress levels, results from progressive loading in increasing order on the same specimen is almost identical to those using separate specimens. One feature of the test program is the manual application of the first cycle load followed by repeated loading. No significant change in deformation was found for frequencies used in the test: 4, 6, 11 cycles per minute.

Allen and Deen(Allen and Deen, 1986) proposed a rutting model for use with all layers of a pavement structure. In developing this model, a soil with maximum dry density of 20.5 kN/m^3 (130.8lb/ft^3) at optimum moisture content of 9.7% was used. This soil was tested at three different confining pressures [34.5 kPa (5 psi), 69 kPa (10 psi), and 104 kPa (15 psi)] and subjected to three different deviator stresses; 17 kPa (2.5 psi), 35 kPa (5 psi), and 69 kPa (10 psi). Test specimens were 152 mm (6 in.) in height and 71 mm (2.8 in.) in diameter. Soils were tested at two different moisture contents: 8.2% and 9.4%. By fitting a third order polynomial to the data derived from their tests Allen and Deen proposed the following equation to predict permanent deformation:

$$\log \varepsilon_p = C_0 + C_1(\log N) + C_2(\log N)^2 + C_3(\log N)^3 \qquad (2\text{-}9)$$

where

C_0, C_1, C_2, and C_3 are coefficients dependent on material properties, stress state, and/or temperature.

For subgrade soils:

$$C_0 = [(-6.5+0.38w) - (1.1 \log \sigma_3)] + (1.86 \log\sigma_1)$$

$$C_1 = 10^{(-1.1+0.1w)}$$

$$C_2 = 0.018w$$

$$C_3 = 0.007 + 0.001w$$

w = moisture content (percent)

σ_3, σ_1 = confining pressure and deviator stress in psi.

Allen and Deen also developed a relationship between moisture content and CBR (California Bearing Ratio) that could be substituted into equation (2-9) for the soil tested.

$$w = 10^{[0.8633-0.05645(\log CBR)]}$$

Pumphrey and Lentz tested a Florida subgrade sand under repeated loading up to 10,250 cycles, (Pumphrey and Lentz, 1986). An inverted haversine wave was used for repeated load tests the load duration was 0.1-sec followed by a 0.9-sec rest period. Various combinations of confining stress, deviator stress, moisture content, dry unit weight were tested. Confining pressures of 34.5, 172.3, and 344.5 kPa(5, 25, 50 psi) were used during testing. The development of permanent strain was approximated by equation (2-10). Pumphrey and Lentz found that confining pressure did not cause significant changes in permanent deformation for low stress ratios (less than 0.60 to 0.75). However, substantial decreases of permanent deformation were observed for large stress ratios. The resilient modulus was found to have logarithmic dependency on the number of load applications. For some densities, moisture content was found to be a significant factor in the accumulation of permanent deformation. A technique for predicting permanent deformation from static triaxial test data was also refined based on previous work (Lentz, 1979).

$$\frac{\varepsilon_p / \varepsilon_{0.95S_d}}{\sigma_d / S_d} = n + m\frac{\varepsilon_p}{\varepsilon_{0.95S_d}} \qquad (2\text{-}10)$$

where

$$n = (0.809399 + 0.003769\sigma_3)\times10^{-4}$$

$$m = 0.856355 + 0.049650\ (\ln \sigma_3)$$

For the Florida sand tested, n = 0.1531, and m = 1.1941. For a Michigan sand tested by Lentz n and m were found to be 0.1970 and 0.9591, respecttively. (Lentz, 1979)

In a study by Raad and Zeid total accumulated strain, ε_a, was related to the repeated load stress level, q_r, and the number of load repetitions N. (Raad and Zeid, 1990) A silty clay (LL = 28, PI = 11) was used for testing. Stress pulses of a triangular shape and average duration of 0.2 sec were applied at a frequency of 10 cycles per minute for up to 10,000 cycles. The axial strain ε_a was defined as the sum of resilient strain and permanent strain. The concept of "threshold stress level" was introduced which governs the rate of change in axial strain $d\varepsilon_a/dN$.

$$q_r = \frac{\varepsilon_a}{a_L + s_L \log N} \qquad \text{when } q_L < q_{rL} \tag{2-11}$$

$$q_r = \frac{\varepsilon_a}{a_h + b_h \varepsilon_a} \qquad \text{when } q_L > q_{rL}$$

where

$$b_h = B_h + S_h \log N \tag{2-11a}$$

q_r = stress level, defined as the ratio of repeated load deviator stress to the ultimate static triaxial strength

q_{rL}= threshold stress level, obtained by repeated load testing and different from soil to soil, 0.80-0.90 for the tested soil

a_L, s_L, a_h, B_h, S_h = material parameters, obtained by conventional regression of repeated load test data

Cardoso and Witczak developed a methodology to predict the permanent deformation under aircraft loading on asphalt concrete pavement systems. (Cardoso and Witczak, 1991) Dynamic triaxial tests were conducted to establish models for predicting plastic deformation. Failure was assumed if the total permanent deformation of the pavement system exceeded 1.22 in. (31.0 mm). The model relates permanent strain to CBR value, stress state, and number of load applications.

$$\varepsilon_p = \frac{128{,}748(N)^{0.1346}(\sigma_1)^{2.664}}{(CBR)^{5.55}(\theta)^{1.1431}} \quad CBR>40$$
$$\varepsilon_p = \frac{(N)^{0.1878}(\sigma_1)^{6.0911}}{55.6313(CBR)^{1.3605}(\theta)^{4.893}} \quad CBR<40 \qquad (2\text{-}12)$$

Li, et al, derived an expanded power model to predict cumulative plastic deformation of subgrade soils (Li et. al, 1996). This model is based on original research as well as on data and mathematical formulations from others research. The original research was conducted at a railroad test track with a soft subgrade composed of Vicksburg Buckshot clay, (PI = 40-45, LL = 60-70). The track was subjected to repeated heavy axle loads (Li, et al 1996). It was found that measured settlements of the subgrade were consistent with the predicted settlements given by equation (2-13).

$$\varepsilon^p = \alpha N^b \beta^m \qquad (2\text{-}13)$$

where:

α, b, m = material constants. (Repeated load tests are required to determine these parameters based on regression of test data. Referenced values for several soils are given in a table that can be used for deformation prediction if sophisticated equipment is not available or affordable.)

$\beta = \sigma_d/\sigma_s$ (deviator stress/ static strength)

The major conclusion from this study was that b is relatively constant for the same soils and could be considered a function of the soil type alone. The parameter, β, was introduced to account for the influence of such factors as moisture content, density, and soil structure.

Bonaquist and Witczak(Bonaquist and Witczak, 1996) applied the theory of plasticity to rutting analyses. A flow theory with well-defined yielding surfaces was presented. A silty sand was selected for subgrade testing. Based on the results of this testing an incremental model was proposed to predict permanent strain:

$$\xi_N = N^{-1.06}\xi_I \tag{2-14}$$

where

ξ_N = permanent strain for load cycle N.

ξ_I =permanent strain for the first load cycle

The yielding surface of the plasticity model was defined as

$$\frac{J_2}{p_a^2} - \left[\gamma \left(\frac{I_1 + \frac{k}{\sqrt{\gamma}}}{p_a} \right)^2 - \frac{\alpha_1}{\xi^{\eta_1}} \left(\frac{I_1 + \frac{k}{\sqrt{\gamma}}}{p_a} \right)^n \right] = 0 \tag{2-14a}$$

where

J_2 = second invariant of the deviatoric stress tensor

p_a = atmospheric pressure

I_1 = first invariant of the stress tensor

a_1, η_1, n= material parameters

ξ = plastic strain trajectory

k, γ = Drucker-Prager material parameters

The material parameters defining yielding surface for the tested subgrade materials had the following values: $\gamma^{1/2} = 0.181$, $k = 17.0$ kPa, $a_1=0.00025$, $\eta_1=0.5$, $n=3.25$.

Equation (2-14) suggests that the cumulative permanent deformation has a strong relationship with the magnitude of the permanent strain induced on the first cycle. So, a reasonable criterion for rutting-control would be to limit the first-cycle permanent strain. If the allowable permanent deformation in the pavement structure over its design life is known, a critical yielding surface could be defined to determine the first-cycle permanent strain. Tabulated data were provided for the allowable first-cycle permanent strain, based on the selected crushed stone subbase, silty sand subgrade and minimum cover requirements.

Although no rigorous correlation between measured and predicted plastic deformations was conducted in this study, it is one of the few examples of a reasonable application of the theory of plasticity to subgrade soils.

Behzadi and Yandell conducted a comprehensive study on subgrade deformation. (Behzadi and Yandell, 1996) A silty clay subgrade material with LL=44 and PI=20 was used for testing. A floating mould compaction technique was used to create samples with a diameter of 101.6 mm (4 in.) and a height of 203.2 mm (8 in.) A rectangular wave form with a load duration of 0.5 sec and a rest period of 1 sec was applied using a universal testing machine. Test specimens were subjected to at least 10,000 load repetitions. Different stress combinations were used with confining pressures of 15, 30, 40, 50 kPa (2.2, 4.4, 5.8, 7.3 psi) and stress ratios (σ_1/σ_3) of 2.5, 5, 7. Behzadi and Yandell found that the data from the study could be best fit with the following equation:

$$\varepsilon_p = Ae^{B\sigma d}N^S \tag{2-15}$$

where

A, B = material constants

σ_d = deviator stress

S = slope of log ε_p versus log N

The parameter, S, was found to be independent of the state of stress and density and might only has some relationship with moisture content. The plastic strain of the first loading cycle was found to be dependent on the deviator stress, moisture content and density. A definite effect of load frequency on permanent deformation was not observed. However, permanent deformation increased with the increase of load frequency and loading duration. Data plotted on a log-log scale had a higher correlation coefficient than data plotted in a semi-log scale.

Several researchers (Allen, Thompson, 1974; Brown, Hyde, 1975) have explored the effect of variable (cyclic) confining pressure on the deformation of subgrade soils. Similar results for resilient and permanent strain were obtained from cyclic and constant confining pressure tests when the constant confining pressure was set to the average of the cyclic confining pressure (Brown, Hyde, 1975). This suggests that a fixed confining pressure could be used in repeated loading tests to get deformation data to simulate the cyclic confining effects of subgrade soils under moving vehicles.

2.3 Other Research

Besides the laboratory and field testing of subgrade soils, numerous numerical studies have been undertaken to predict permanent deformation of pavement structures. Although these research efforts were not test-oriented, they have borrowed more or less some ideas from laboratory researchers. All major numerical techniques, to include; the Boundary Element Method, BEM, and the Finite Element Method, FEM, have been utilized in these studies. A major advantage of numerical analysis is its capability to incorporate comprehensive constitutive laws of materials. Unfortunately, many researchers assume material properties rather than define those properties through comprehensive testing programs. Other techniques that have been utilized include digital mapping, image processing and neural networks.

Based on the Shell design method (Peattie, 1962), and Dorman (Dorman, 1962) suggested a permissible strain of 6.5×10^{-4} at the top of subgrade as a design criteria to prevent pavement failure due to excessive deformation of subgrade soils. In addition, a 1 MPa (45 psi) horizontal tensile stress was determined to be the maximum permissible stress on the lowest surface of the asphalt-bound layer. Provisional design charts were developed in accordance with these criteria.

Dorman and Metcalf (Dorman and Metcalf, 1965) developed a design chart for flexible pavements that related number of load applications to strains of pavement structures.

$$N = Min\left(\frac{a_1}{\varepsilon_{CVS}^{b_1}}, \frac{a_2}{\varepsilon_T^{b2}}\right) \tag{2-16}$$

Where

N = EASLs

a_1, a_2, b_1, b_2 = empirical constants. (Repeated load tests are required for asphalt concrete and subgrade soils to determine these regression parameters.)

ε_{CVS} = maximum vertical compressive strain on top of subgrade,

ε_T = horizontal tensile strain at the bottom of asphalt layer

This was the first effort to quantitatively relate subgrade strain to pavement life. These design criteria suggested that limiting elastic deformation of the subgrade soils could control total deformation of pavement structures. Equation 16 served as the basis for the various design methods that followed.

Edwards and Vakering conducted a numerical study that was based on data from the AASHO Road Test. They determined an average relationship between load applications and subgrade strain using the relationship in Equation 17 (Edwards and Vakering, 1974).

$$\varepsilon_3 = 2.8\times10^{-2}\times N^{-0.25} \tag{2-17}$$

where

ε_3 = permissible compressive strain in subgrade.

Many researchers have devoted their attention to pavement analysis using FEM, for the computation of pavement distress. These analyses normally provide a complete plastic solution to the deformation problem. The mechanistic equations are based on the theory of continuum mechanics and the material parameters needed as input for those equations are normally derived from constitutive relationships. For flexible pavements a number of different constitutive relationships must be developed to predict deformations and stresses in the various materials of the pavement structure. They include a viscoelastic asphalt concrete model, a nonlinear granular base/subbase model, and an elastoplastic subgrade model (Kirkner et al 1994). This approach for predicting pavement distress has the advantage of providing a theoretically rigorous solution procedure in a short period of time with very little expense. However the results are only as reliable as the material properties incorporated into the model. The prediction of pavement distress is a time-space-temperature-dependent problem and requires very sophisticated material properties. Unfortunately, many of the plastic properties of subgrade soils have not been quantified and are therefore assumed in these models. This requires substantial judgment on the part of the researcher to achieve reliable results from an analysis. Most of the work done in this field is principally related to theoretical mechanics and mathematics rather than to the development of material properties (Kirkner et al 1994, Zaghloul and White 1993). The validity and versatility of these methods is yet to be verified. If a database of subgrade properties were developed, the use of these numerical methods would be subject to less judgmental interpretation by the researcher

An interesting and promising technique for analyzing pavement rutting was reported by Simpson etc. (Simpson et al., 1995). Using the transverse profiles collected from LTPP (Long-Term Pavement Performance) Program, neural networks were utilized to distinguish rutting modes and facilitate performance prediction and model development.

The data set used in this analysis came from 152 sections of the LTPP GPS (General Pavement Studies). A 30mm (12 in.) wide transverse profile was taken at 15m (50ft.) intervals across the monitored lane width. Four categories of transverse profiles were assumed to represent the origin of rutting (a) subgrade rutting, (b) base rutting, (c) surface rutting, and (d) heave (environmentally-induced increases in soil volume). The algebraic area between the collected profile and the straight line connecting its end points was used to determine which category a transverse profile would fall in.

A neural network computer program called BrainMaker was used in these efforts. The entire data set as well as a data set categorized based on rutting origin was used as input to the program. Not surprisingly, it was found that categorized inputs could help obtain much better prediction models. The output is a neural network model, a program that is ready to accept inputs and produce rutting predictions.

2.4 Summary of Literature Review.

The following is a synopsis of the major findings of the various research reported in this literature review.

- Most researchers found that permanent deformation and log of the number of load applications have a linear relationship: $\varepsilon_p=AN^b$. This equation is widely accepted in practice, especially for cohesive subgrade soils.
- The exponent, b, is relatively stable and is mainly dependent on soil type. Stress state and soil physical state (density, moisture content) do not significantly change the value of b.
- Parameter A is dependent on applied stress, moisture content, density, etc. Since it is not a common convention to incorporate moisture content and density into prediction equations, another index could be used instead. Strength of a soil is a good indicator of moisture content and density of that soil.
- The accumulation of permanent strain for cohesionless subgrade soils, may be predicted using $\varepsilon_p = a + b\ (\log N)$.
- The first-cycle of deformation dominates the magnitude of permanent deformation. It is vitally important to identify and separate the deformation of soils under the first cycle of load application when conducting repeated load tests.
- Load duration of 0.1 sec. was usually used. The number of load applications ranged from 10,000 to 100,000.
- There exists a threshold stress for soils. Loading above this stress will cause failure after a small number of load applications. This suggests that an "endurance limit" or

"dynamic strength" could be a good indicator for evaluating permanent deformation behavior of subgrade soils.

- Most researchers used 20 to 30 cycles per minute as the load frequency. At least one researcher reported that the rest period did not affect the development of permanent strain as long as the rest period was greater than 0.33 sec. Another researcher found that frequencies in the range of 1 to 20 cpm did not affect the accumulation of specimen deformation, provided that the soil did not demonstrate thixotropic potential and the degree of saturation was not high.
- The compacted densities ranged from 90% to 100%. Moisture contents were either close to Plastic Limit and/or on the wet side of optimum moisture content.
- No information about the effect of freeze-thaw on soils was found.

A summary of test configurations and soil properties for all of the studies reported in this literature review. The initial testing conditions for this study were developed from a synthesis of the information presented in Table 2-1.

Table 2-1 Summary of the Test Configurations for Various Permanent Deformation Studies

	Seed et al (1958)	Larew & Leonards (1962)	Barksdale (1972)	Monismith et al (1975)	Poulsen el at (1979)	Lentz (1979)	Raad and Zeid (1990)	Behzadi & Yandell (1996)	This Project (1997)
Load Frequency (repetition per minute)	20	20-22	30	20	120	60	40	40	30
Load Duration (second)	0.2-0.33	1.4	0.1	0.1	0.1		0.2	0.5	0.1
Rest Period (second)	2.7	1.2	1.9	2.9	0.4		1.3	1	1.9
No. of Applications (in thousands)	100	60 to 80	100	10 or 100	100	10	10	10	100
Confining Pressure(kPa)	100	34.5, 69	20.7, 34.5, 69	34.5	34.5-69	34.5, 137.9, 345	100	15 to 50	0, 20.7, 41.4
Deviator stress (kPa)	up to 294	up to strength	1 t0 6 times σ_3	34.5 to 138		25% to 90% of Strength	.7 to .95 of strength	34.5 to 345	>= 41.4 up to failure
Sample Diameter(mm)	35.6	35.6	71.2	71.2	63.5	55	38	100	73
Sample Height (inch)	71.2	71.2	152.4	152.4	127	140	76.2	200	114
Soil Tested	Silty Sand PI=15 LL=37	Three Soils: Sand/Clay	Sand	Silty Clay PI = 15 LL = 35 OMC=13% γ_{dmax}=18.6kN/m^3	16 soils in-situ specimens	Sand c_u=1.8	Silty Clay PI=11 LL=28 OMC=8.5% γ_{dmax}=20.6kN/m^3	Silty Clay AASHTO T-99 PI=20 LL=44 OMC=14% γ_{dmax}=22.5kN/m^3	Silty Clay PI = 10 LL = 27 OMC=14.6% γ_{dmax}=18.2kN/m^3
Dry Density (% of γ_d)				90 to 95			98.5	90 to 100	95
Moisture Content(%)				16-20			7, 10	16	16
Note					Specimen size was estimated from[27]				

3. Soil Preparation and Test Program

3.1 Soil Processing

The soil used in this phase of project was taken from the east shoulder of Highway 365, Section 12, T13N, R14W, southeast corner of Faulkner County, Arkansas. The soil at this location is mapped as Gallion in the Faulkner County soils report (USDOA, 1979). Basically, two distinct layers of soils were found in this small Gallion area: 3.5-feet-thick top layer of deep gray clayey soil and one layer of reddish brown silty soil. To simulate use of the soil in a constructed subgrade, the two layers of material were mixed together prior to preparing test specimens.

The soil taken from the field was air-dried at room temperature in the laboratory. Rocks, twigs, and other deleterious materials were removed. During the air drying process large soil clods were broken into smaller clods by hand, but the average clod size could not be reduced below about 20 mm. To overcome this problem, soil aggregations were further reduced by placing the air-dried soils into a Los Angeles Abrasion chamber for mixing. Using the Los Angeles abrasion machine, without shot, allowed the rapid mixing of the two different soil types and reduced the small and hard clay lumps to individual particles.

3.2 Soil Properties

Soil properties were determined using conventional engineering soil tests: These properties are compared to those obtained for a similar soil described as Gallion which was used in a previous study of the resilient modulus of Arkansas subgrade soils. The comparison of results of the index property testing of the current test soil to those of the presumably similar soil from the previous study is presented in Table 3-1. All index tests indicate that the current soil is different from the Gallion soil reported in TRC-94(Elliott et.al, 1988). Gradation curve for the current test soil, obtained using AASHTO T-88, was illustrated in Figure 3-1.

Table 3-1 Comparison of Basic Soil Properties of Gallion Soil

Property	Specification	Current Study	Previous Study (TRC-94)
Liquid Limit	AASHTO T-265	27	67.9
Plastic Limit	AASHTO T-265	17	25.2
Optimum Moisture Content (%)	AASHTO T-99	14.56	25
Maximum Dry Density (kN/m^{-3})	AASHTO T-99	18.22	14.81
Specific Gravity	AASHTO T-100	2.6740	2.6199
% of Fines	AASHTO T-87, 88	78	
% of Clay Particles	AASHTO T-87, 88	22	55
AASHTO Classification	A-4		A-7-6
Unified Classification	ASTM D-2487	CL	CH

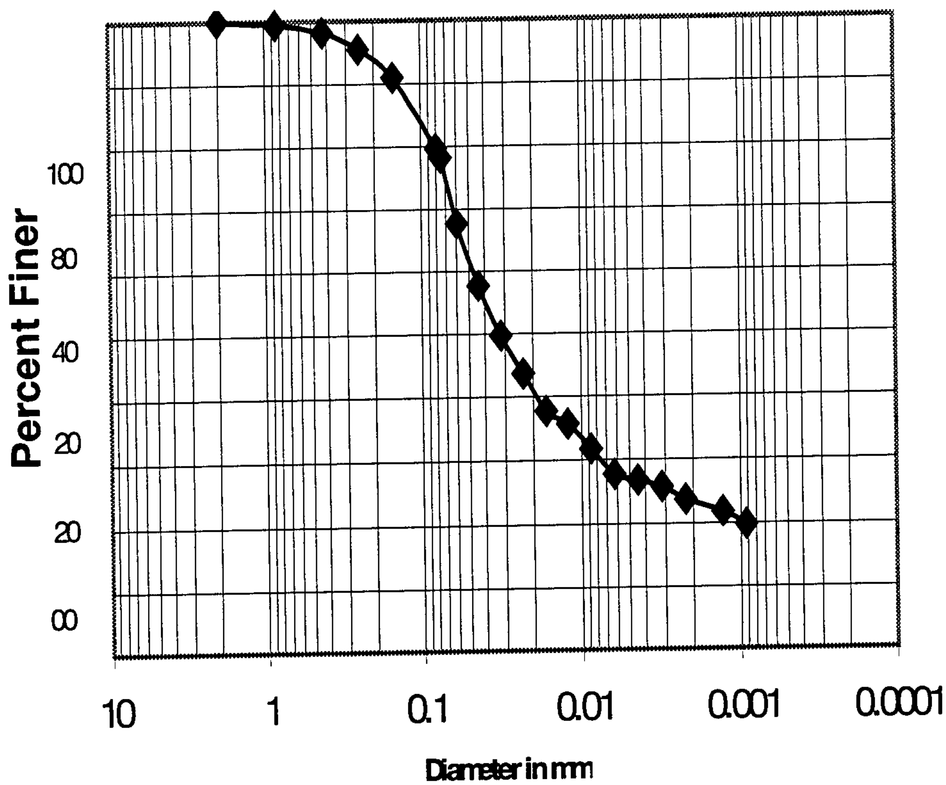


Figure 3-1 Gradation Curve for Gallion Soil

3.3 Fabrication of Specimens

3.3.1 Mixing

Compacted soil specimens were prepared at various moisture contents to simulate the range of moisture expected in a subgrade under a pavement. In an effort to insure uniform moisture distribution in each test specimen the following procedure was followed. Once the natural moisture content of the air-dried soil was determined, a sufficient volume of water was measured to produce the target moisture content plus two percent. An extra 2% of the calculated amount of water was added to account for moisture loss during mixing, weighing, curing, compaction, and other handling so that the actual target moisture content was achieved. Approximately 1kg of air dried soil was placed in a stainless steel the mixing bowl. Mixing water was slowly added while the soil was agitated with a stainless steel spatula. Once all of the water was thoroughly mixed with the soil, the combination was placed into a sealed plastic bag of 3.8 l (one-gallon-size) for 72 hours at room temperature for conditioning. Experience proved this time period to be adequate to achieve a uniform distribution of moisture throughout the soil specimen.

3.3.2 Compaction

Research has shown that different compaction methods impart differing soil structures to compacted specimens. The laboratory compaction method selected should replicate the soil structure of a subgrade both at the time of construction and later, when the road is actually in service. A major consideration in the design of the current testing protocol is the ability to measure both resilient modulus and permanent deformation in the same test. Therefore the compaction technique selected for this test protocol must be consistent with that prescribed for the resilient modulus test. The compaction methods suitable for use in resilient modulus testing, as recommended by AASHTO T-292-91, are presented in Table 3-2.

Table 3-2 Selection of Compaction Method for Laboratory Compacted Specimens

In-Place Conditions		Applicable Compaction Methods
Saturation at Time of Compaction %	Post-Construction in-service Moisture Content (MC)	
< 80	< MC at time of construction	Impact, Static, Kneading
> 80	>=MC at time of construction	Impact, Kneading
< 80	>MC at time of construction	Static

Source: (AASHTO T-292-91 I)

In a previous study on Arkansas subgrade soils Elliott concluded that the degree of saturation after compaction is greater than 80 percent for 75 to 80 percent of the soils in Arkansas, (Elliott, 1988). Based on Elliott's work and the information presented in Table 3-1, both kneading and impact compaction methods are acceptable. Elliott concluded that kneading compaction adequately replicated in situ conditions for Arkansas subgrade soils in his resilient modulus test program, (Elliott, 1988). Based on Elliott's work and other previous studies, kneading compaction was selected to fabricate specimens for this study. Figure 3-2 shows the kneading compactor used in this study. The mold for this compaction system is 101.6 mm (4 in.) in diameter and 127 mm (5 in.) in height.

Both the number of applications of the tamping foot and the system pressure of the kneading compactor affect the density of a compacted specimen. In order to achieve the target density, a trial-and-error approach is used to determine the combination tamping foot applications and kneading pressure. This combination changes for different soils and even for the same soil under different moisture contents. Several trials were normally required before the correct combination could be achieved.

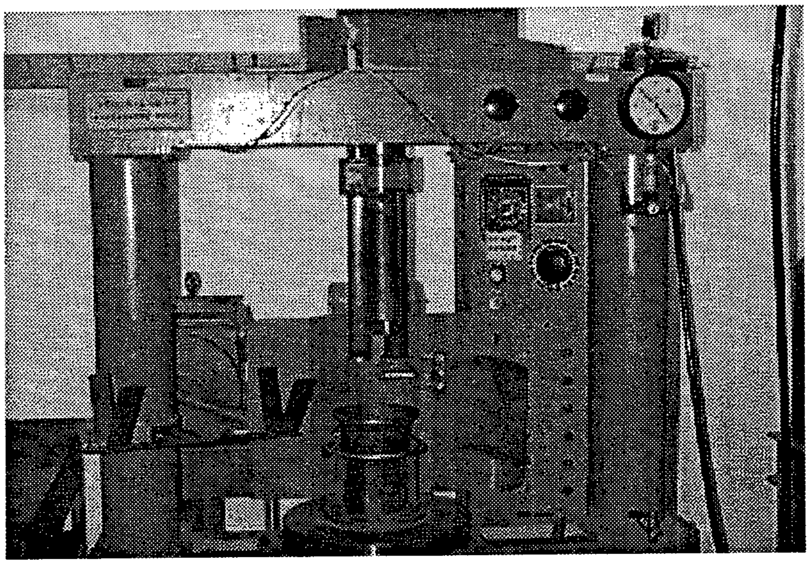


Figure 3-2 Kneading Compactor

Once the correct number of tamps and the magnitude of pressure were established, specimens were prepared in mass. A detailed compaction procedure is attached in Appendix B.

3.3.3 Extrusion

The test specimen was created by trimming the compacted soil in the mold using a portion of Shelby tube having a diameter of 73.025 mm (2.875 in.). Using a Universal Test Machine, (UTM), to supply the necessary force, the greased Shelby tube was forced into the soil in compaction mold. This process is much the same as used in the field to collect an undisturbed soil sample. After removing the tube form the mold, it was placed in an extrusion frame and a Teflon piston having a diameter nearly equal to that of the

Shelby tube was forced into the tube using the UTM. This extruded the soil specimen from the Shelby tube. An extruded specimen is shown in Figure 3-3.

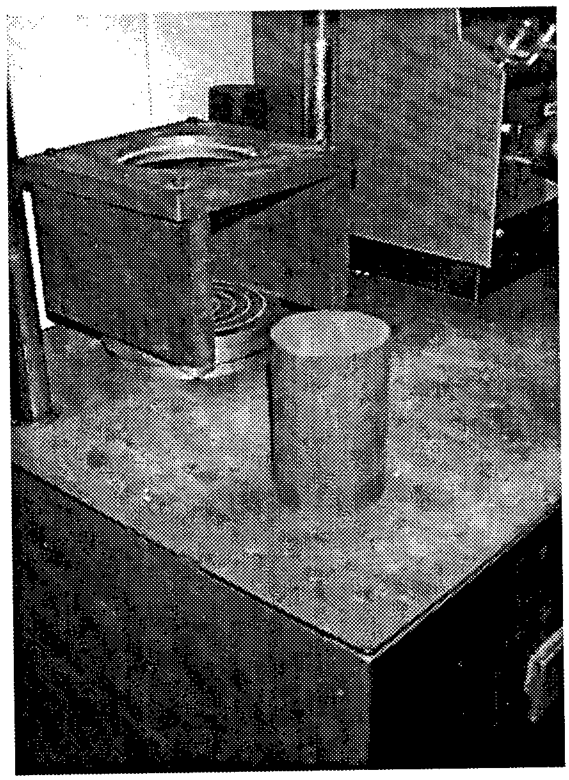


Figure 3-3 Soil Specimen

3.4 Triaxial Test

Conventional, unconsolidated-undrained, triaxial tests were conducted on unsaturated specimens prepared in the same manner as those for repeated load tests. The triaxial chamber was pressurized using air pressure and loaded under controlled strain conditions, using an MTS, UTM. Load and deformation were measured external to the chamber using a 4.45 kN (1000 lbf) load cell and a 12.7 mm (0.5 in.) LVDT, respectively. The tests were conducted at varying moisture contents to establish the effect of moisture content on the ultimate undrained monotonic strength of the soil. Figure 3-4 illustrated triaxial test results.

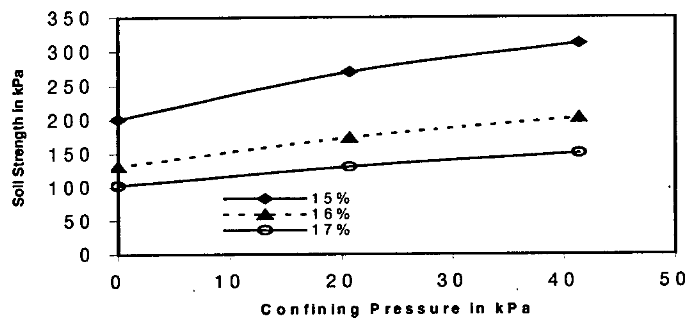


Figure 3-4 Triaxial Test (Data in Percent are MC)

3.5 Repeated Load Test Configuration

The same MTS machine was used as the loading device for all repeated load testing. The basic setup of this device is illustrated in Figure 3-5. All repeated load testes were conducted in a triaxial chamber. Pneumatic pressure was applied as confining pressure to a pre-determined magnitude. A cyclic deviator stress was supplied by the hydraulic actuator of the MTS device.

Initially, a strain gage-type deformation gage, having a range of 5.08 mm (0.2 in.) was attached between the loading piston and top plate of the triaxial chamber to measure the deformation of soil specimen. However, at high deviator stresses the deformation capacity of this gage was exceeded later and it was replaced with a LDT device having a deformation range of 25.4 mm (1.0 in.).

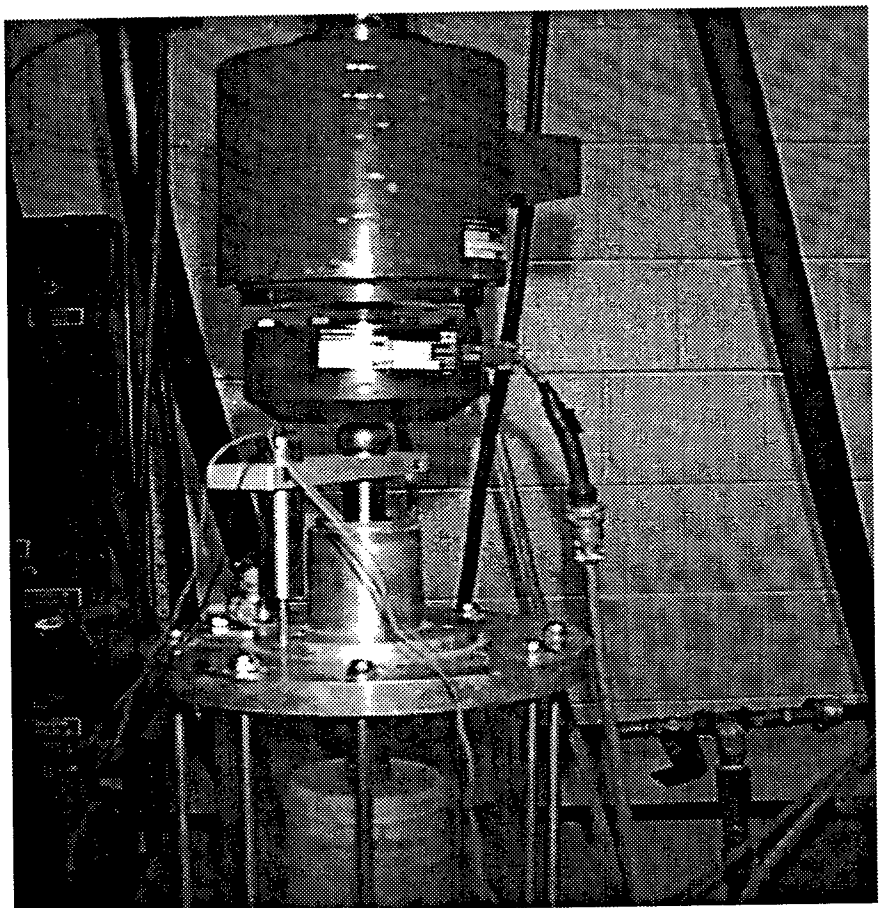


Figure 3-5 Test Setup

3.6 Data Acquisition

A HP-VEE program was written to capture data points. HP-VEE is a powerful visual programming language, a product of ComputerBoards, now becoming Virtual Instrumentation. The data acquisition program is a mouse-driven GUI application (Figure

3-6). This program makes it possible to collect data at user-defined intervals. Four values form a data set for each sample time: maximum load, minimum load, maximum deformation, and minimum deformation. It takes 5 seconds to capture a complete data set. Data acquisition was developed as a two-stage process: 1000 consecutive data points at the beginning of the test followed by points at 600-second intervals for the remain of the test.

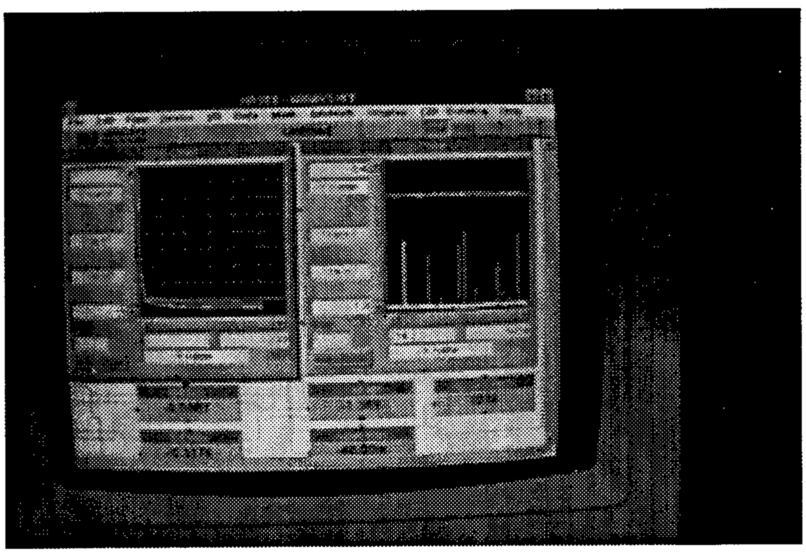


Figure 3-6 Data Acquisition Monitor

3.7 Test Program

In order to establish a repeatable and reliable test methodology for the permanent deformation test, factors such as loading frequency, confining pressure, load applications, stress path, moisture content and deviator stress will be investigated. The major goal is to zero in those primary factors that play important roles in the accumulation of permanent deformation of subgrade soils. On the other hand, those insignificant factors will be excluded in the next phase research to keep the test procedures simple and reliable

Based on the literature review, the following configurations were selected for the initial tests:

Soils: moisture content = 15.3 % (105 % of OMC), dry density = $0.95\Upsilon_{dmax}$

Confining Pressure, 3 levels: 0 kPa (0psi), 21 (3 psi), 41 kPa (6 psi).

Load frequency: 30 cycles per minute.

Load duration: 0.1 second.

Rest period: 0.9 second.

Load applications: 100,000.

Deviator stress, 3 levels: 41 kPa (6 psi), 62 kPa (9 psi), and 82.74 kPa (12 psi).

In addition to the default configuration, two more different load frequencies, 60 and 120 cycles per minute were used to see if the machine-hours could be reduced. However, the load duration remained constant as 0.1 second. This load duration represents a great range of load pulses that subgrade soils are subjected to under moving traffic conditions (Barksdale, 1971).

The effect of stress history on deformation behavior was also explored. After a specimen was tested under a lower deviator stress, a higher deviator stress would be applied for another repeated load test. The purpose of this testing was to determine if a single specimen could be used to test for permanent deformation at more than one deviator stress level. If a single specimen could be used, time could be saved and the influence of variability between specimens could be reduced.

3.8 Test Procedure

Detailed procedure was attached in Appendix C.

4. Test Results

Most of the tests in this phase of study were run up to 100,000 load repetitions. However, one specimen was subjected to 1,600,000 load applications to examine the accumulation of permanent deformation at higher numbers of applications. The load duration used for all tests was 0.1 second.

Test results for all repeated load testing are presented in Appendix A.

Presented in Figures A-1, A-2, A-3, A-4 are the results of permanent deformation versus load application for various confining pressures and deviator stresses, using arithmetic scales. Figures A-1a, A-2a, A-3a, A-4a present the same data using logarithmic scales. Figure A-1b illustrates an attempt to plot the data corresponding to figure A-1 on a semi-log scale (arithmetic for axial permanent strain and logarithm for load repetitions).

Figures A-1 and A-1a present the test data for a soil specimen subjected to 41 kpa (6 psi) confining pressure and a deviator stress of 62 kpa (9 psi). The ultimate number load cycles was 1,600,000. The rest period was 0.5 sec. If the first 100 cycles of loading are ignored in Figure A-1 the permanent deformation and total deformation formed two parallel lines.

Figure A-2 and A-2a presents data set for a specimen subjected to 21 kpa (3 psi) of confining pressure and 41 kpa (6 psi) of deviator stress. The specimen was subjected to 90,000 load applications with rest period of 1.9 sec and load duration of 0.1 second.

Figure A-3 and A-3a are the test results for a specimen subjected to 62 kpa (9 psi) of deviator stress with no confining pressure. The specimen was subjected to 120,000 load applications with rest period of 0.9 seconds and load duration of 0.1 second.

Figure A-4 and A-4a illustrate the results of a specimen with a stress history of 21 kpa (3 psi) of confining pressure and 41 kpa (6 psi) of deviator stress. The specimen was then subjected to an increased deviator stress of 83 kpa (12 psi) while the confining pressure remained unchanged. Permanent deformation and total deformation observed the same general trend as those without previous stress history.

Figures A-5 through Figure A-8 present the results of a series of tests conducted at three different load frequencies; 2 seconds, 1 second and 0.5 seconds. The confining pressure varied from 21 kPa to 41 kPa (3 to 6 psi) and the deviator stress was 41 kPa and 62 kPa (6 and 9 psi). Deformation versus load application is presented on an arithmetic scale.

Figure A-9 to Figure A-16 illustrates the effects of varied confining pressure. The deviator stress was 41, 62 , or 83 kPa (6, 9, or 12 psi) while the confining pressure was 21 or 41 kPa (3 or 6 psi). Each figure illustrates the effects of increasing confining pressure for a particular deviator stress.

Illustrated in Figures 17 through Figure 20 are test results showing the effect of varying deviator stress under constant confining pressures. Again, deviator stresses of 41, 62, and 83 kPa (6, 9 and 12 psi) were used and confining pressures were maintained at 41 and 62 kPa (3 and 6 psi).

The investigation of stress history is summarized in Figure A-21 to Figure A-24. These figures illustrate the affects of previous loading history on permanent deformation of the test specimen. One virgin specimen and another specimen subjected to previous loading history were tested under the same deviator stress and the results were plotted in the same figure. Take the data presented in Figure A-21 as an example, one test specimen was subjected to a deviator stress of 62 kPa (9 psi) and a confining pressure of 21 kPa (3 psi). The second specimen was subjected to the same deviator and confining pressures but it had previously been tested to 10,000 load cycles at a deviator stress of 41 kPa (6 psi.)

Figure A-25 illustrates the effects of moisture contents on the accumulation of permanent deformation. Three different moisture contents: 105%, 110%, and 120% of OMC, were used to demonstrate the critical role of moisture content on the accumulation of permanent deformation.

5. Data Analysis

5.1 General Trend of Deformation Behavior

A large part of the total sample deformation was developed during the first 100 cycles of the test. The general relationship between axial strain and log of the number of load application is approximately linear. The linearity of this relationship improves if the first 100 data points are ignored. The initial data points are not entirely representative of the soil's behavior because the loading system is being tuned to apply the correct deviator stress during the first 10 to 30 cycles of load. This early loading oscillation has residual effects on some later load cycles. However, by 100 load cycles the system seems to be operating under steady state conditions.

Both total deformation and permanent deformation appear to observe the same trend under the repeated loading. Figures A-1 through A-4 illustrate that the data produce two nearly parallel lines in a log $\varepsilon_p \sim$ log N or $\varepsilon_p \sim$ log N plot. Inspection of Figures A-1 through A-25 illustrates that this general trend is independent of stress configuration, stress history and rest period.

5.2 Analysis of Deformation Development

Regression analyses were conducted for the development of permanent strain and total axial strain. Regression results for Figure A-1a, b are summarized in Table 5-1. Regression results relating to Figure A-2b are presented in Table 5-2.

Table 5-1 Regression Results for Permanent and Total Deformation (Figure A-1)

	Permanent		Total	
	Log-Log	Semi-Log	Log-Log	Semi-Log
R^2	0.9593	0.9802	0.9424	0.9637
Intercept	0.3142	1.9650	0.3597	2.2222
Slope	0.0350	0.2234	0.0289	0.1935
Standard Error for R^2	0.0086	0.0380	0.0085	0.0229
Standard Error for Intercept	0.0018	0.0077	0.0017	0.0091
Standard Error for Slope	0.0004	0.0002	0.0004	0.0022

Table 5-2 Regression Results for Permanent and Total Deformation (Figure A-2a,b)

	Permanent		Total	
	Full-Log	Half-Log	Full-Log	Half-Log
R^2	0.9975	0.9955	0.9959	0.9921
Intercept	0.0257	1.0215	-0.0340	0.8708
Slope	0.0350	0.1110	0.0427	0.1251
Standard Error for R^2	0.0016	0.0065	0.0024	0.0099
Standard Error for Intercept	0.0005	0.0023	0.0009	0.0035
Standard Error for Slope	0.0002	0.0007	0.0003	0.0011

Interpretation of the regression information presented in Tables 5-1 and 5-2 indicate that the deformation data can be well-fitted with a linear relationship of log $\varepsilon_p \sim$ log N(log-log, Figure A-1a) or $\varepsilon_p \sim$ log N(semi-log, Figure A-1b) plot. Note that this is consistent with the power model first proposed by Monismith, in Equation (2-1) (Monismith et. Al, 1975). Regression results for other tests produce R^2 values for both semi-log and log-log that were in close agreement. In some tests the semi-log plots produced higher R^2 values while in others the log-log plot produced the higher value. Since no solid conclusions can be made about which fitting method produces higher R^2 values, the log-log fitting parameters will be used for further analyses.

The permanent deformation under the first load application for the load tests reported in Fig. A-1a constituted 64 % of total permanent deformation that was accumulated after 1,600,000 applications. The accumulated permanent deformation, expressed as a percent of the total permanent deformation at 1,600,000 cycles of load, for first 100, 500, 1000, and 10000 load applications, was 75, 83, 85, and 90 respectively.

Because permanent deformation under the first several load applications makes up a large portion of potential permanent deformation, it is essential to get the deformation data under first few repeated loads. Unfortunately, it is not easy to isolate and identify these data considering the gradual increase of the applied load from zero to specified magnitude and the oscillating nature of the self-balance mechanism of MTS machine, especially at high load frequencies.

A practical way to eliminate any unreasonable data generated under the first few load applications is to remove these data points from the analysis and extrapolate them from the regression. In this research program, only data after 100 applications will be used in determining regression coefficients. The deformation data for load applications between 1 to 100 can then be extrapolated using the regression equations.

The regression coefficients for the data presented in Fig. A-1a are listed in Table 5-3. Two sets of regression coefficients were generated. One with all data points include and another with the first 100 data points eliminated. All regression coefficients improved when the first 100 load applications were excluded from the data set.

Table 5-3 Refined Regression Results for Permanent Deformation

	Excluding first 100 applications	Using All Data
R^2	0.9774	0.9593
Intercept	0.3240	0.3142
Slope	0.0328	0.0350
Standard Error for R^2	0.0056	0.0086
Standard Error for Intercept	0.0013	0.0018
Standard Error for Slope	0.0003	0.0004

5.3 Permanent Deformation Under First Repetition

Testing by other researchers, (Behzadi and Yandell, 1996; Lentz, 1979; Bonaquist and Witczak, 1996) indicated that the first couple of load repetitions would produce a large portion of potential deformation over a large number of load applications. To explore the exact magnitude of the first cycle deformation a series of tests were conducted at a confining pressure of 21 kPa (3 psi) and deviator stresses varying from 41 to 145 kPa (6 to 21 psi). Table 5-4 lists the percentage of permanent deformation under specific load repetitions based on a total of 10,000 applications.

Inspection of Table 5.4 indicates that the first load produced an average of 55% of the permanent deformation accumulated over 10,000 repetitions. The percentage at 10, and 100 repetition was 70% and 87%, respectively.

An obvious trend is the increasing percentage of first cycle deformation with increasing deviator stress. A linear model may be proposed as Equation (5-1): (Note: to be refined)

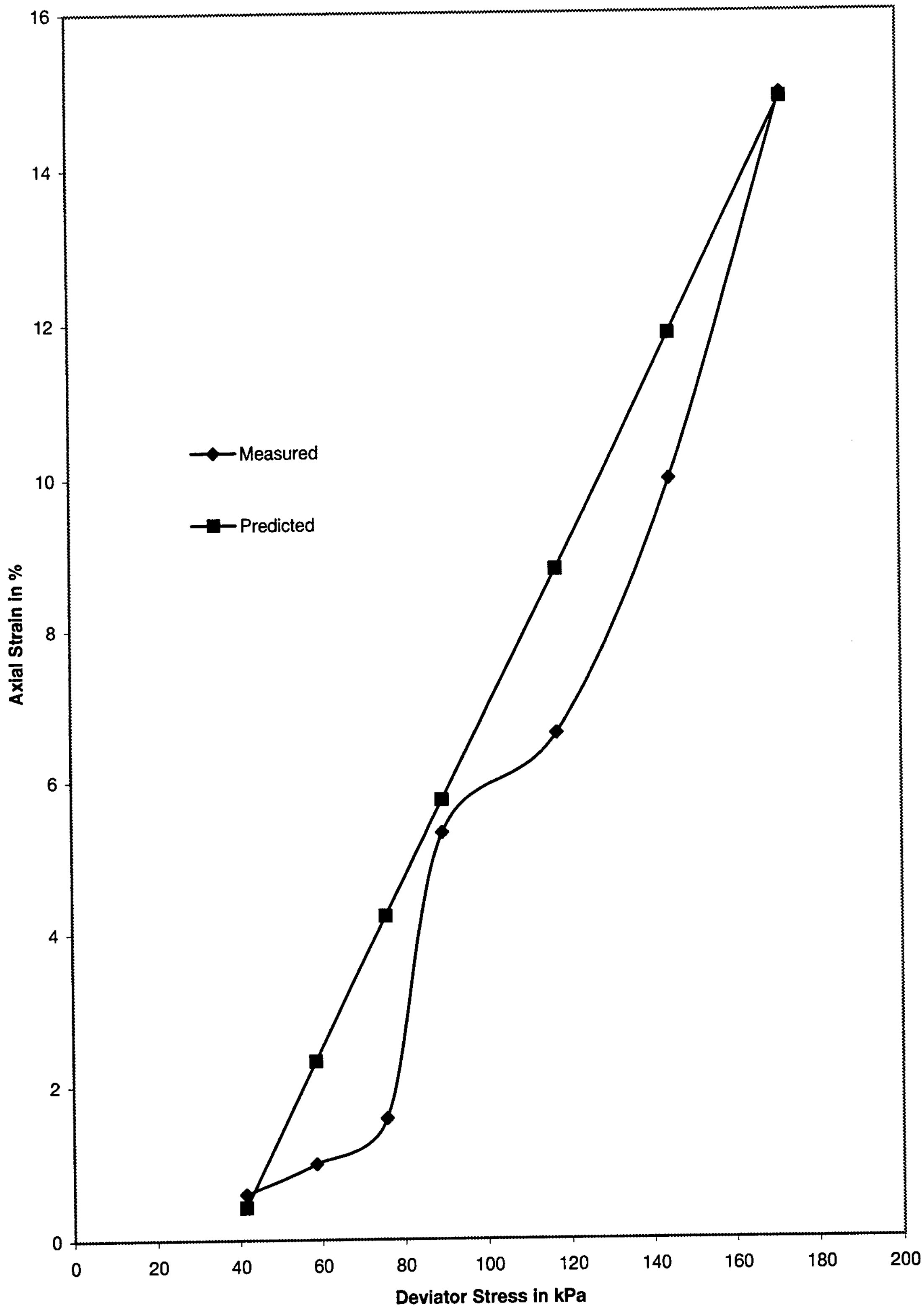
$$\varepsilon_1^p = -4.4193 + 0.7618\,\sigma_d \qquad R^2 = 0.9571 \qquad (5\text{-}1)$$

Equation (5-1) suggests that permanent deformation will accumulate faster under higher deviator stress than under lower deviator stress. Figure 5-1 presents a comparison of the actual measured deformation and predicted deformation using Equation (5-1)

Table 5-4 Accumulation of Permanent Deformation at Various Load Cycles, Expressed as a Percentage of Deformation at 10, 000 Cycles

Deviator Stress	1st	10th	100th	1,000th	10,000th
6	54	66	83	92	100
8.5	49	65	82	92	100
11	36	59	80	92	100
13	59	74	91	98	100
17	61	77	92	97	100
21	68	76	92	98	100
Average	55	69	87	95	100

Figure 5-1 Comparison of Predicted and Measured First-Cycle Deformation



5.3 Number of Load Applications

Permanent deformation testing is expensive and time-consuming work. If the number of load applications can be reduced to a point where usable information about permanent deformation can be extracted from a limited data set, the expense and time can be reduced greatly.

Table 5-5 lists the regression equations using different data sets from the test presented in Fig. A-1a. Detailed regression coefficients are summarized in Table 5-6. Using the equations in Table 5-6, predicted permanent deformation can be calculated as a function of load applications. Measured and predicted data are summarized in Table 5-7.

Table 5-5 Summary of Regression Equations

Date Range	Number of Equation	Equation $\text{Log}(\varepsilon^P) = a + b \times \log N$
Range 1 *Using All Data* (1600000 repetitions)	(5-2)	$\text{Log}(\varepsilon^P) = 0.3142 + 0.0350 \times \log N$
Range 2 (100 to 1.6E6 repetitions)	(5-3)	$\text{Log}(\varepsilon^P) = 0.3240 + 0.0328 \times \log N$
Range 3 (100 to 1.0E6 repetitions)	(5-4)	$\text{Log}(\varepsilon^P) = 0.3199 + 0.0340 \times \log N$
Range 4 (100 to 1.0E5 repetitions)	(5-5)	$\text{Log}(\varepsilon^P) = 0.3134 + 0.0360 \times \log N$
Range 5 (100 to 1.0E4 repetitions)	(5-6)	$\text{Log}(\varepsilon^P) = 0.3049 + 0.0389 \times \log N$
Range 6 (100 to 5000 repetitions)	(5-7)	$\text{Log}(\varepsilon^P) = 0.2933 + 0.0432 \times \log N$
Range 7 (100 to 1000 repetitions)	(5-8)	$\text{Log}(\varepsilon^P) = 0.2525 + 0.0588 \times \log N$
Range 8 (100 to 500 repetitions)	(5-9)	$\text{Log}(\varepsilon^P) = 0.2310 + 0.0678 \times \log N$

Table 5-6 Regression Results Using Different Data Sets

	Range 1 *Using All Data* (1600000)	Range 2 (100 to 1.6E6)	Range 3 (100 to 1.0E6)	Range 4 (100 to 1.0E5)	Range 5 (100 to 1.0E4)	Range 6 (100 to 5000)	Range 7 (100 to 1000)	Range 8 (100 to 500)
R^2	0.9593	0.9774	0.9803	0.9837	0.9626	0.9496	0.9855	0.9969
Intercept	0.3142	0.3240	0.3199	0.3134	0.3049	0.2933	0.2525	0.2310
Slope	0.0350	0.0328	0.0340	0.0360	0.0389	0.0432	0.0588	0.0678
Standard Error for R^2	0.0086	0.0056	0.0056	0.0041	0.0045	0.0044	0.0019	0.0008
Standard Error for Intercept	0.0018	0.0013	0.0012	0.0011	0.0019	0.0026	0.0021	0.0015
Standard Error for Slope	0.0004	0.0003	0.0003	0.0003	0.0006	0.0009	0.0008	0.0006

Table 5-6 Regression Results Using Different Data Sets

	Range 1 *Using All Data* (1600000)	Range 2 (100 to 1.6E6)	Range 3 (100 to 1.0E6)	Range 4 (100 to 1.0E5)	Range 5 (100 to 1.0E4)	Range 6 (100 to 5000)	Range 7 (100 to 1000)	Range 8 (100 to 500)
R^2	0.9593	0.9774	0.9803	0.9837	0.9626	0.9496	0.9855	0.9969
Intercept	0.3142	0.3240	0.3199	0.3134	0.3049	0.2933	0.2525	0.2310
Slope	0.0350	0.0328	0.0340	0.0360	0.0389	0.0432	0.0588	0.0678
Standard Error for R^2	0.0086	0.0056	0.0056	0.0041	0.0045	0.0044	0.0019	0.0008
Standard Error for Intercept	0.0018	0.0013	0.0012	0.0011	0.0019	0.0026	0.0021	0.0015
Standard Error for Slope	0.0004	0.0003	0.0003	0.0003	0.0006	0.0009	0.0008	0.0006

The measured permanent deformation after 1,600,000 applications is 3.29% while the predicted value using Equation 5-4 (Table 5.6) is 3.64% which is 13% higher than the measured deformation. If repeated load test was terminated after 5,000 application, the data gathered could be used to predict deformation under higher number of application with reasonable and conservative confidence.

A comparison of measured deformation versus predicted deformation, using all of the fitting equations of Table 5-6 are presented in Figures 5-2 and 5-2a. All data are for the load test described in Fig A-1a. Equations (5-3) and (5-4) where exclude the first 100 cycle-points give better predictions than the others do. It is not unexpected that Equation (5-3) which is based on the whole data set except the first hundred cycles gives the best estimation. Equation (5-5) is a regression formula based on 100 to 10,000 load applications. This equation predicts deformation accumulated after 1,600,000 cycles with good precision. The predicted deformation for 100,000 cycles using Equation (5-5) is 3.45% as compared to the measured magnitude of 3.2412%. In this case, the measured deformation is 6.6% lower than predicted value. Based on these results, it is concluded that for practical purposes, testing can be concluded after 10,000 load applications.

5.4 Rest Period

Three rest periods used during the testing were: 0.4 second, 0.9 second, and 1.9 seconds. These rest periods, coupled with the 0.1 second load duration, resulted in load frequencies of 120 cpm, 60 cpm, and 30 cpm respectively. The literature review found that a frequency of 30 cpm was used often. However, there was little rationale given for the selection of that frequency. Perhaps the major reason for selecting low loading frequencies in the earlier research was the inability of the early testing equipment to

Figure 5-2

Figure 5-2a

operate at higher frequencies. For a regular test with a rest period of 2.0 sec. and load frequencies of 30 cpm, it would take 55 hours to subject a specimen to 100,000 load applications. It would be more economical and much less time-consuming if the rest period could be reduced to a smaller value while still achieving reasonable results. Figure A-5 and A-6 presented results with deviator stress set to 41 kpa (6 psi) using different rest periods. The confining pressures are 21 kpa (3 psi) and 41 kpa (6 psi), respectively. Obviously, the difference among the data sets under different rest periods is not big enough to make clear claim. Therefore, it is a reasonable postulation that load frequency in the range used for these tests does not affect the permanent deformation response of soils under repeated loads, at least under this deviator stress.

Figure A-7 and A-8. Present data for tests where the confining pressures are 21 kpa (3 psi) and 41 kpa (6 psi), respectively, and the deviator stress is 62 kpa (9 psi).For this higher deviator stress, the difference in deformation is great enough to assume that higher frequency would cause greater deformation. It was recommended that a frequency of 60 cycle per minute should be used in the next phase research.

5.5 Confining Pressure

Figure A-9 to Figure A-15 present data illustrating the difference in deformation as a result of varying confining pressures. For most tests shown in Figure A-9, A-10, A-11, A-13, A-14, A-15, the confining pressure does not affect the deformation behavior significantly. As illustrated in Figures A-10, A-12, A-14, A-15, a relatively small decrease in deformation would be expected for higher confining pressures. This decrease in deformation can be attributed to the increased strength of the specimen as a result of higher confining pressure. However, when the deviator stress is low, the consolidation effect of the confining pressure would offset the increased resistance of specimen due to confining pressure as shown in Figure A-9, A-11, A-13. Additionally, when the deviator stress is low and confining pressure is high, the consolidation effects would exceed the increased resistance. As illustrated in Fig. A-16, the same characteristic was also observed for specimens that were subjected to previous loading history

Deviator stress plays a dominate role in the accumulation of axial strain as shown in Figure A-17 to A-20. When the deviator stress was increased from 41 kpa (6 psi) to 83 kpa (12 psi), the permanent deformation nearly doubled. Also, from Figure A-19 and Figure A-20, it is quite obvious that the increase of deformation was much greater when the deviator stress was increased form 41 kpa (6 psi) to 62 kpa (9 psi) compared to when it was increased from 62 kpa (9 psi) to 83 kpa (12 psi).

5.7 Stress History

Figures A-21 to A-24 illustrates the effects of stress history on the deformation behavior of soils. These figures suggest the work-hardening effects of stress history. Previous loading at lower deviator stresses increased the soil's resistance to deformation under repeated loads at high deviator stress as suggested by Seed et al(Seed et. al, 1955). The work hardening of previous load application made the specimen more solid and less prone to permanent deformation. Base on this observation, it does not appear possible to test a single specimen at several deviator stress levels

5.8 Dynamic Strength

Figure A-19 compares permanent deformation as a function of load applications for tests with varying deviator stresses. With increasing deviator stress, the accumulation of deformation tends to stay the same until the applied deviator stress approaches the dynamic strength of the soil, about 67.5% of its static strength. When the applied deviator stress reaches dynamic strength, the specimen fails with only a couple of load applications. While the exact number of repetitions to cause failure is not easy to identify, it is within the first 100 load cycles.

Permanent deformation data for different deviator stress up to failure are summarized in Table 5-8, and Table 5-9 lists the regression data for these test series. Inspection of Table 5-8 reveals that the slope of the power model remains unchanged for applied stress up to 11 psi. The average slope is 0.4187. The magnitude of the slope dropped from 0.43 to about 0.2 as the deviator stress increased. This, in part, explained why permanent deformation develops earlier under high deviator stress than under low deviator stress.

5.9 Moisture contents

The effects of moisture content on the development of permanent deformation are illustrated in Figure A-25. Obviously, moisture content is a significant factor contributing to the accumulation of deformation. Comparing Figure A-19 and Figure A-25, it was found that the specimen having a moisture content of 120% of OMC and subjected 34.5 kPa (5 psi) developed more deformation than a specimen having a moisture content of 105% of OMC and subjected to 75.9 kPa (11 psi).

Table 5-8 Measured Permanent Deformation under Repeated Loads (in %)

Deviator Stress	@ 1st	@ 10th	@ 100th	@ 1000th	@ 10000^{th}
6	0.592441	0.723216	0.905272	1.002266	1.094646
8.5	0.98458	1.310186	1.637108	1.849336	2.003218
11	1.575938	2.561394	3.517746	4.00959	4.37668
13	5.319651	6.591323	8.174571	8.743397	8.967365
17	6.633238	8.354286	9.999778	10.57832	10.88378
21	9.970635	11.20381	13.52337	14.36147	14.69987
25	14.946541	19.71101			

Table 5-9 Regression Results for Different Applied Deviator Stresses
($\log \varepsilon_p = \log A + b \log N$)

Deviator Stress	6	8.8	11	13	17	22
R^2	0.9939	0.9944	0.9852	0.9671	0.9770	0.9614
Intercept	-0.1276	0.1343	0.4764	0.8788	0.9707	1.1104
Slope	0.0428	0.0426	0.0403	0.0193	0.0169	0.0143
Standard Error for R^2	0.0035	0.0025	0.0052	0.0026	0.0021	0.0026
Standard Error for Intercept	0.0010	0.0010	0.0015	0.0011	0.0008	0.0008
Standard Error for Slope	0.0003	0.0003	0.0004	0.0004	0.0003	0.0002

5.9 Discussion of Test Results

The following major findings can be induced from the results. All the discussions are based on the results from the single soil tested in this research phase:

- Confining pressure is not a critical factor in influencing permanent deformation.
- Deviator stress plays a major role in the development of permanent deformation, especially on the first cycle deformation
- The log of axial permanent strain versus the log of the number of load applications demonstrates linear relationship.
- The first-cycle deformation is not easily identified and separated from the whole data set. A good way to determine earlier data points is to back-calculate from the linear equation between axial strain and log of the number of load applications.
- On average, the first load repetition produced about 51% of the accumulated permanent deformation measured over 10,000 applications. This percentage increases with the increasing applied stress. It suggests that permanent deformation will accumulate faster under high deviator stress.
- Permanent deformation under first load application has a linear relationship with applied deviator stress as shown in Equation (5-1).
- Higher load frequency would increase the potential of soils to deform under repeated load, especially under high deviator stress. One reason might be that the rest period is not long enough for soils to develop an increasing resistance to succeeding load applications. Base on the results on this soil, the rest period could be reduced to 0.9 seconds in future tests. A shorter rest period, 0.4 seconds could also be used to

accelerate the testing time and produce a conservative estimation for the accumulation of permanent deformation, especially for high deviator stress. However, a 0.9 second rest period is required in resilient modulus test(AASHTO T 292-91 I), it is recommended that the same rest period be used in permanent deformation test in order to integrate the measuring of resilient and permanent deformation.

- For the stress levels tested, stress history has a strong effect on the accumulation of permanent deformation. The work hardening of soils subjected to repeated loads would increase its deformation resistance if the soil structure remains reasonably the same.

6. Conclusion and Recommendation

From the test results and analysis in the first phase of this study, the following recommendations are made for testing in the next phase:

1. Testing will be conducted at a single confining pressure of 21 kPa (3psi). Results of phase I work indicate that confining pressure does not constitute a significant factor in the development of permanent deformation. This confining pressure reasonabley represents in-situ stress conditions that a typical pavement subgrade is subjected to.
2. Reduce the number of load applications 10,000 applications instead of 100,000 applications recommended by other researchers. This is supported by the linear relationship between permanent strain and load applications.
3. Set the load frequency to1 Hz with a rest period of 0.9 second. Using this length of rest period, the resilient modulus can be acquired simultaneously and the data will be consistent with AASHTO T292-91I.
4. A virgin soil specimen should be used for each deviator stress. The results of phase I indicate that sequential testing of soil under increasing loads increases the resistance of soils to permanent deformation. Specimens subjected to previous load applications would demonstrate lower deformation than fresh specimens without stress history.
5. When constructing the relationship between ε_p and log N, a reasonable approach is to only use data beyond the first 100 cycles of. The results of phase I indicate that data collected for the first hundred cycles do not constitute a reasonable part of the whole data set because of gradual application of load over the first several cycles of testing.

6. Test specimens with different deviator stresses starting 28 kPa (4 psi). Dynamic strength can be obtained when quick failure occurs. For the soil test in Phase I study, the dynamic strength is about 60% of static strength.
7. For practical use, a reasonable method for predicting permanent deformation would consist of two steps. (1) Test one specimen up to 10,000 cycles to get the exponent of the power model for Equation 3.1. (2) Test a series of specimens under different deviator stresses for only 1000 cycles each to get the prediction model expressed in Equation (5-1)
8. Test specimens should be subjected to a conditioning period after they are subjected to confining pressure in the triaxial cell. This conditioning phase should consist of applying a small static deviator stress at the same time the confining pressure is applied. The deformation versus time decay is then monitored to determine when the specimen has achieved full consolidation under the applied confining pressure. Once the decay of the deformation is complete the actual dynamic testing of specimen can proceed.
9. Representative soils in Arkansas should be tested to investigate changes in deformation behavior from soil to soil. Three levels of moisture content: 105%, 110%, and 120% of OMC, and three density levels: 90%, 95%, and 100% of MDD should be used to explore the effects of moisture content and density on the accumulation of permanent deformation. Detailed test plan for Phase II research is attached in Appendix D.

Acknowledgement

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SI Unit-Conversion Factor

1 kPa = 6.895 psi

1 mm =25.4 inch

1 g = 0.002203 pound

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Appendix A

Plots of Test Results

Figure A-1 Development of Total and Permanent Deformation (test 1 in arithmetic scale)

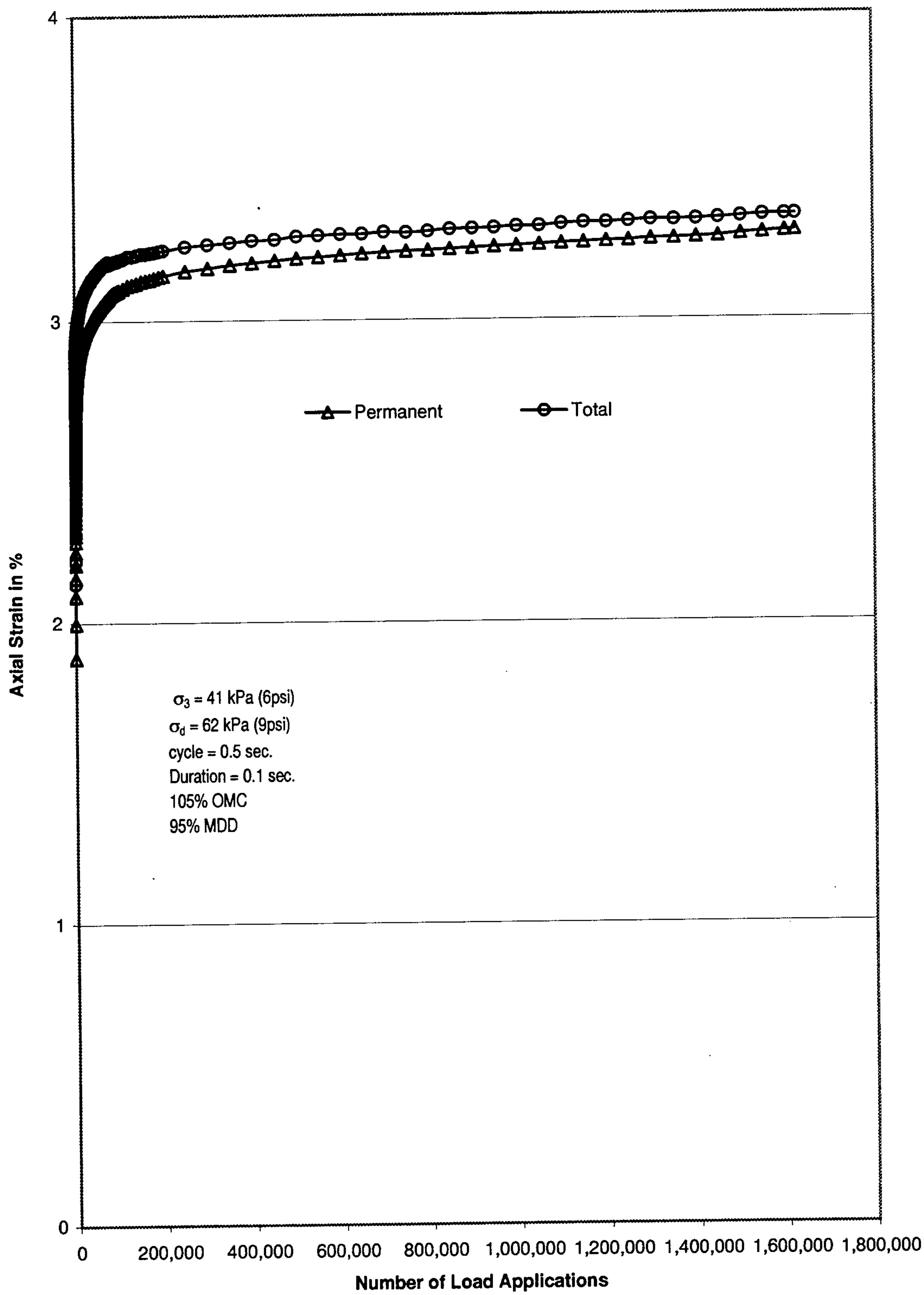


Figure A-1a Development of Total and Permanent Deformation (test 1 in log-log scale)

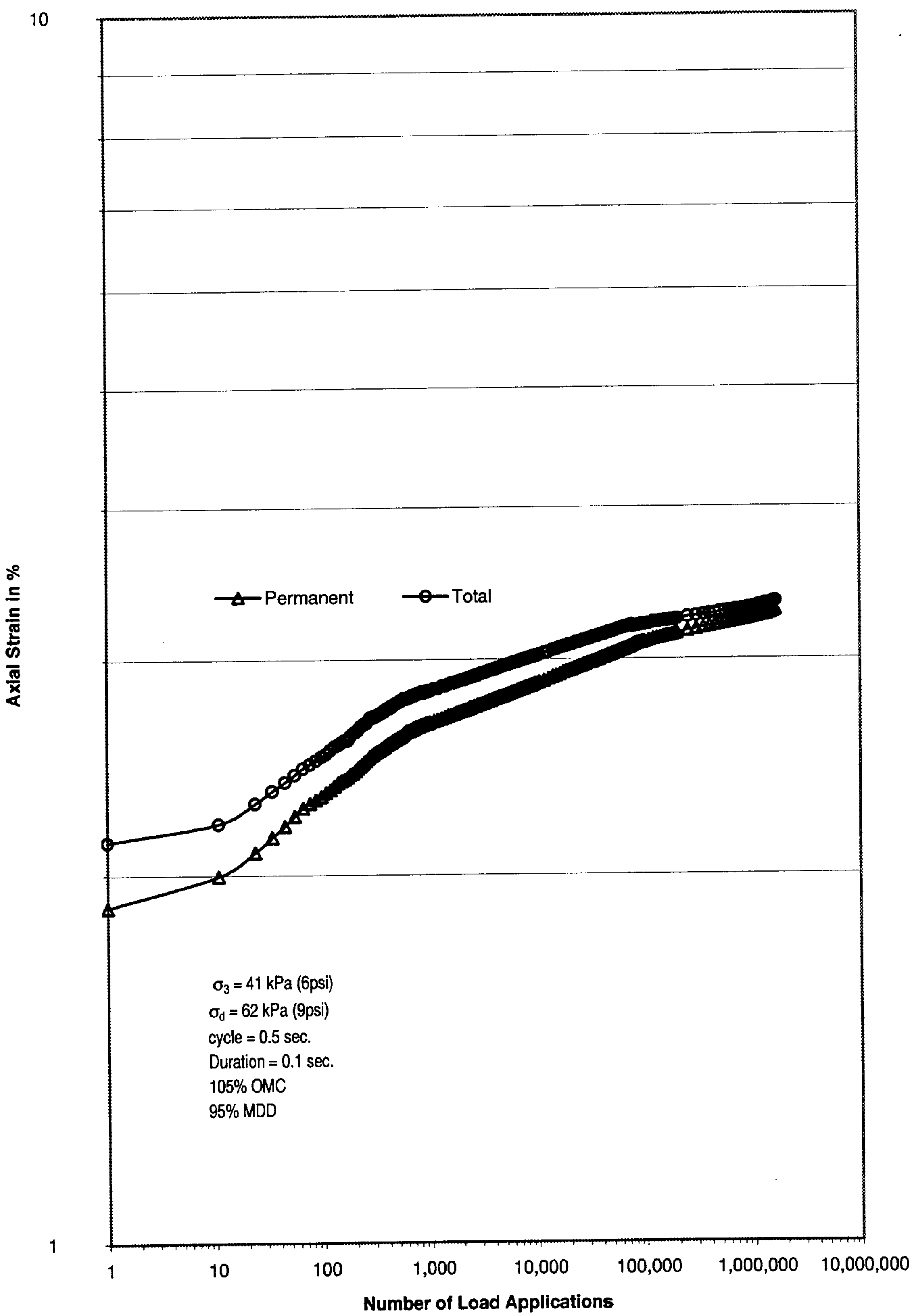


Figure A-1b Development of Total and Permanent Deformation (test 1 in semi-log scale)

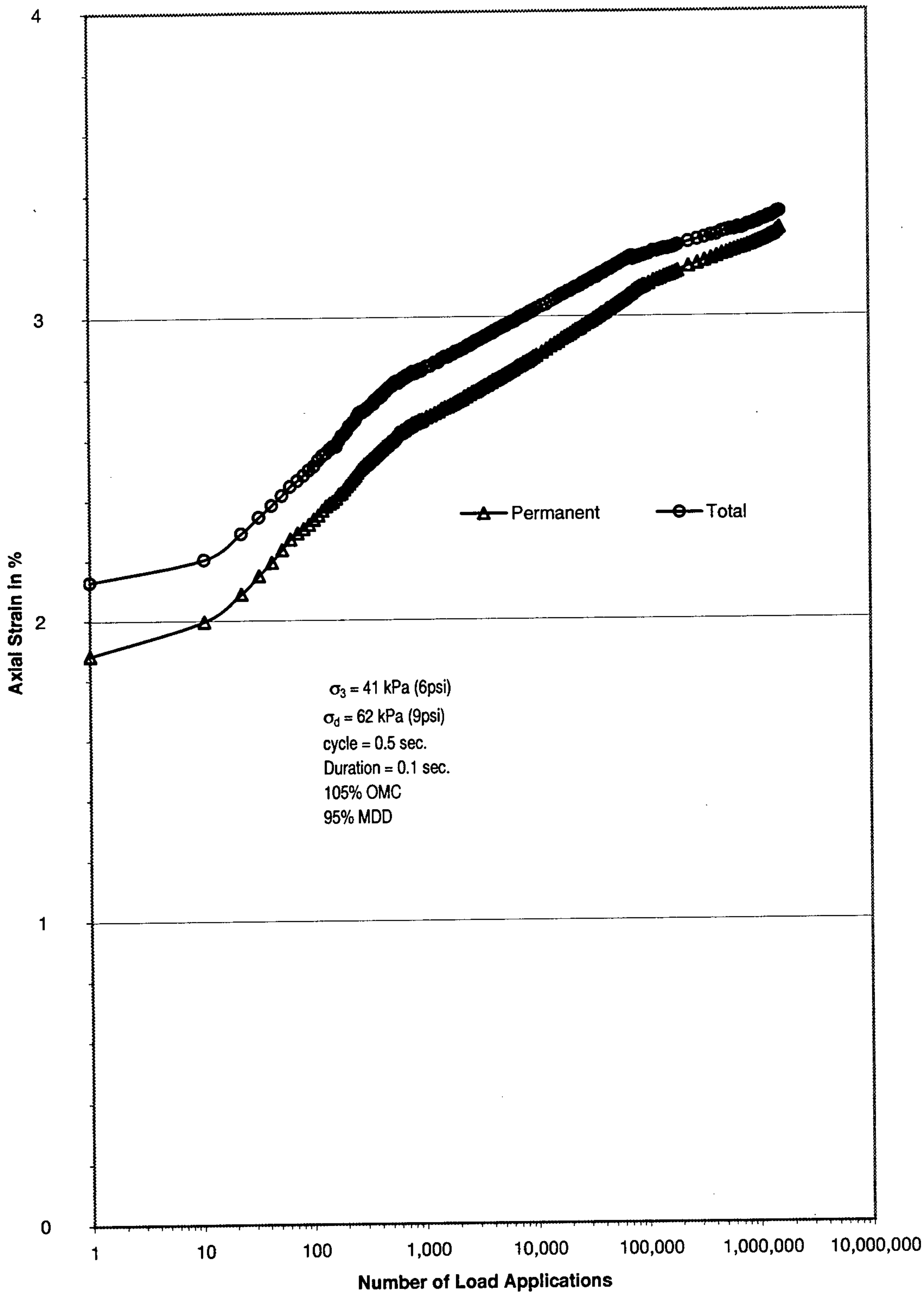


Figure A-2 Development of Axial Total and Permanent Deformation (test2 in arithmetic scale)

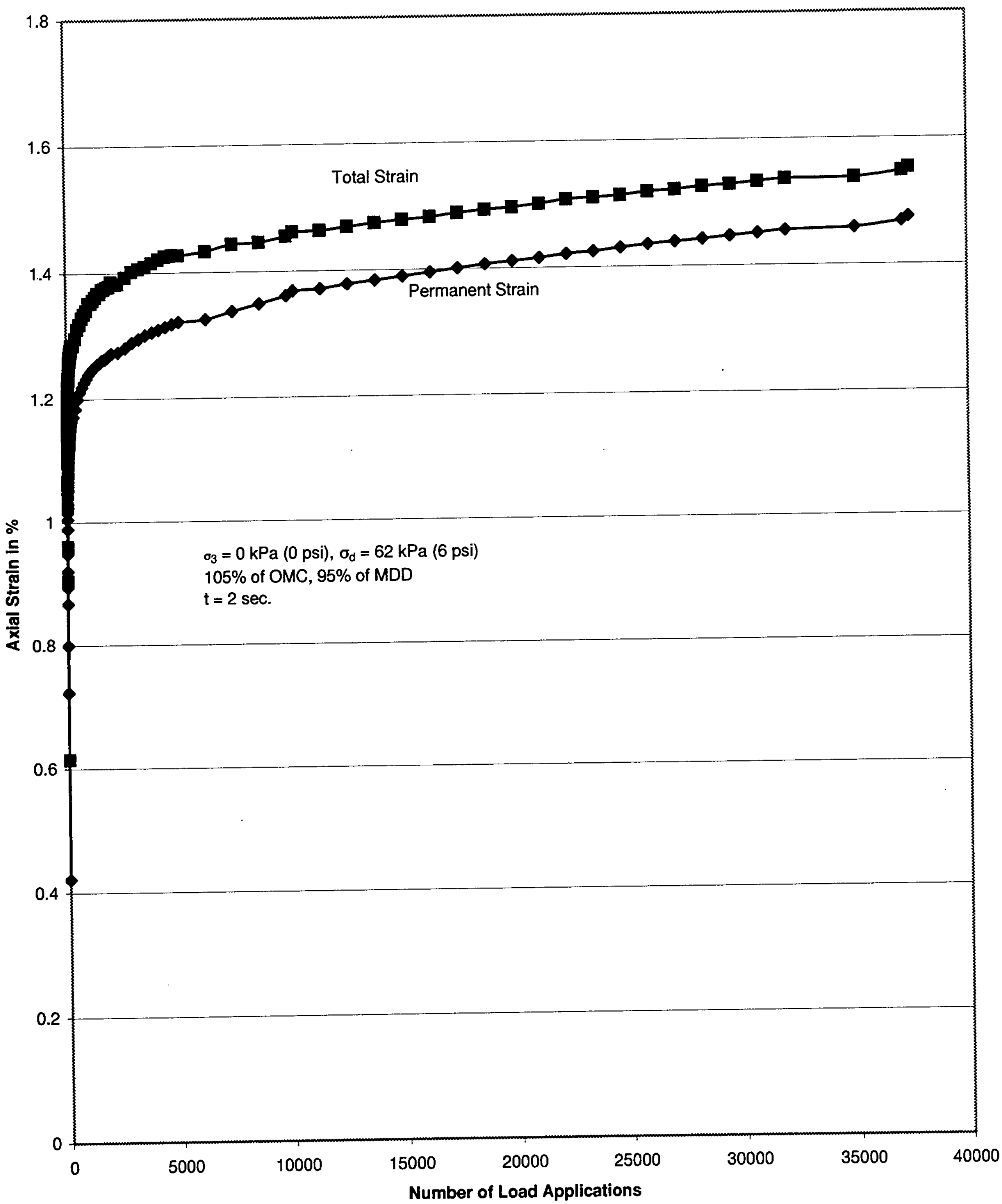


Figure A-2a Development of Axial Total and Permanent Deformation (test 2 in semi-log scale)

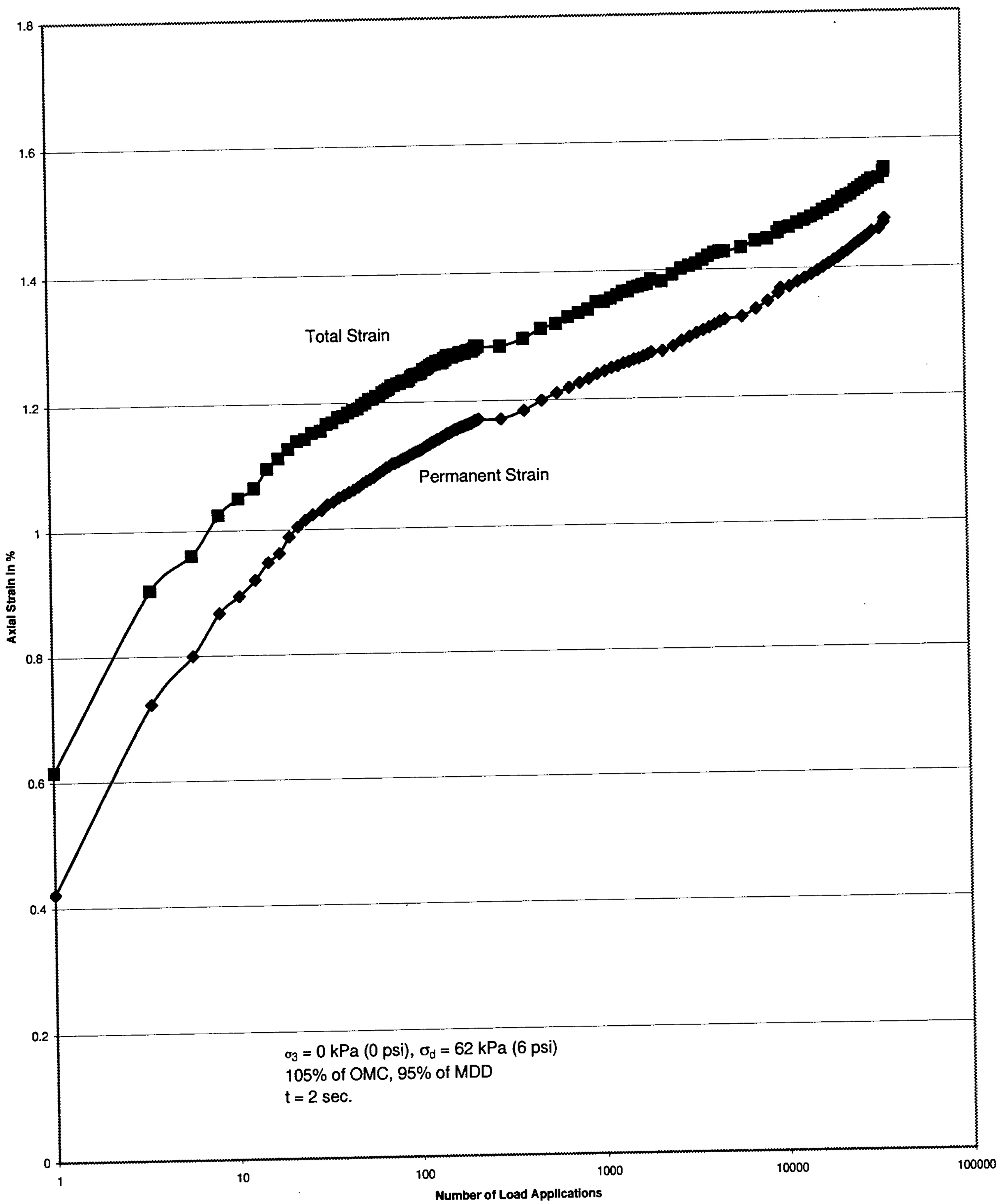


Figure A-3 Development of Axial Total and Permanent Deformation (test3 in arithmetic scale)

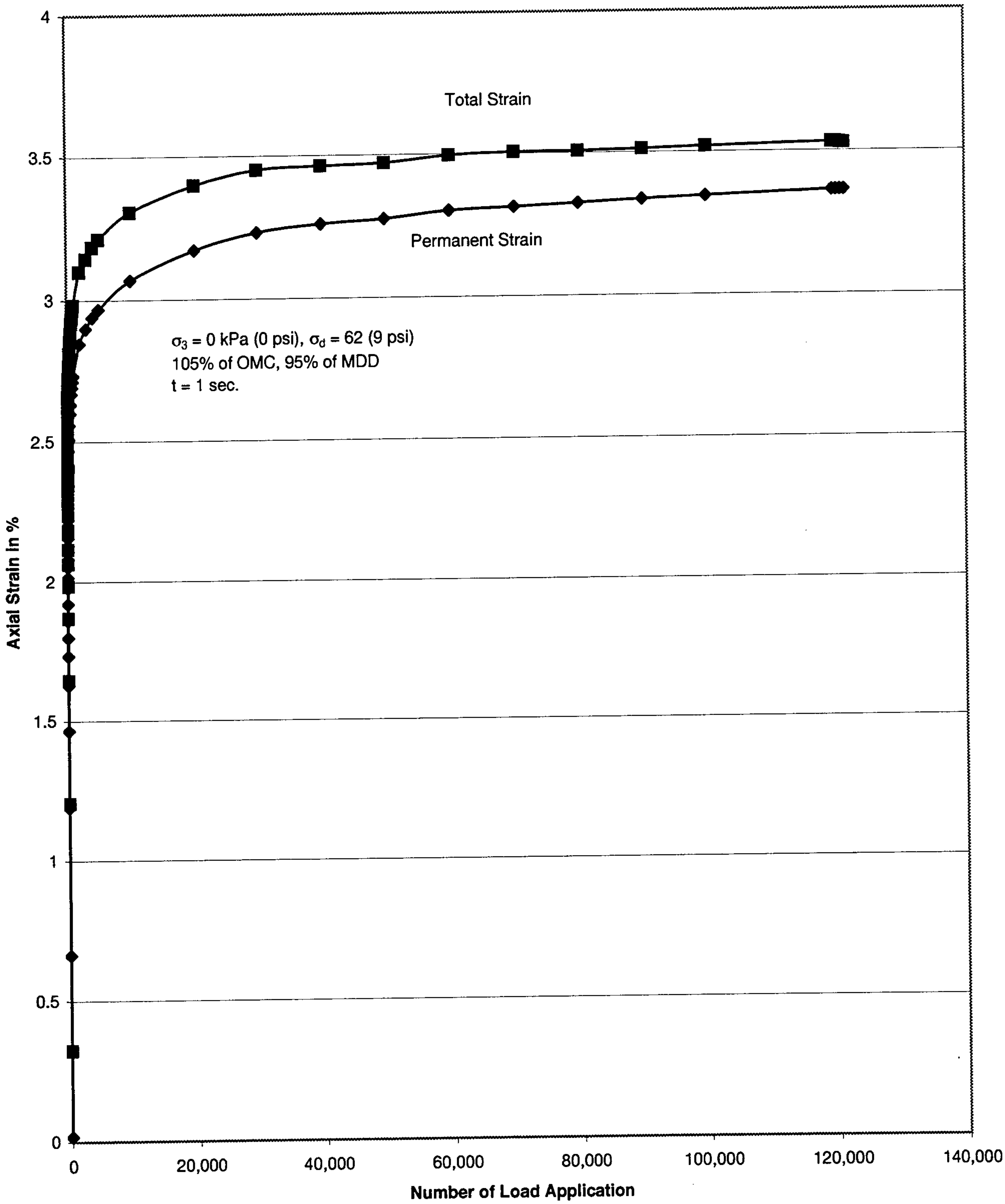


Figure A-3a Development of Axial Total and Permanent Deformation (test 3 in semi-log scale)

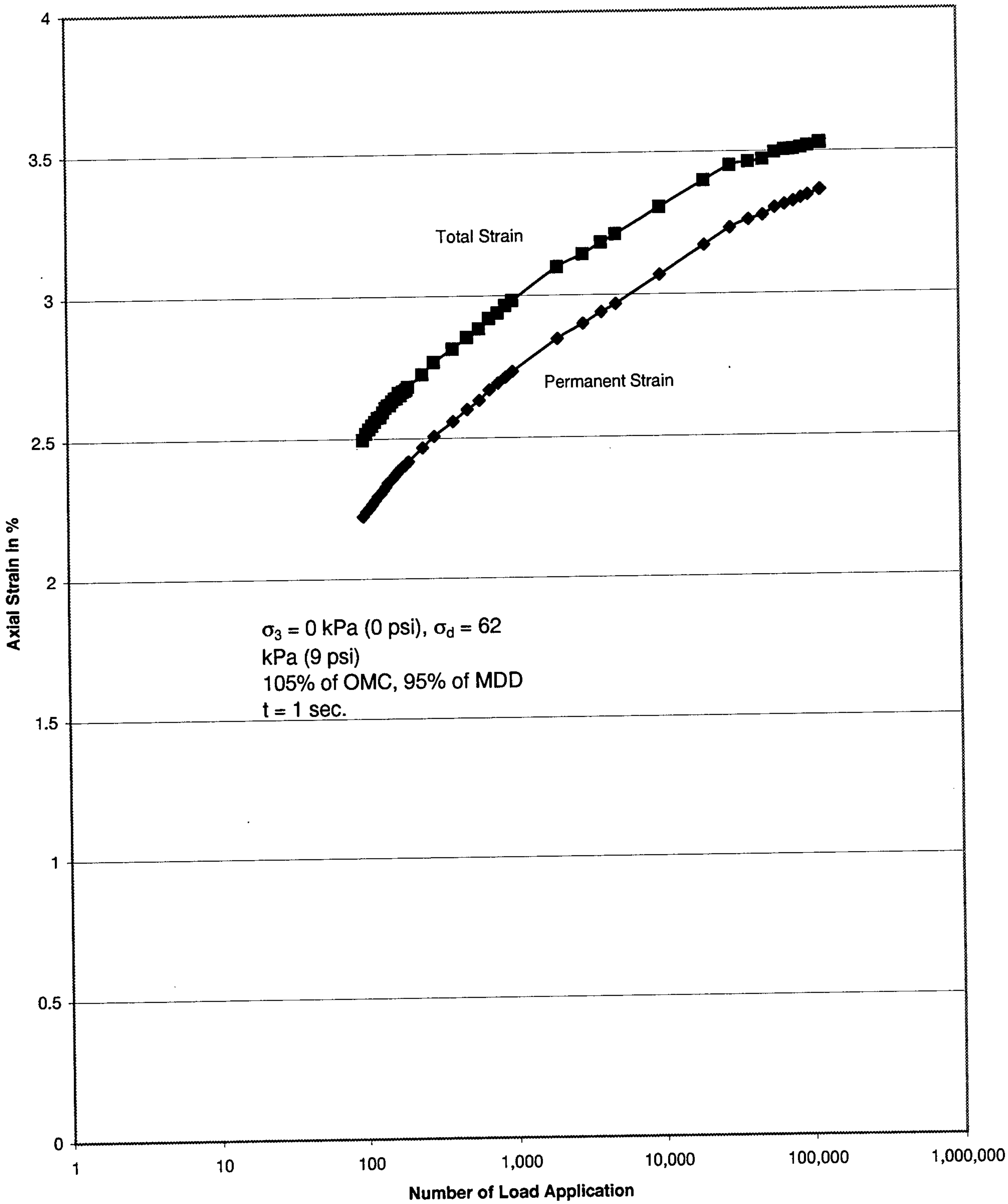


Figure A-4 Development of Axial Total and Permanent Deformation (test 4 in arithmetic scale)

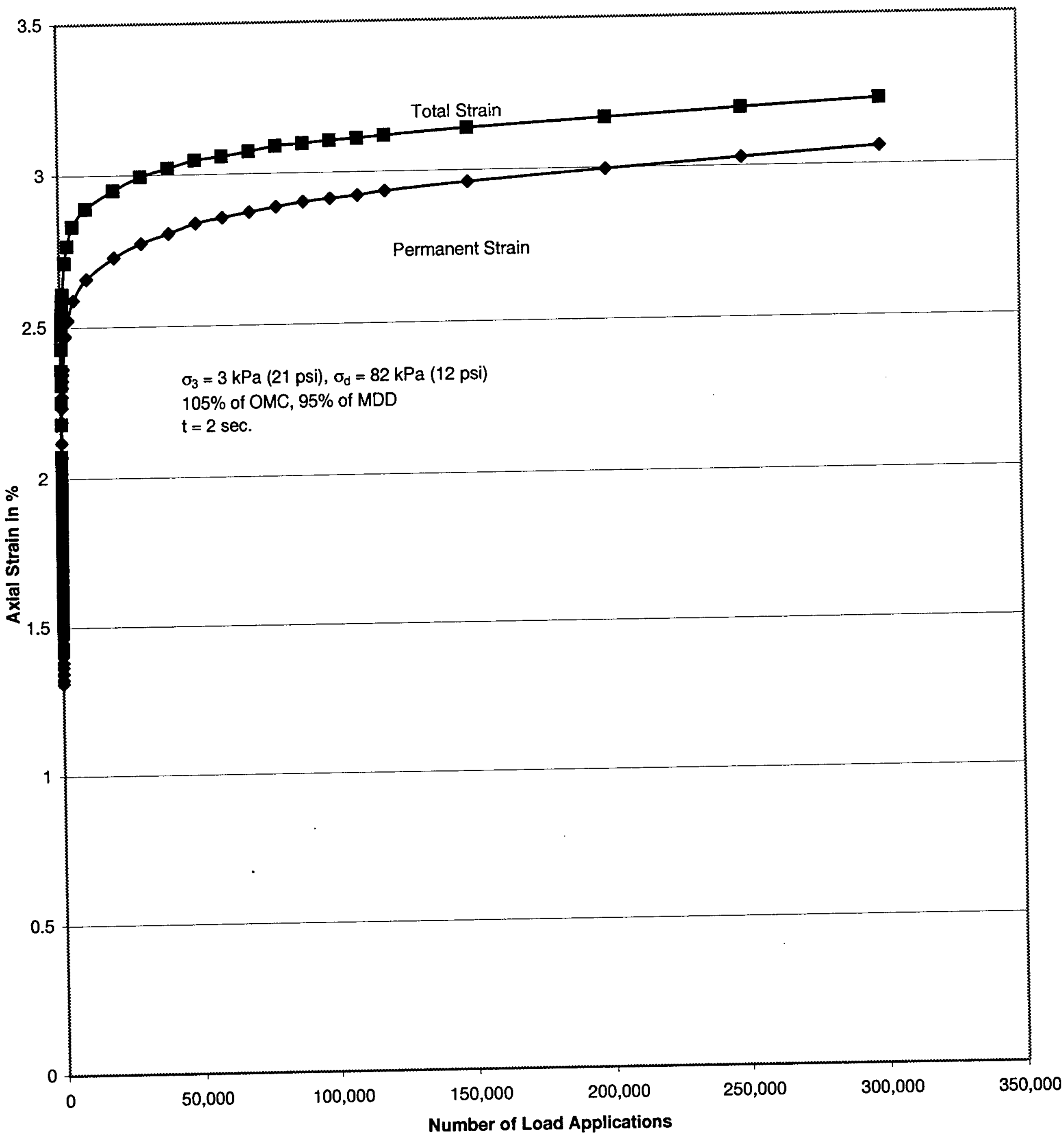


Figure A-4a Development of Axial Total and Permanent Deformation (test 4 in semi-log scale)

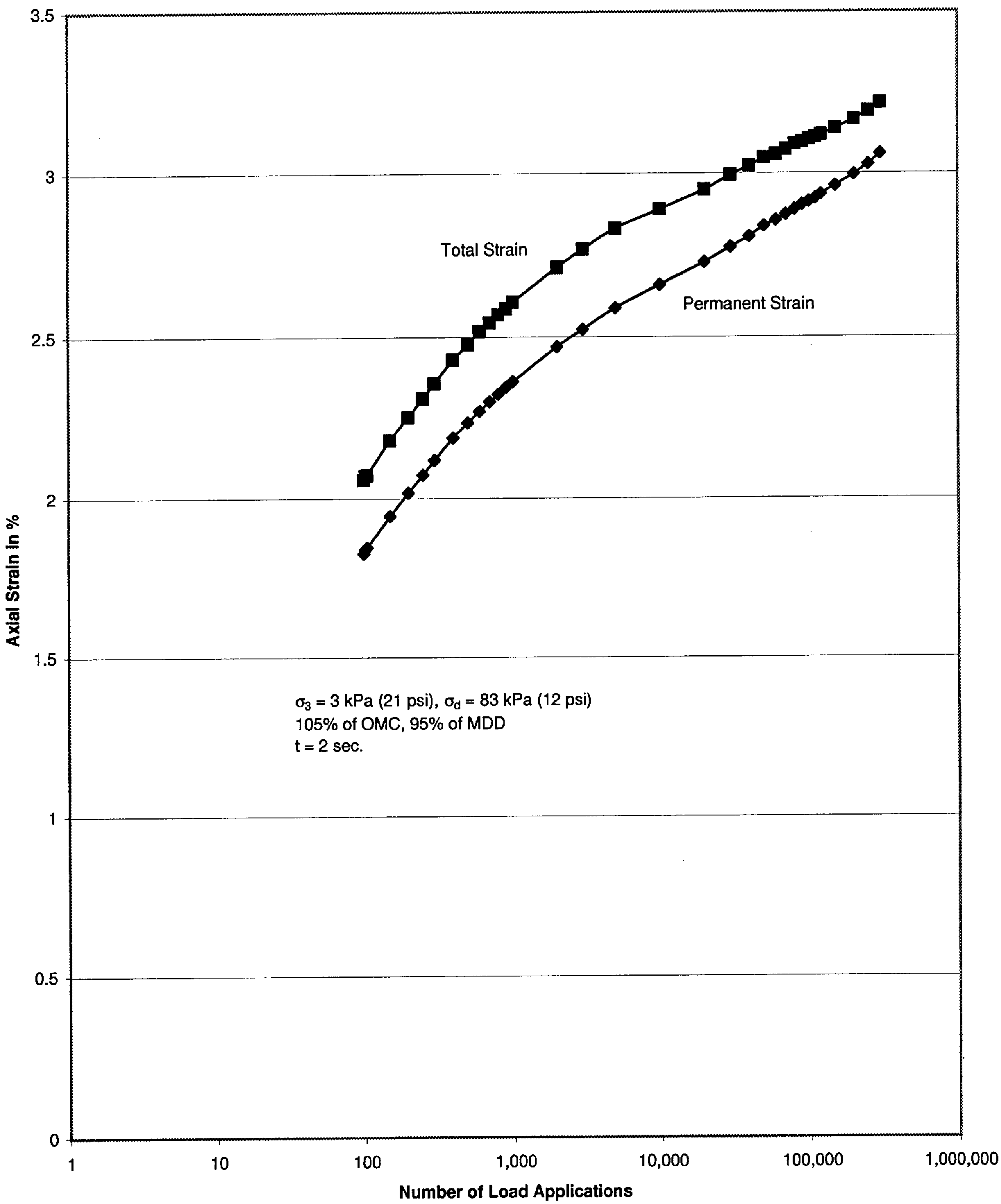


Figure A-5 Comparison of the Axial Deformation with Different Load Periods
(series 1)

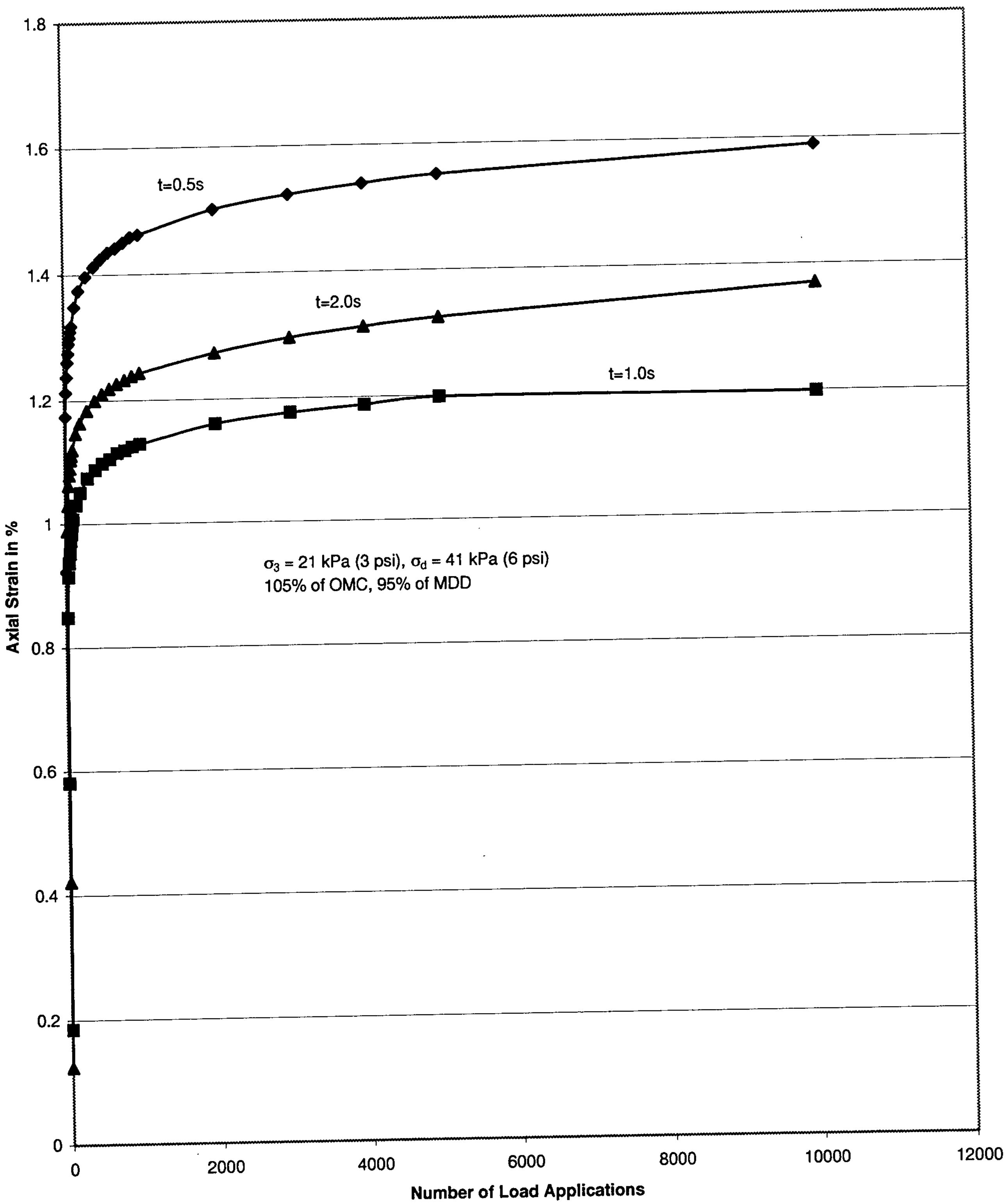


Figure A-6 Comparison of the Axial Deformation with Different Load Periods (series 2)

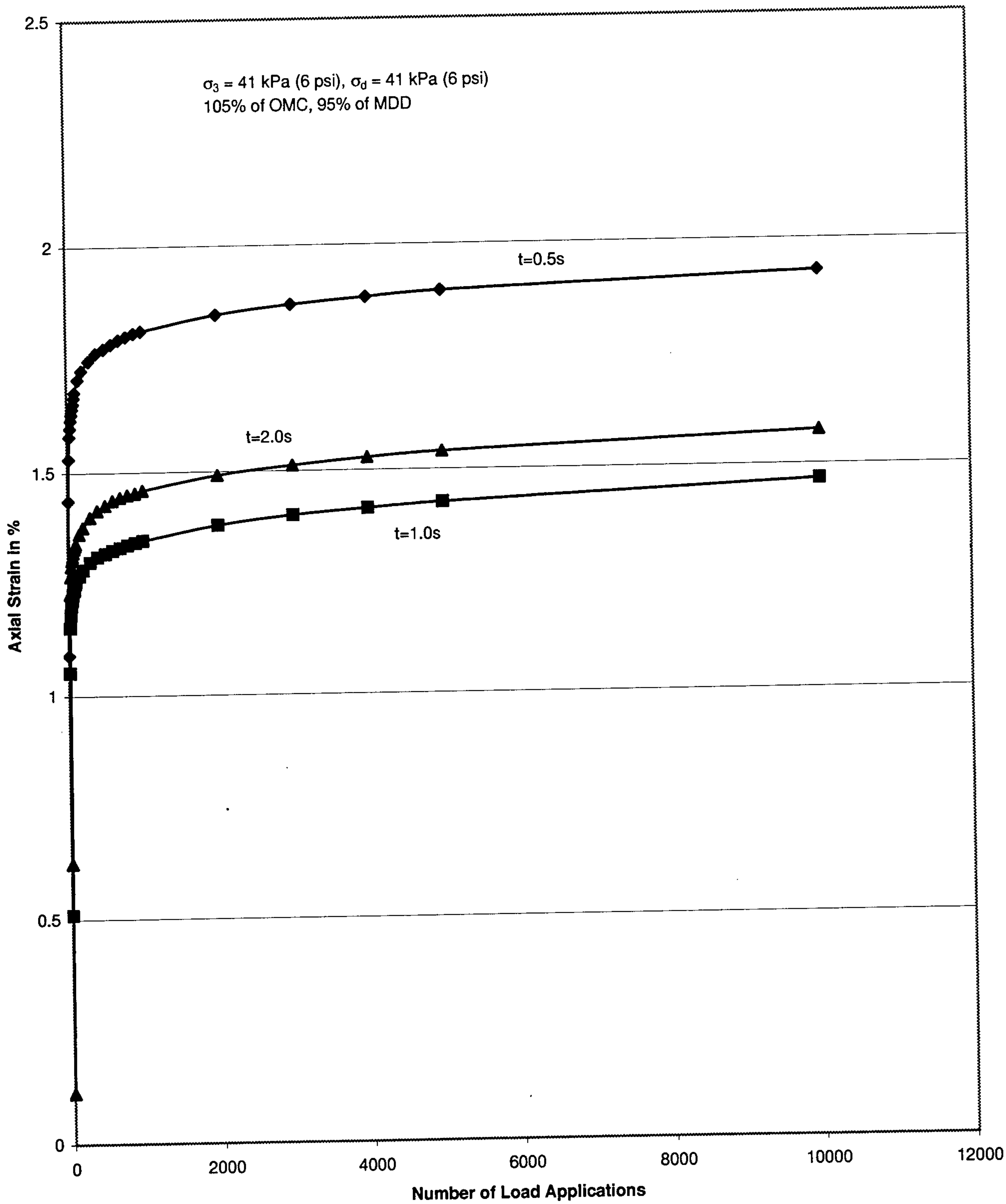


Figure A-7 Comparison of the Axial Deformation with Different Load Periods (series 3)

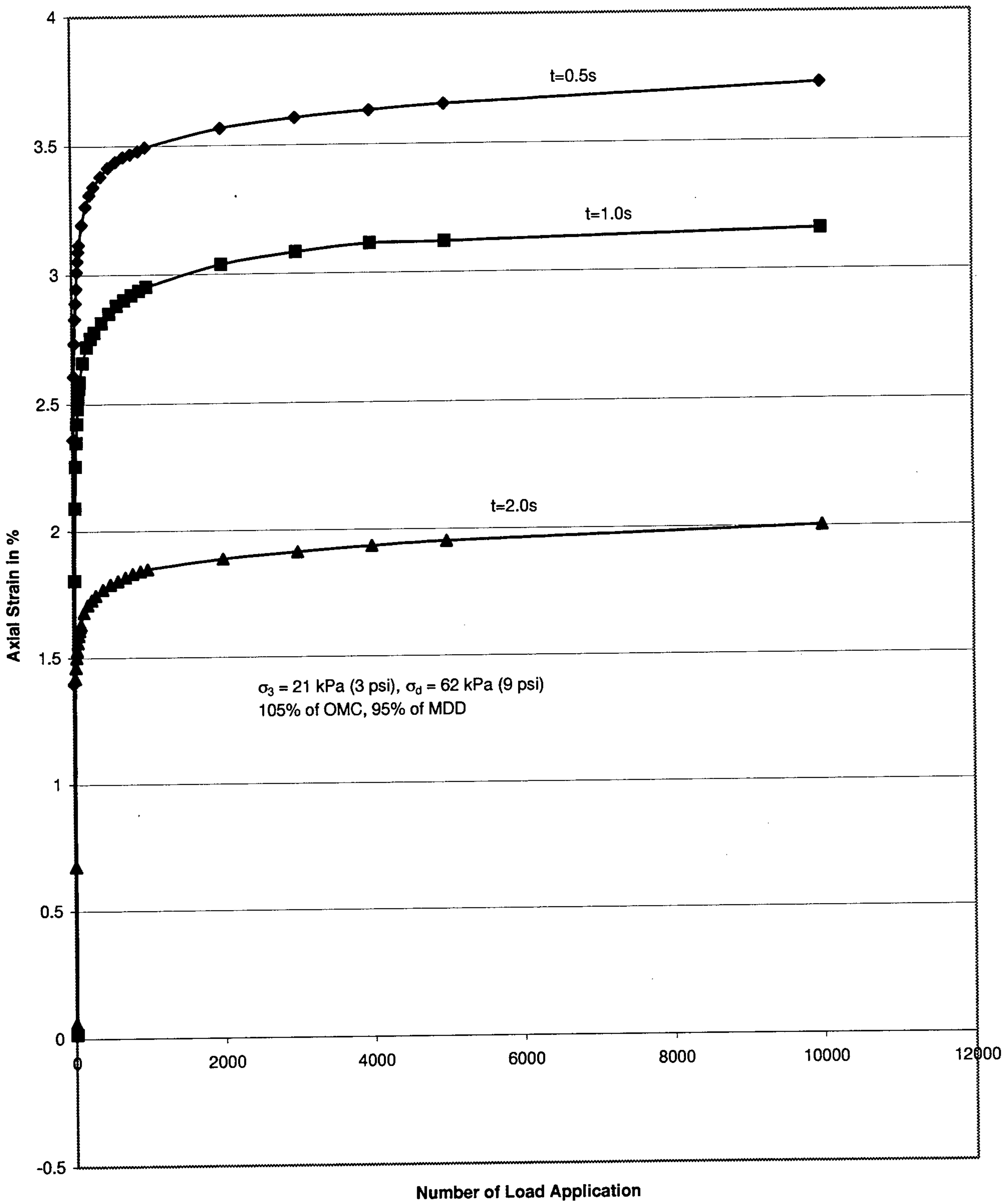


Figure A-8 Comparison of the Axial Deformation with Different Load Periods (series 4)

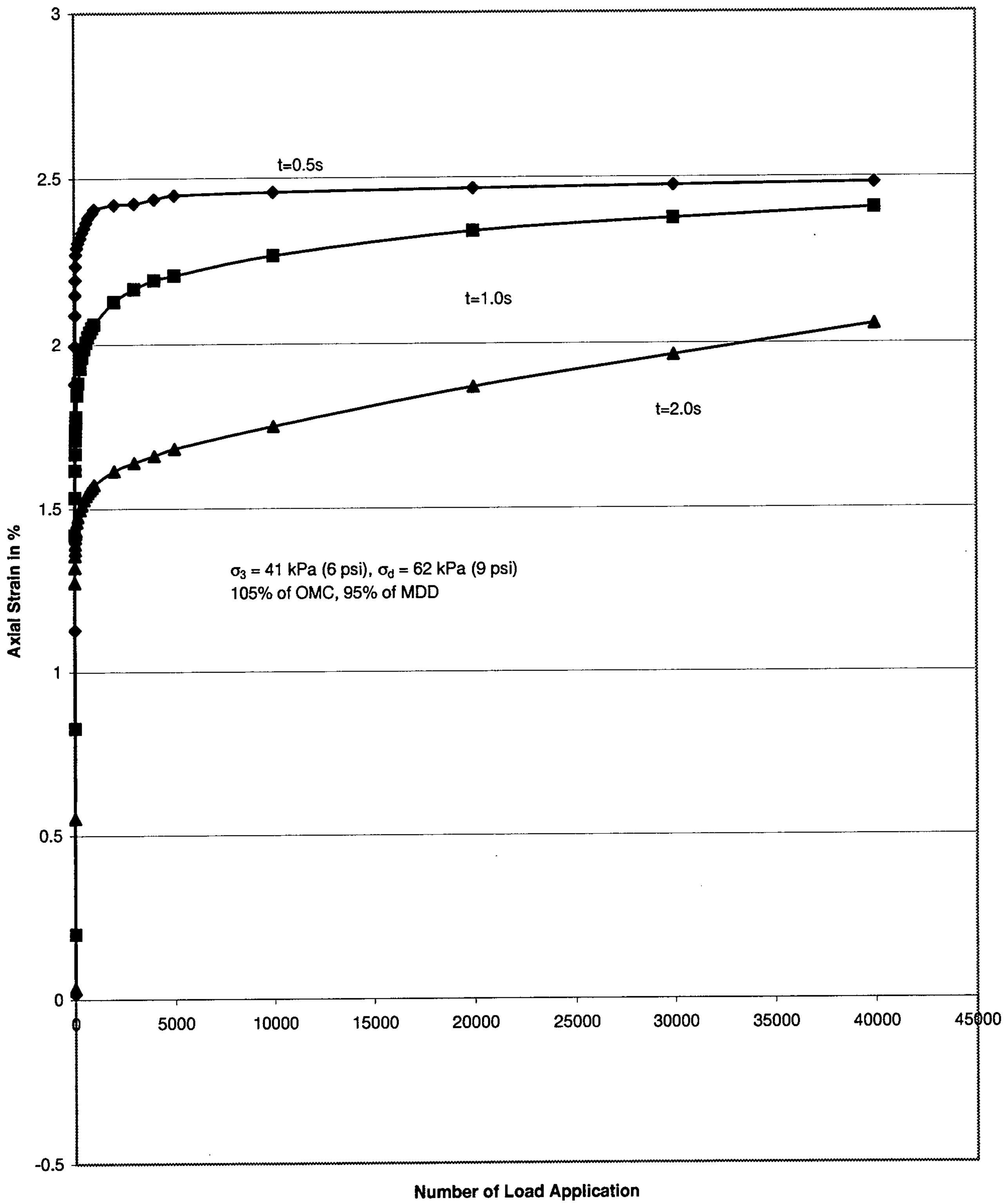


Figure A-9 Effects of Confining Pressure on Deformation (series 1)

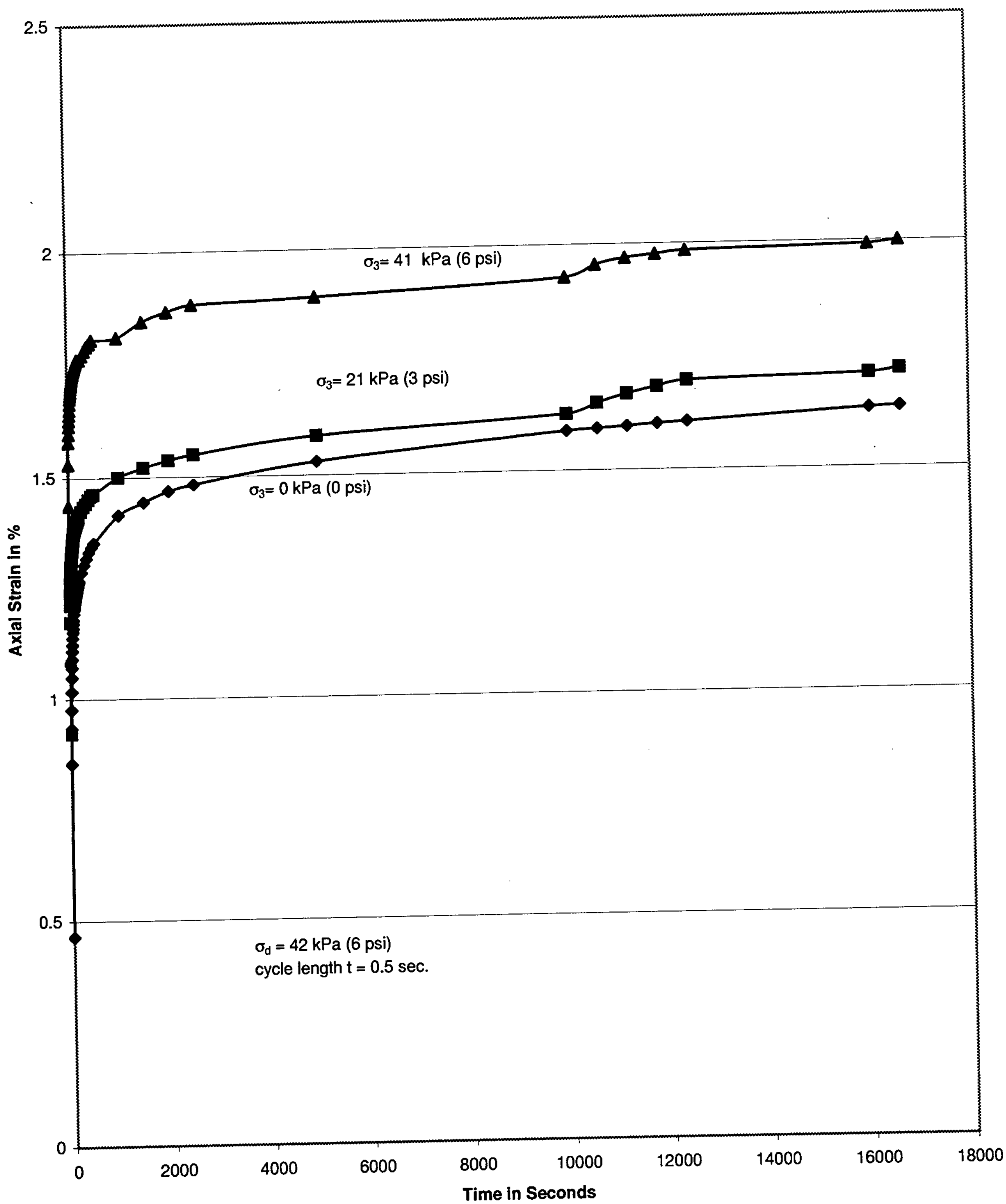


Figure A-10 Effects of Confining Pressure on Deformation (series 2)

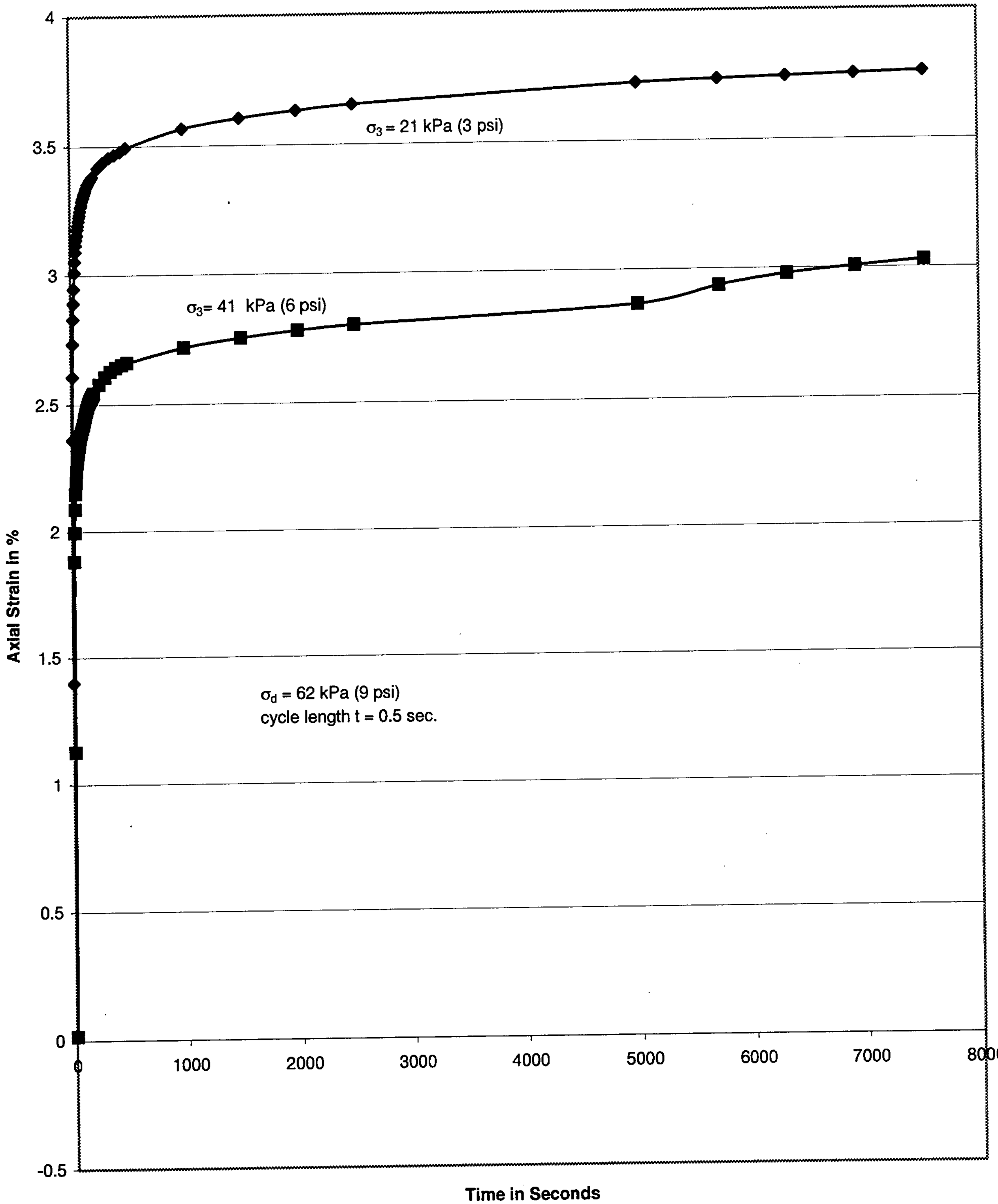


Figure A-11 Effects of Confining Pressure on Deformation (series 3)

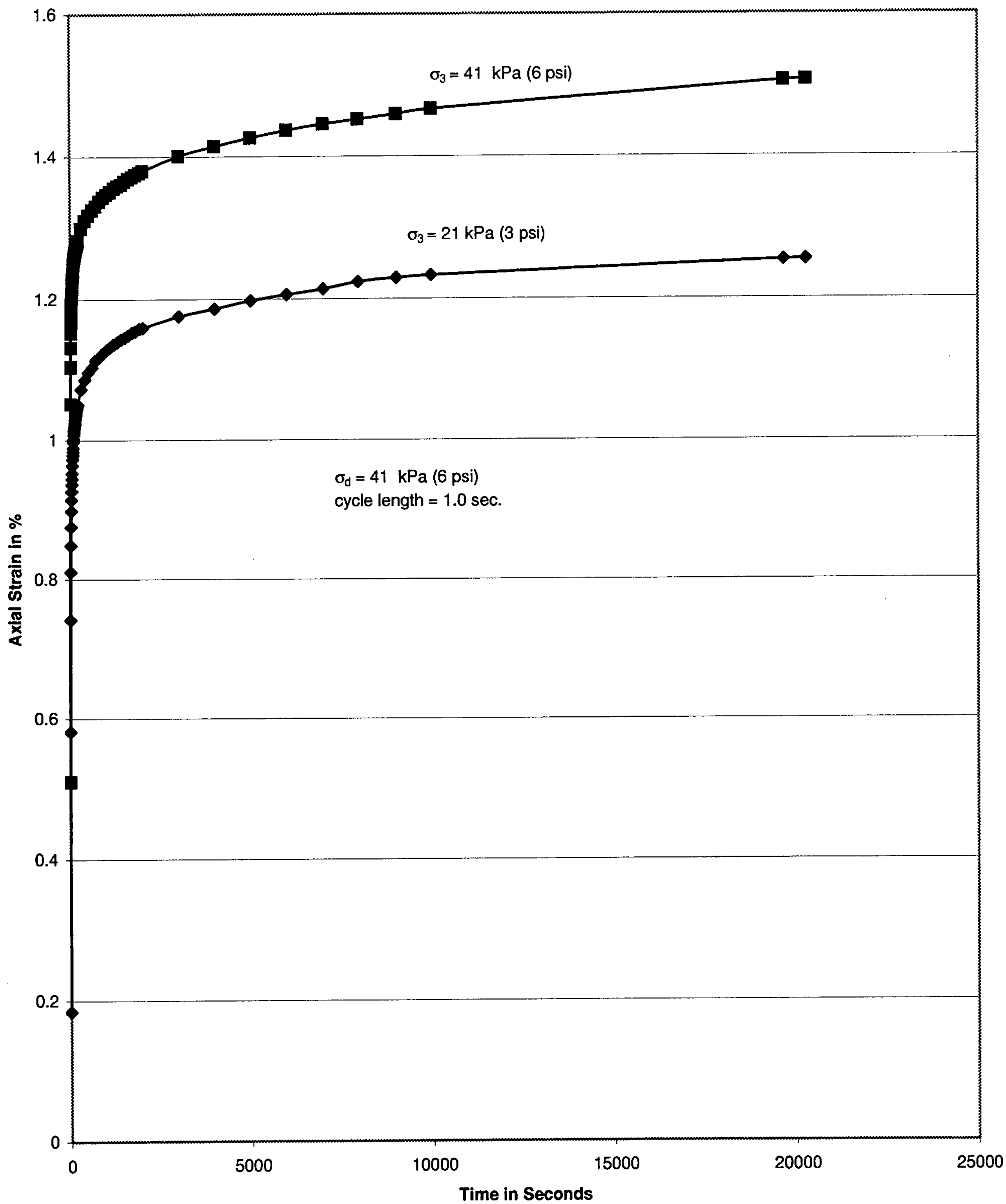


Figure A-12 Effects of Confining Pressure on Deformation (series 4)

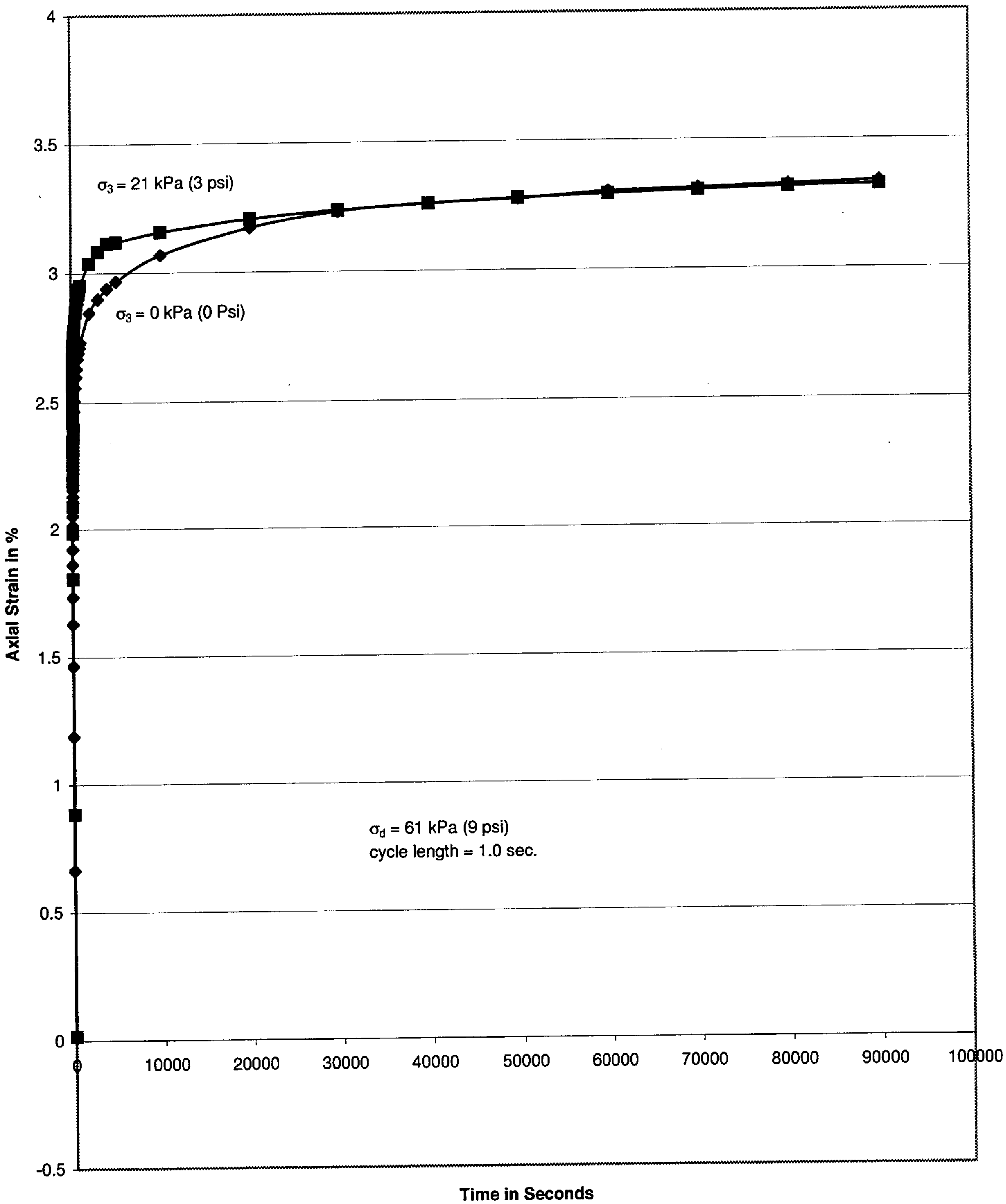


Figure A-13 Effects of Confining Pressure on Deformation (series 5)

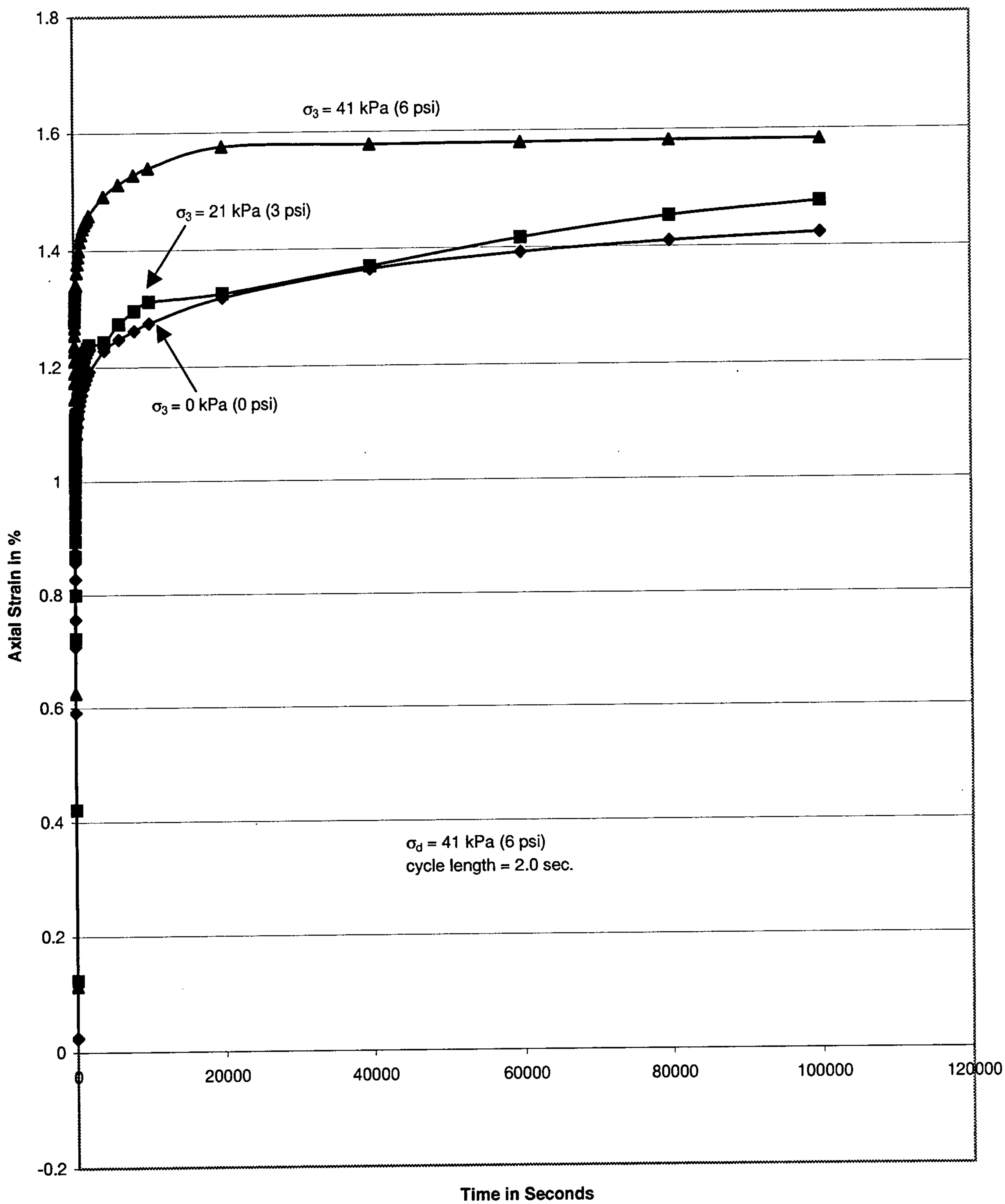


Figure A-14 Effects of Confining Pressure on Deformation (series 6)

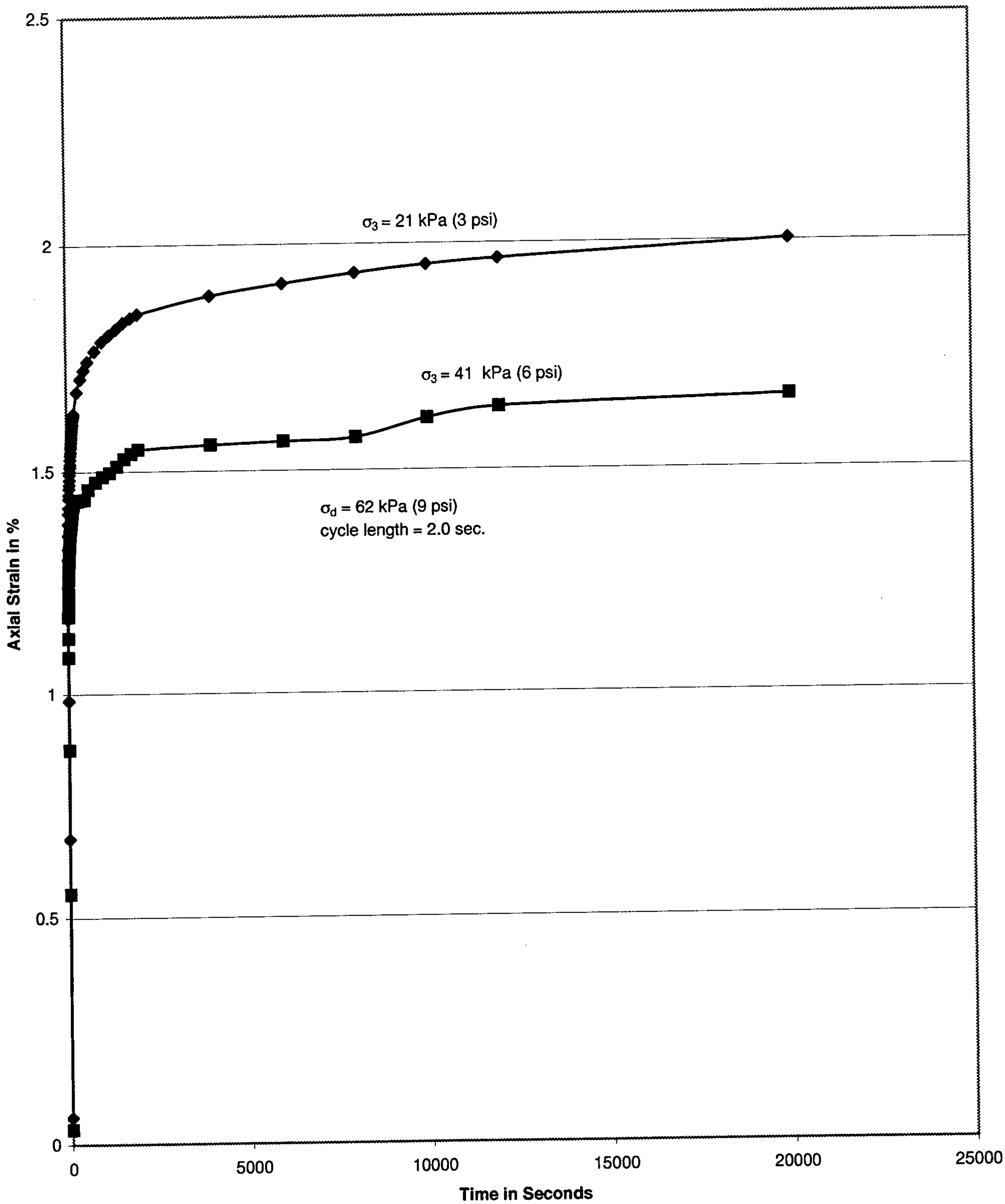


Figure A-15 Effects of Confining Pressure on Deformation (series 7)

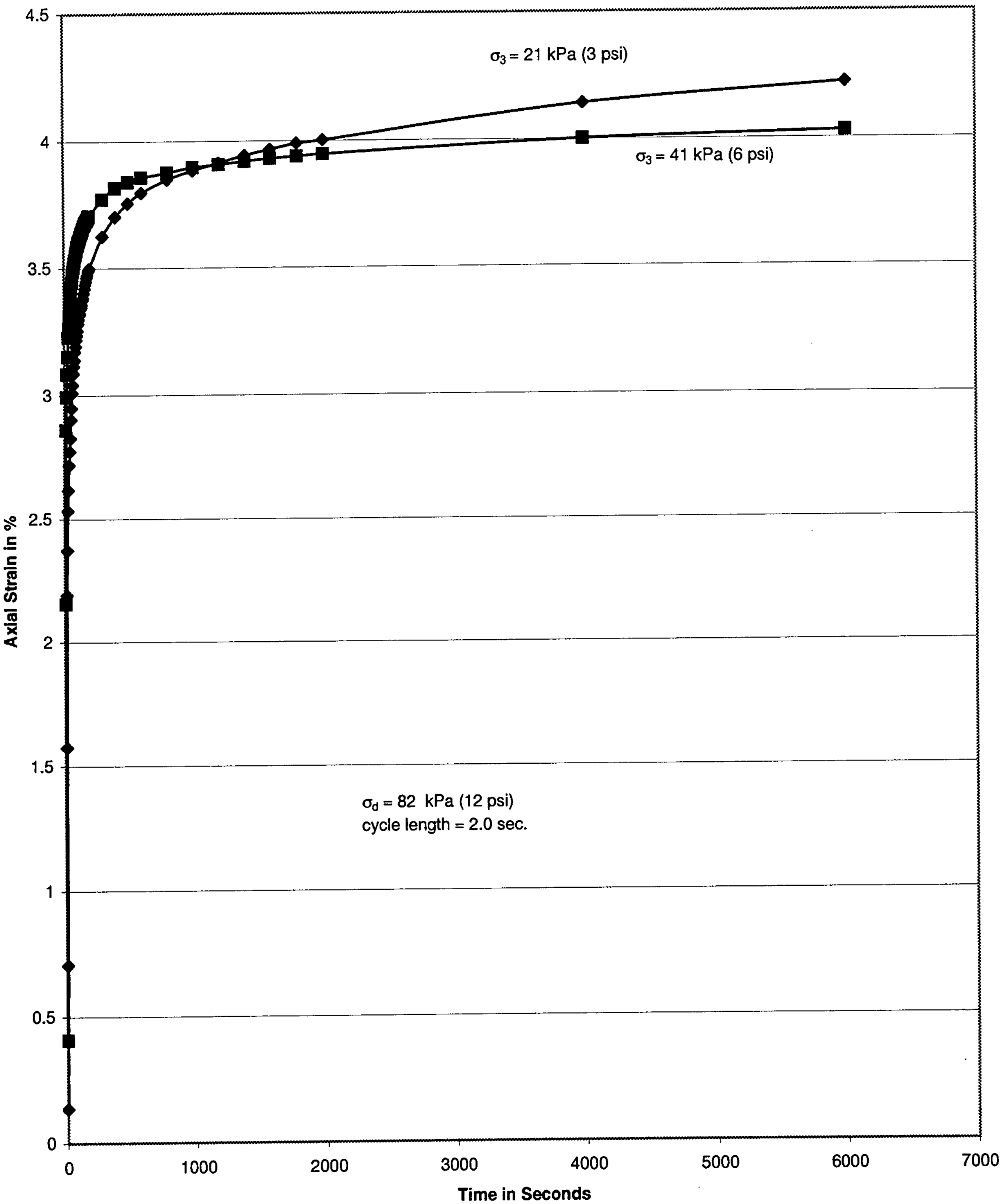


Figure A-16 Effects of Confining Pressure on Deformation
(series 8: with loading history of sd = 41.37 kPa or 6 psi)

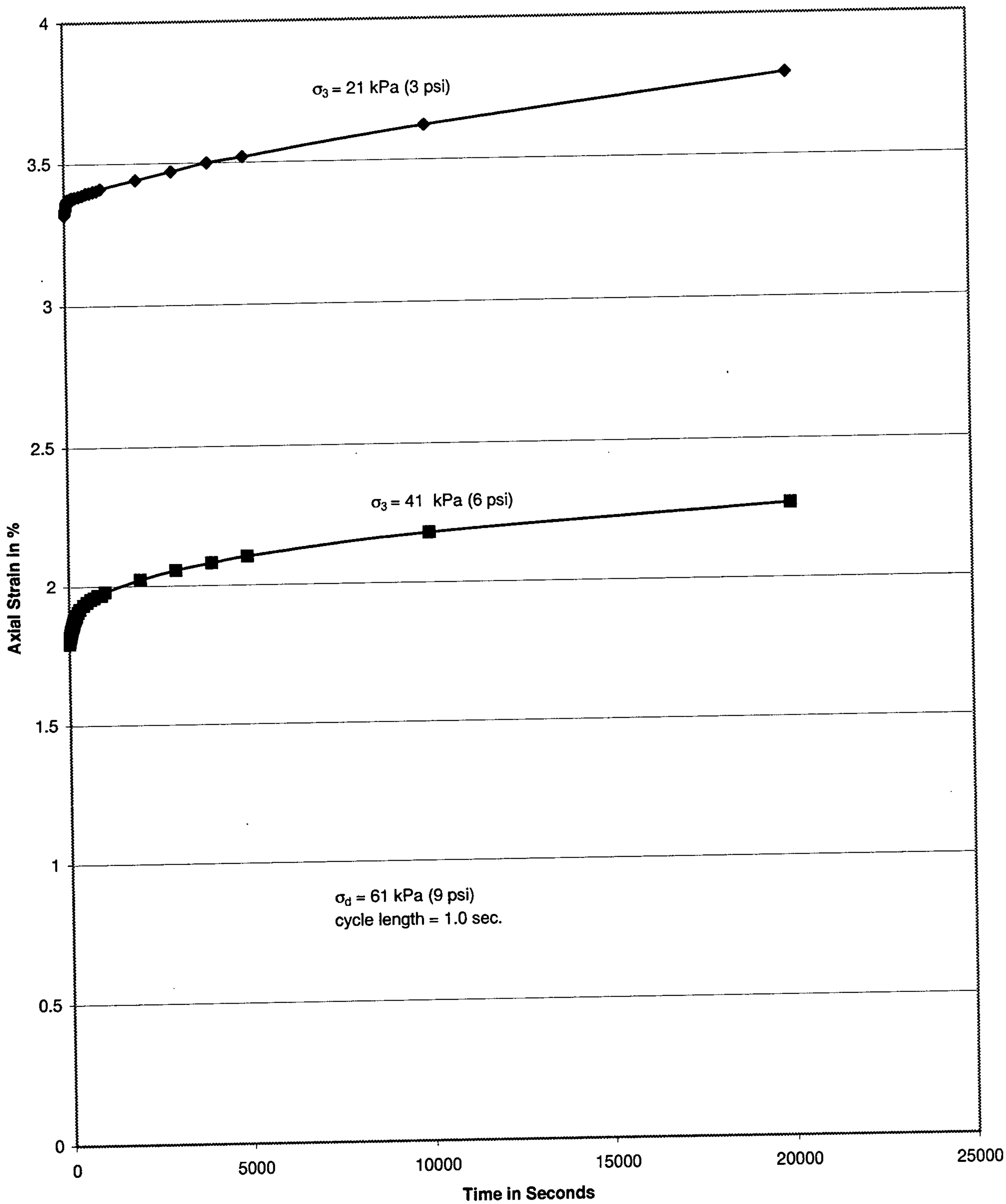


Figure A-17 Effects of Deviator Stress on Deformation
(series 1)

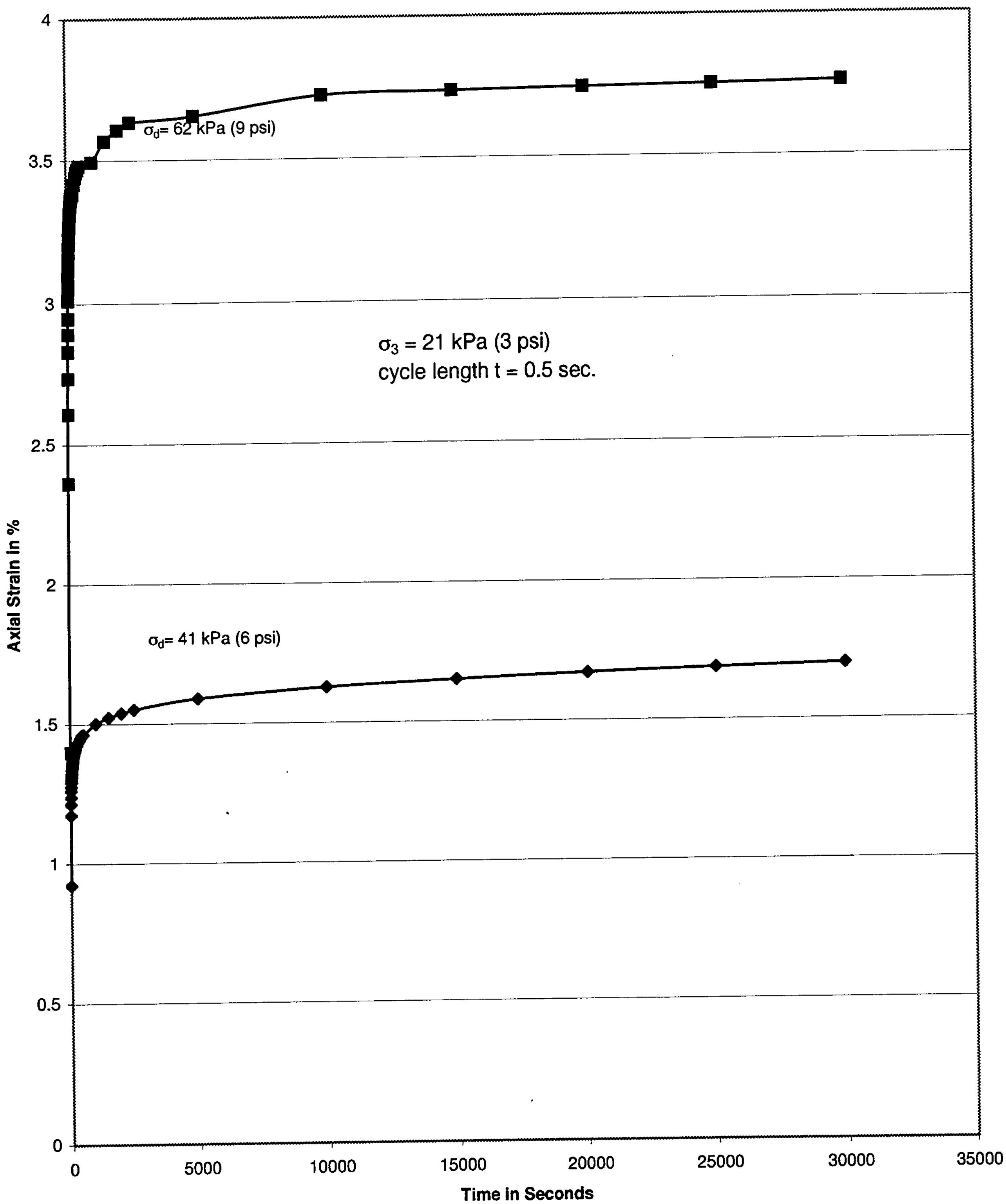


Figure A-18 Effects of Deviator Stress on Deformation (series 2)

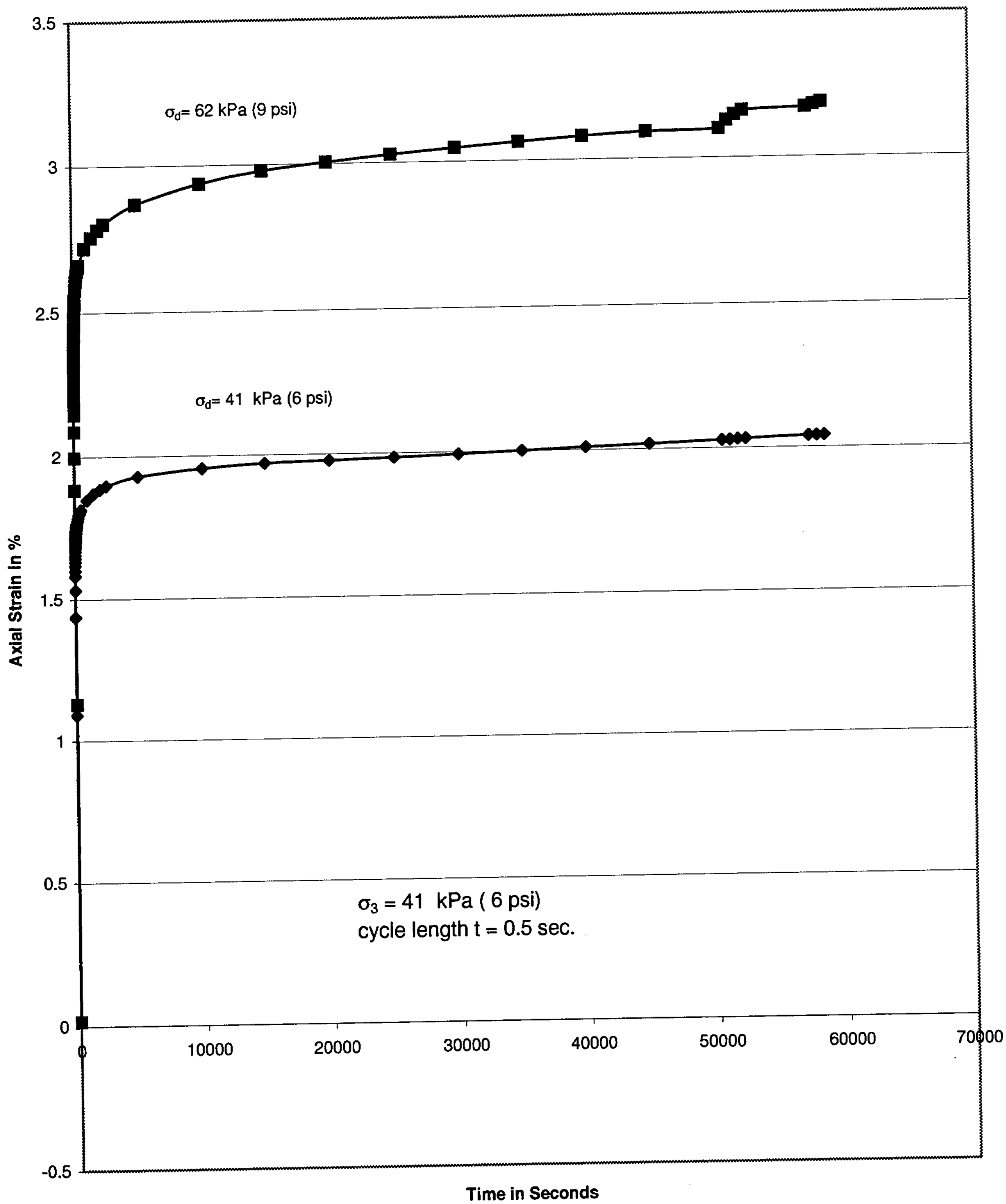


Figure A-19 Effects of Deviator Stress on Deformation
(series 3)
Series Data of Permanent Deformation(arithmetic)
105% OMC, 95% MDD, σ_3 = 21 kPa
Cycle = 2 sec. Load Duration = 0.1 sec.

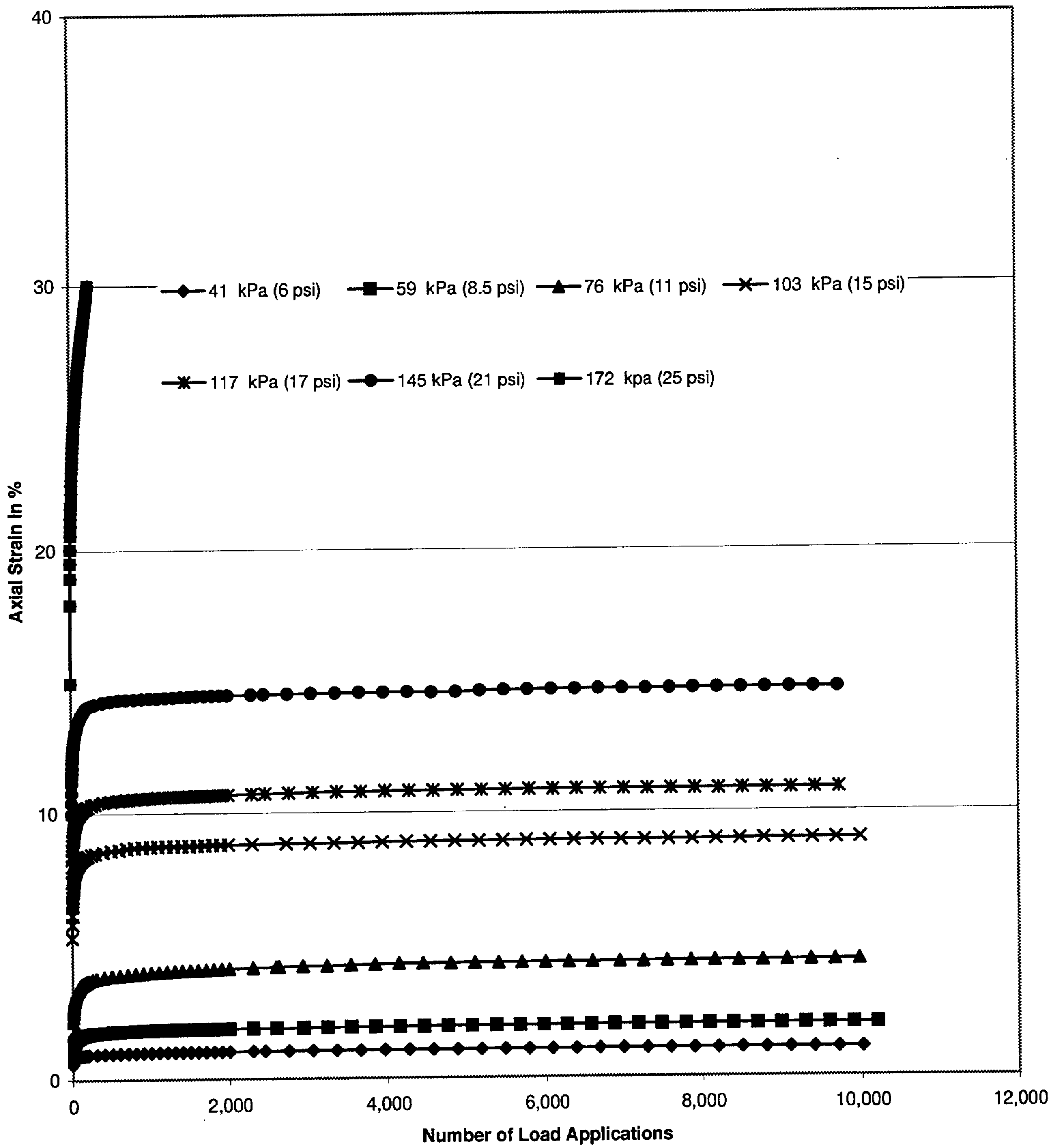


Figure A-19a Effects of Deviator Stress on Deformation (series 3)
Series Data of Permanent Deformation(semi-log)
105% OMC, 95% MDD, s3 = 21 kPa
Cycle = 2 sec. Load Duration = 0.1 sec.

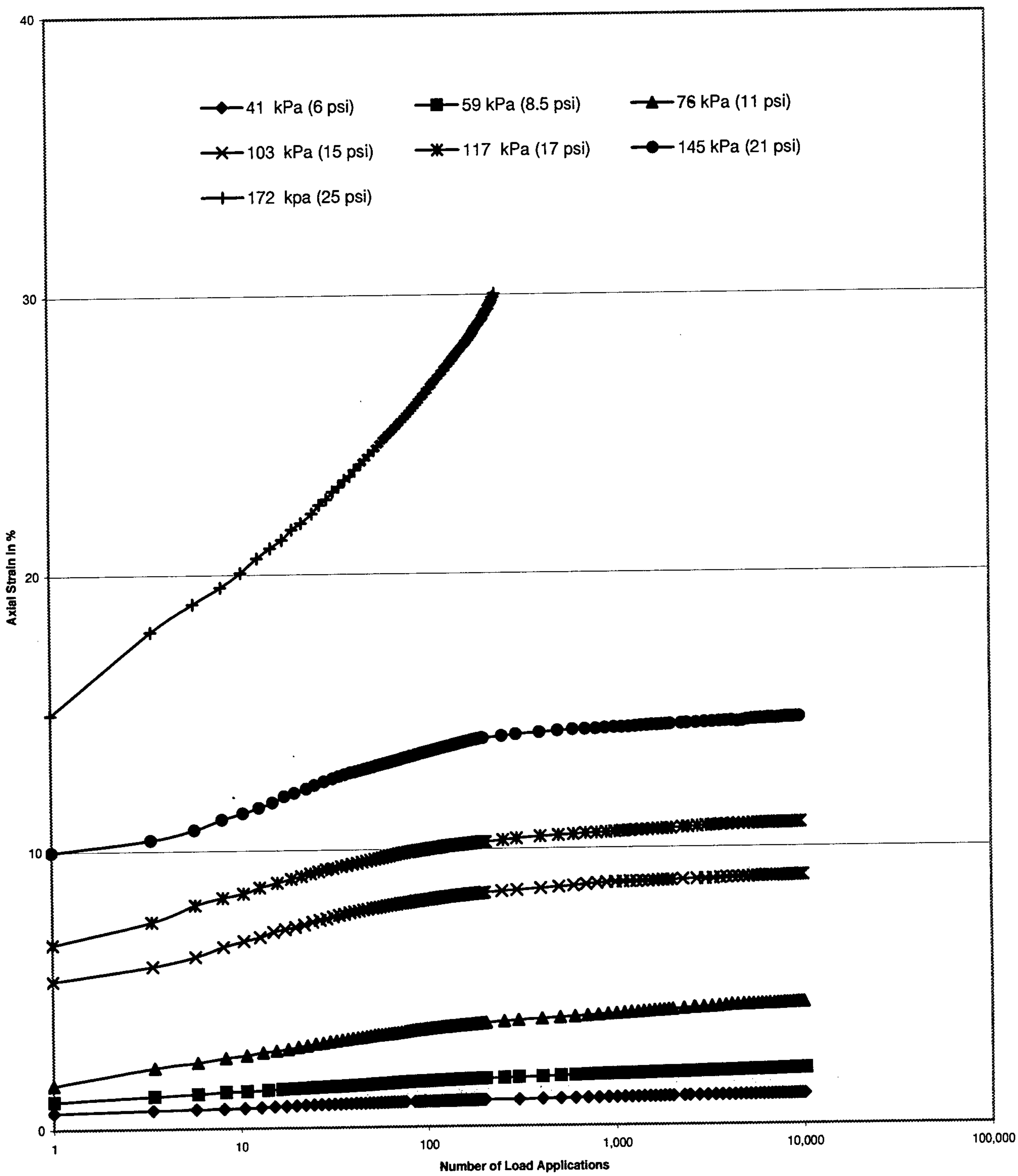


Figure A-19b Effects of Deviator Stress on Deformation
(series 3)
Series Data of Permanent Deformation(log-log)
105% OMC, 95% MDD, s3 = 21 kPa
Cycle = 2 sec. Load Duration = 0.1 sec.

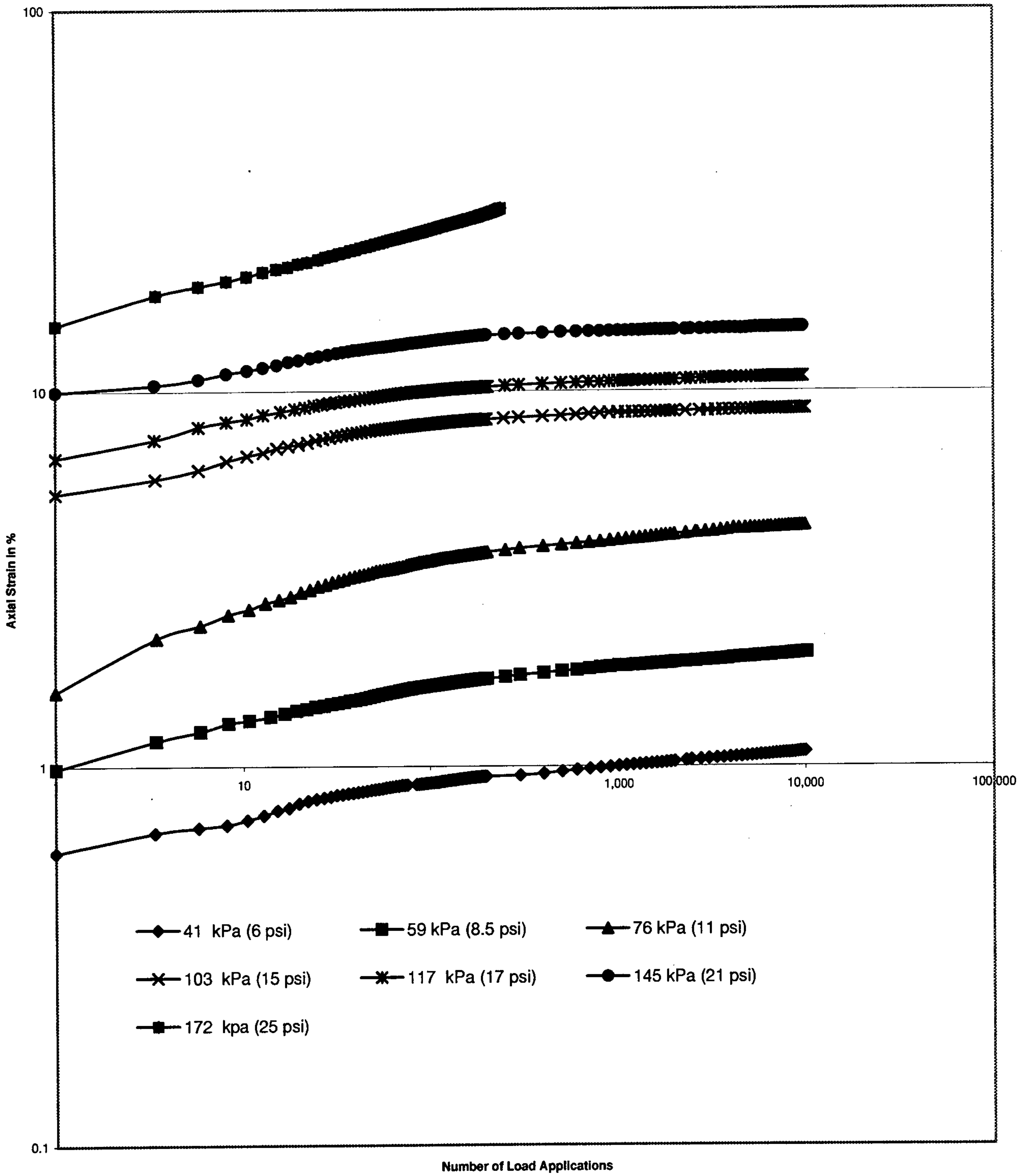


Figure A-20 Effects of Deviator Stress on Deformation (series 4)

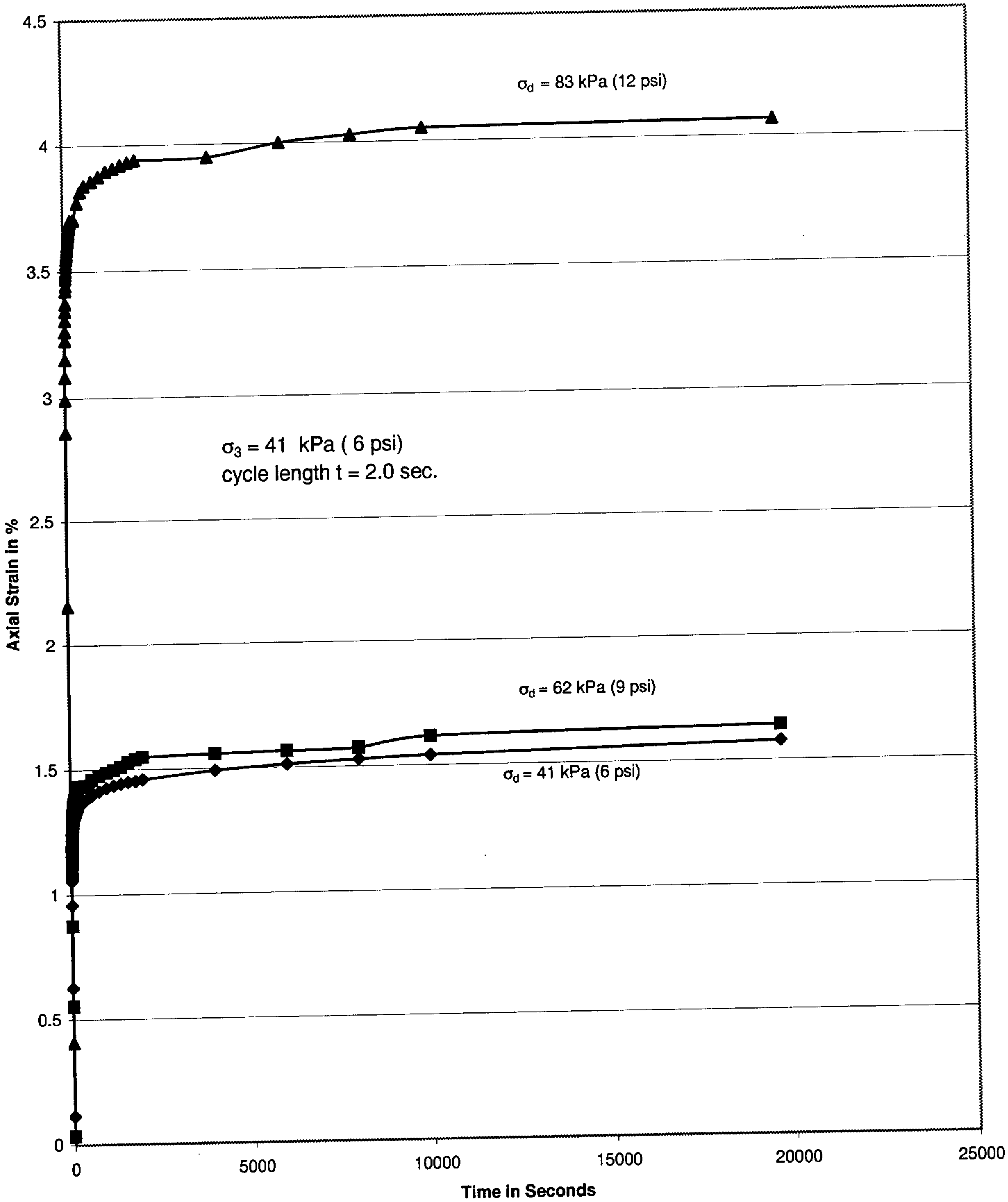


Figure A-22 Effects of Stress History on Deformation (series 2)

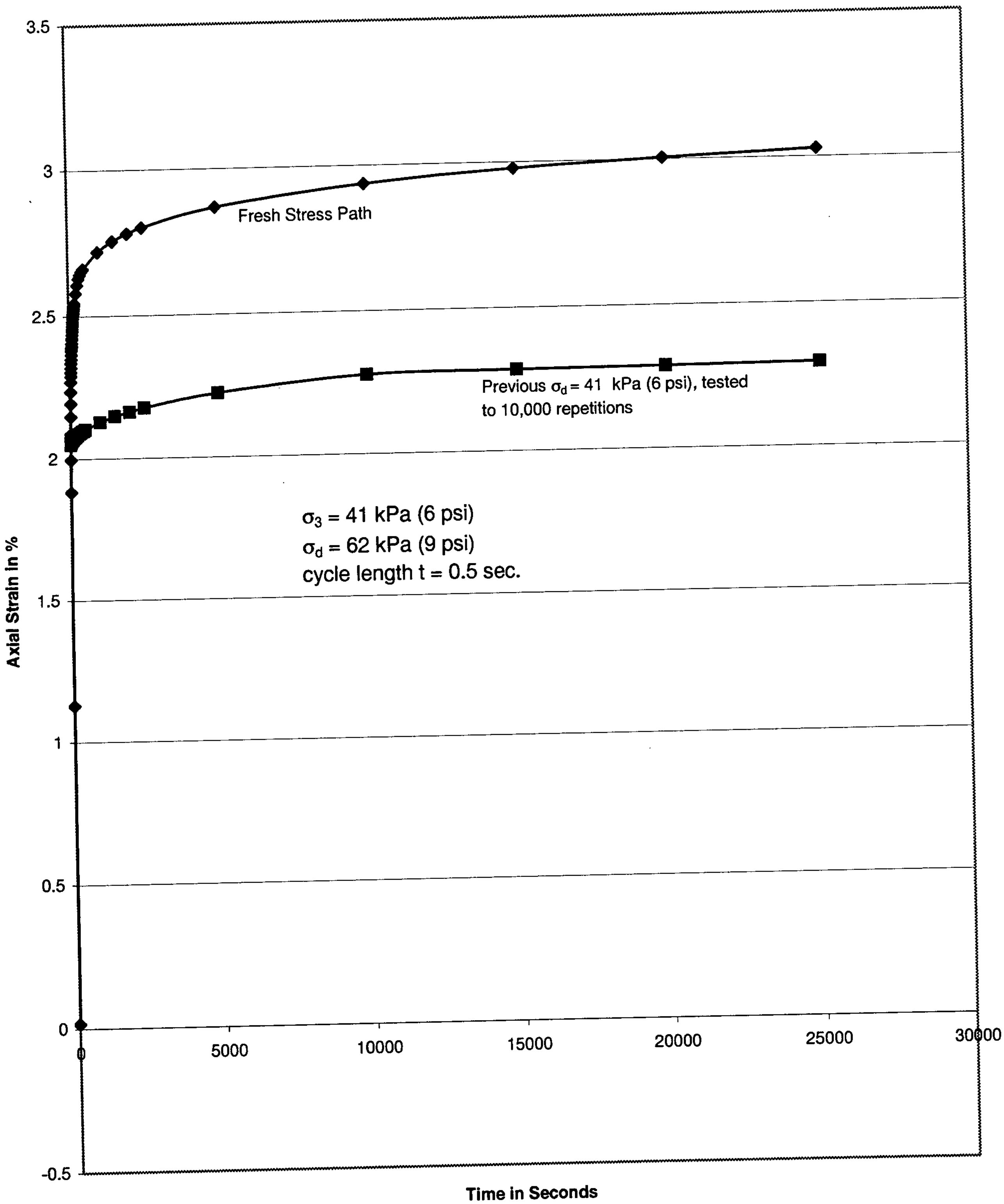


Figure A-23 Effects of Stress History on Deformation (series 3)

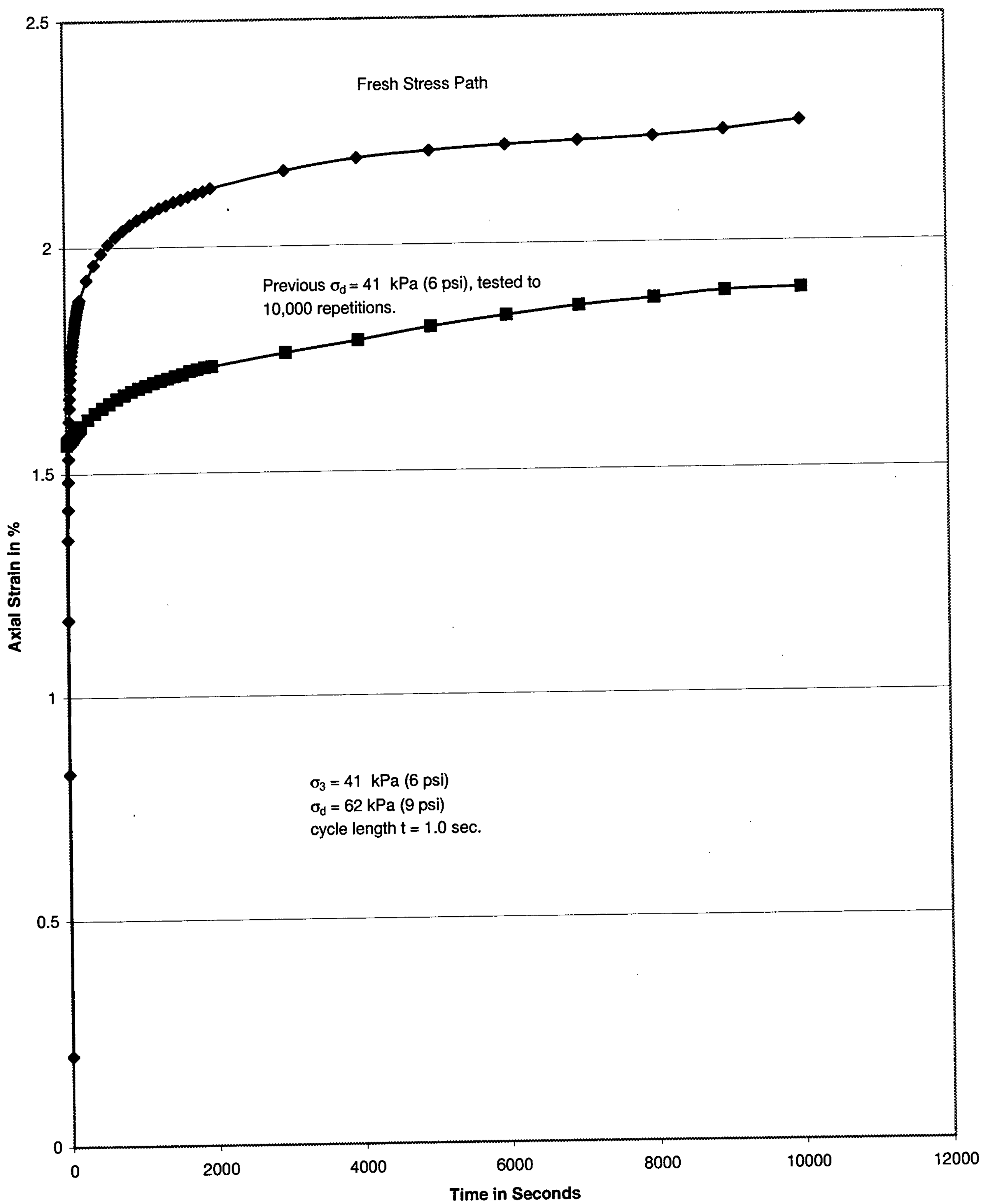


Figure A-24 Effects of Stress History on Deformation (series 4)

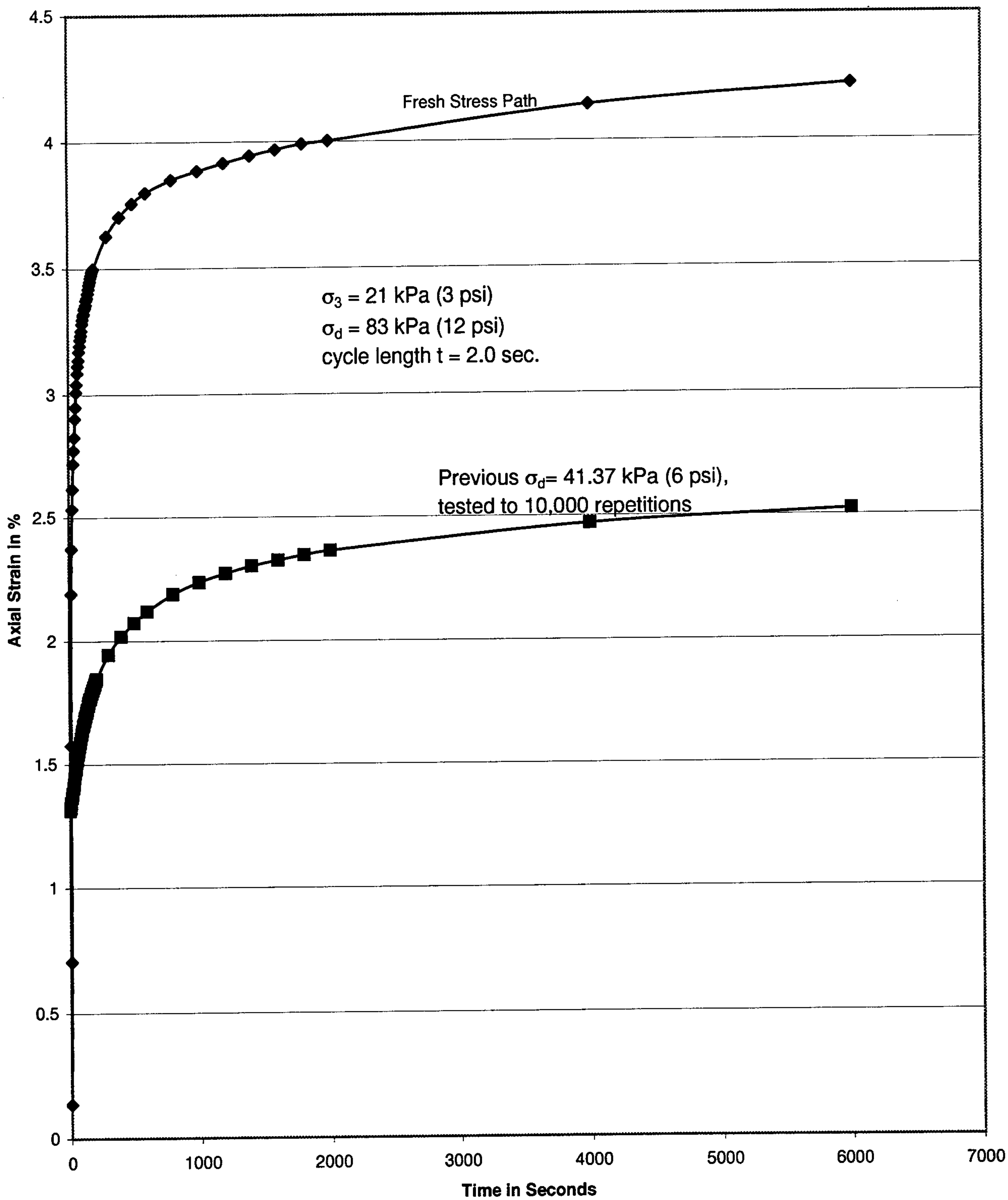
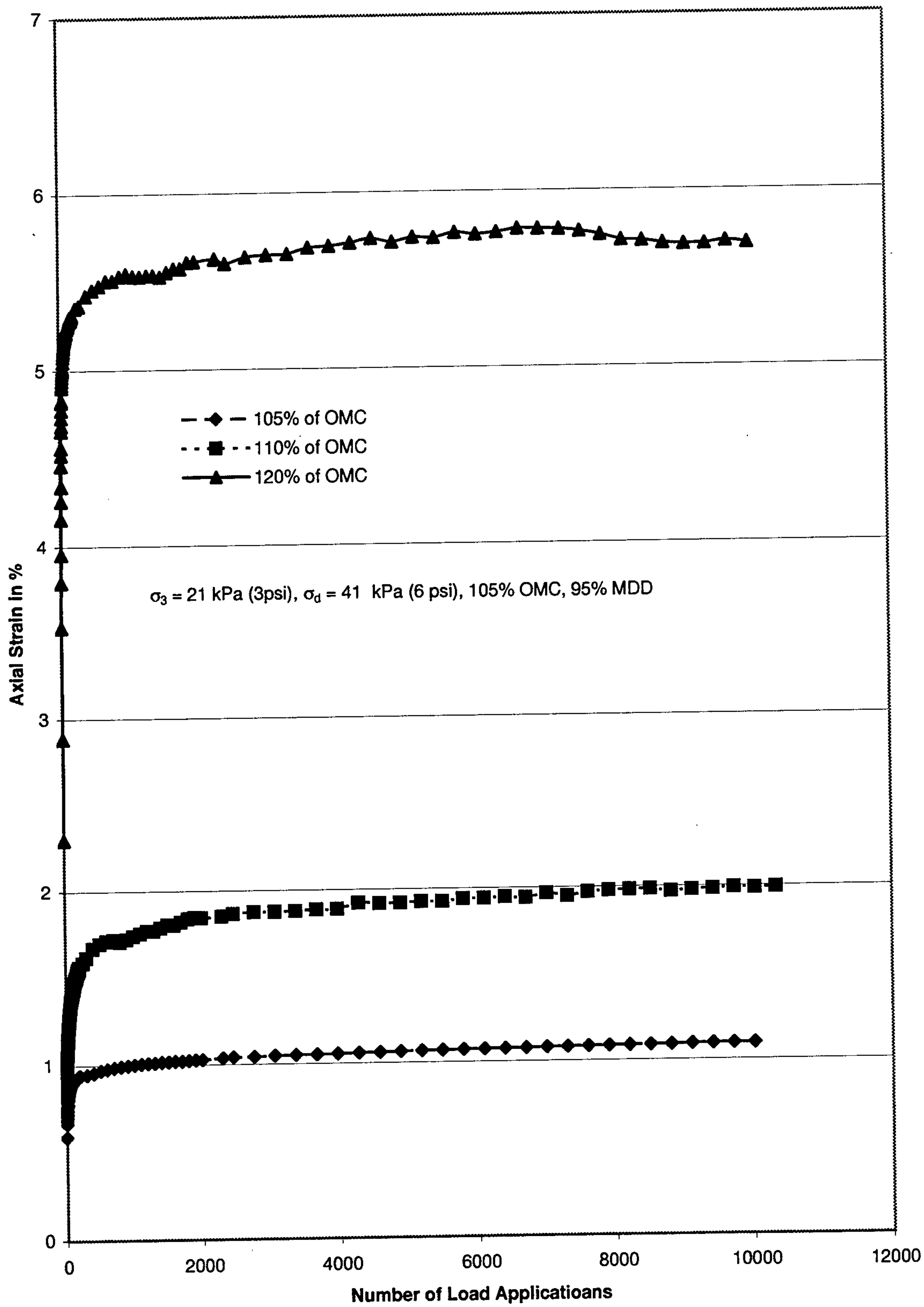


Figure A-25 Effects of Moisture Content on Deformation



Appendix B Detailed Procedure for Compaction

1. Turn on the air-pressure and MASTER SWITCH of the kneading compactor.
2. Make sure that the pressure is set to required magnitude
3. Set the compaction mode onto the base and lock it.
4. Put a greased rubber disk inside the mode, on the base.
5. Turn on the COUNT switch and zero it.
6. Set the chute in the chute-holder such that it could feed the soil into the mold properly.
7. Weigh the soil mixture into three equal-size portions.
8. Take one portion of the soil mixture, put it in the chute, and feed it through the chute into the mold.
9. Use a straight edge to stir up the mixture in the mold such that a slight mound would be formed.
10. Lower down the tamping foot by: (1) turning the direction switch to down-position; (2) switching the pressure latch to the driving-pressure position; and (3) switching back to compaction-pressure position once the tamping foot began to move.
11. Once the tamping foot is about 50 mm (2 in.) above the soil mixture, hit the START button to start kneading compaction, Keep an eye on the COUNT.
12. Toggle the START button to stop compaction once the count reached the required number. The first layer was finished.
13. Lift the tamping foot by turning the direction switch to up-position.

14. Scarify the surface of the compacted layer.

15. Repeat steps (8) through (14) for the second portion of soil mixture to get the second layer compacted.

16. Set the greased collar on top of the mold.

17. Repeat steps (8) through (13) for the third portion of soil mixture to get the third layer compacted.

18. Remove the mold together with collar away from the base. Life the collar. Trim the top of the compacted mixture into an even surface.

19. Turn off the MASTER SWITCH and air pressure. Release the air and lower down the tamping foot to maximum travel to avoid system clogging.

20. Ready for extrusion.

Appendix C Detailed Procedure for Repeated Load Testing

1. Check the power, pressure, and software.
2. Hit the INTERLOCK RESET button and then RESET button. Hit the HYDRAULIC PRESSURE button to turn on the hydraulic system and then toggle it to HIGH to get high pressure. Zero the count.
3. Put a porous stone of the same diameter as that of specimen on the base of triaxial chamber.
4. Set the specimen on top of the porous stone in step (1)
5. Put another porous stone of the same diameter of that in step (1) on top of the specimen. Add a top platen on top of the porous stone. Put a loading ball on top of the platen.
6. Set the chamber on the base to check the alignment. If alignment is not good enough, lift the chamber and mover the specimen along with porous stones, platen, ball to keep the alignment.
7. After checking the alignment, lift the chamber and wrap the specimen along with porous stones and platen with a membrane using a membrane expander.
8. Set the chamber and fasten the four screws.
9. Put a loading ball on top of load piston
10. Put the sliding rod back into the tube of measuring LDT.
11. Lower down the loading ram gradually such that it would touch the loading ball on top of the piston without significant load by tuning the SET-POINT switch counter-

clockwise (This load could be checked in the display window by setting the DISPLAY to DC mode).

12. Limit this pre-load to acceptable magnitude (This pre-load is necessary to keep the test configuration in contact. The deviator load that would be applied later could not be kept stable unless a certain magnitude of pre-load was present.) For deviator stress less than 62.055 kPa (9 psi), a pre-load of 13.344 N (3 pounds) is desirable. For deviator stress greater than 62.055 kPa (9 psi), a pre-load of 17.792 N (4 pounds) to 22.24 N (5 pounds) is acceptable.
13. Apply confining pressure as required by turning on pressure switch in the gas-control panel.
14. Hit the RUN button in the HP-VEE computer screen to monitor the settlement of configuration. After both the load out and the deformation out become stable, hit the STOP button to end monitoring. This process would take 5 to 10 minutes.
15. Hit the RUN button in the HP-VEE computer screen to begin test. After at least one data point was acquired, toggle the CONTROL-MODE button to REMOTE-position and apply the deviator stress to the required magnitude.
16. Keep tuning the SPAN 1 (load-control) in the MTS control panel to stabilize the applied load.
17. Check the load and data out put frequently while the test is running, especially the test would run overnight.
18. When the specimen has been subjected to the required number of load applications, hit the STOP-button in the HP-VEE screen to stop data acquisition. Copy the data file into disk.

19. Toggle the CONTROL-MODE button to LOCAL-position. Remove the applied deviator stress by turning the SPAN 1 switch counterclockwise back to zero. Remove confining pressure. Lift the loading ram by tuning the SET-POINT switch clockwise back to zero. Toggle the HYDRAULIC-PRESSURE button to LOW and then hit the HYDRAULIC OFF button to turn off the hydraulic pressure.
20. Remove the sliding rod from the tube of LDT and put it in a safe place. Dismantle other settings.

Appendix D Recommended Testing Program for Phase II

1. Testing Configuration:

   Load Frequency: 1 Hz

   Load Duration: 0.1 second.

   Rest Period: 0.9 second.

   Confining Pressure: 21 kPa (3psi).

   Number of Load Application: 10,000 before failure

2. Compaction Specification:

   Compaction Method: Kneading Compactor.

   Compacted Density: Three Levels (90%, 95%, 100% of OMC).

   Compacted Moisture Content: Three Levels (105%, 110%, 120% of MDD).

3. Deviator Stress: Starting from 28 kPa (4 psi) up to quick failure.

4. Soils to be Tested: 7 soils will be tested. The selection of these soils was based on the soil association, coverage, distinct properties. TRC-94 was used as the source in selecting these soils. Table D-1 listed those 7 soils and their related properties.

Table D-1 Soils to be Tested in Phase II

#	Soil	Coverage (%)	LL	PI	AASHTO	OMC (%)	% clay
1	Enders	10.8	22.3	4.0	A-4(1)	17	23
2	Carnasaw	9.4	32.8	10.0	A-4(5)	15	25
3	Sharky	8.3	71.4	36.3	A-7-5(43)	28.5	57
4	Calloway	6.2	34.8	12.5	A-6(3)	17.4	11
5	Sacul	6.1	33.6	11.6	A-6(5)	19.5	23
6	Houston	1.0	59.3	37.7	A-7-6(35)	16	34
7	Jackport		54.9	33.8	A-7-6(32)	20	41