"UPDATE ON THE MECHANICAL AGING OF SOILS (25TH TERZAGHI LECTURE)"

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INTRODUCTION

Time marches on. So do all types of soil aging effects, including the mechanical effects described in the writer's Terzaghi Lecture paper (Schmertmann, 1991), herein abbreviated TL. The lecture and paper appear to have helped generate interest in the subject, as evidenced by this Symposium. This update provides a brief review, more soil aging examples, plus some new thoughts and data related to the subject of soil aging.

REVIEW

Table 1 herein, provides a repeat of Table 5 in the TL. It gives a summary of the lab and field aging examples in the paper, and includes natural and artificial soils, sands and clays, for small and large strain moduli, shear strengths, penetration tests, pile capacity, and a possible non-flow reason for pore pressure dissipation. The writer intended this table to show the wide range of geotechnical observations of aging effects and their potential importance as shown by the typical 50-100% improvement in the given behavior due to aging.

Figure 1 herein, a repeat of Figure 38 in the TL, demonstrates how aging produces an increase in the low-strain mobilization of tan ø" basic friction. It shows further that one can associate this aging improvement behavior with increased interlocking, plus an increase in the dispersion (reduced flocculation) of the clay structure due to particle slippages and internal arching.

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The lecture ended with some Conclusions and Practical Applications as given in two slides, reproduced herein as Table 2 and Table 3. Please refer to the TL for more complete conclusions and the supporting data.

ADDITIONAL EXAMPLES

During the past two years the writer has accumulated additional examples of aging effects that add to and perhaps expand upon those presented in the TL, as follows:

Aging Preconsolidation: More examples of such aging in geologically normally consolidated clays continue to come to the writer's attention. Ellstein (1992) noted that Mexico City clays, below the desiccation crust and over the 7 m to 15 m depth interval, had an apparent aging OCR of around 1.45. He also pointed out that this OCR allowed the construction of one- or two-level housing without experiencing the NC settlements predicted by (ordinary) oedometer tests. Duncan et. al. (1991) presented another example in San Francisco Bay mud, again below a desiccation crust.

Field G_0: Consider Figure 8 in the TL. On page 1307 of the TL the writer noted that an initial attempt by Stokoe to provide an updated data point for Figure 8 failed to show the expected increase in G_0. He and his students have now made another attempt, which again failed to show the G_o increase at the 7' depth. He suspects that this results from measured long-time clay swelling at the 7' depth. However, Long (1980) also obtained similar but less data at the 13' depth. Stokoe (1993) reports that their latest field work also included the 13' depth and he has provided Figure 2 which combines Long's 1980 data with the latest data point measured 13 yrs. later. At the 13' depth the aging trend appears to have continued over 13 more yrs, with N_G = 18%. He reports that the lab N_G on undisturbed samples ≈ 10% suggesting that in some soils aging effects can occur more rapidly in the field vs. in the lab. Stokoe also reports that a similar series of tests at the 22' depth at the same site produced results similar to Figure 2. These data remain perhaps the only published example of an aging G_0-increase effect in the field.

Pile Set-up: Seidel et. al. (1988) reported a comparison between results from dynamic and static tests on an about 10 m (30 ft) long, prestressed concrete pile driven through alluvial sands and into a limestone. Dynamic tests performed at the end of initial driving, and 7 and 535 days thereafter, gave capacities of 2290, 3290, 4120 kN (257, 370, 463 tons), respectively. The paper points out that, because the soil consisted of sand, the authors did not anticipate the observed set-up between the 7-day and 535-day tests.

A paper by Riker and Fellenius (1992) described a CAPWAP and static test result comparison and used pile set-up in sand to help explain the comparison. They used the aforementioned Seidel et. al. (1988) reference, as well as the TL for support. It appears that the profession has begun to take note of important pile set-up effects in sands.

However, not all piles experience an aging set-up. Bond and Jardine (1991) in their Figure 21 present and example of fast- and slow-jacked piles in a highly OCR clay that showed no capacity increase with time up to 500 days after installation. The writer has also become aware of case history wherein the lack of an expected set-up embarrassed the engineers involved and forced the use of expensive splicing. In view of our present imperfect knowledge of when and when not to expect aging set-up, it seems prudent to check this with local experience and better still with a test pile program before design.

Explosives in Sand: Thomann and Hryciw (1992) presented data showing how the cone penetration test (CPT) resistance q_c increased with time to at least 217 days after a test blast in a partially saturated, medium-dense sand. The increase from 1-217 days ranged from approximately 70% at 0.6 m to 30% at 2.1m from the blast. Jefferies

(1991) gave another example, reproduced here as Figure 3. Charlie et. al. (1992) present still another example wherein q_c increased 18% over 18 weeks after blasting, but the mechanical friction ratio decreased 80%. A recent update, which will become part of their closure, shows q_c increasing 211% in 5.5 years, with the friction ratio decreasing a further 20% from the 18 week value. These examples show that significant aging improvement can take place in sands following the destructuring effects of using buried explosives. These papers do not discuss all the possible reasons for this aging effect, but it seems clear that when using this technique for ground improvement any check testing must incorporate the aging variable.

Tailings Dams: The resistance to earthquakes of old tailings dams has become important in active seismic zones. On this issue the aging improvement of soils appears to have important consequences. Although old tailings dams, built to old construction standards or non-standards, may not have had adequate earthquake resistance at the time of construction (by today's standards), aging effects may make them acceptable today. Troncoso has written extensively on this subject. Figures 4, 5 and 6 herein, taken from Troncoso (1990), demonstrate the importance of aging in tailings dams .composed of silty sands. These data show aging improvements in cyclic strength ratio, SPT-N values, and peak and residual vane strengths.

Highly Organic Soils: The TL, p. 1318, expressed some uncertainty about how highly organic soils behave with respect to aging. Some data by Wolski and Larsson, et. al. (1989), shown here in Figure 7, suggests that highly organic soils such as amorphous peats will rapidly exhibit aging affects. This figure shows that significant aging preconsolidation developed in a peat layer under a research test embankment during aging times of 13 months and 24 months during the second and third stages of loading. The organic layer appeared to show this aging affect more rapidly than the underlying much less organic calcitic marl layer.

Vibration Probes and Stone Column Improvement: The installation of stone columns by the vibro-replacement method produces some destructuring of the soils in the vicinity of the probe and, as with explosives and vibroflotation, one should expect some aging improvement of soil properties in the near vicinity. Figure 8, taken from Baez and Martin (1992), shows a significant aging improvement in CPT bearing resistance over the interval from one day to one year after inserting the stone columns. The effect diminishes as the spacing of the columns increases. The soils involved consisted of an approximate ten foot layer of alluvial, loose to dense medium sand.

Another example involved the installation of stone columns in sand on Treasure Island in the San Francisco harbor. Jones (1993) reported that CPT q_c values over a depth range 0-25 feet, initially 40-60 tsf, improved about 20-30% following a forty day wait after installation. He also reported a previous vibroflotation project at the same site where post-vibroflotation q_c values ranged from 40-60 tsf. After 15 years these had improved to 150-200 tsf, with no apparent reason other than aging.

Pore Pressure Coefficients and Dissipation: Page 1304 of the TL introduced a possible theoretical explanation for why the **A** and **B** coefficients might decrease with aging. The associated Table 4 presented an example from the writer's research. Mayne and Stewart (1990) called attention to another example taken from anisotropicaly consolidated triaxial tests by Koutsoftas and Ladd (1985). These authors gave a 2 and 3-test average example of how $\mathbf{A_f}$ decreased from 1.25 to approximately 0.60 in the relatively short period from 1.5 to 3-5 days following completion of primary consolidation.

The writer found another example in Wu et. al. (1983) of pore pressure dissipation without the expected resulting settlement, similar to that in Fig. 19 of the TL. Figure 9 shows the additional example. A careful analysis comparing the measured with the expected settlements in a clay layer, computed from the piezometer data in that layer and the lab data from all the layers as given in the paper, gave measured/expected ratios of 1/2 to 1/3 over the 250-350 and 425-585 day intervals. It again appears that the pore pressure dissipation in a normally consolidated clay under the center of a large embankment produced much less settlement than expected from an equal change in vertical effective stress.

A possible explanation for the above behavior, as discussed in the TL, involves the soil structure hardening changes that may have occurred during the aging times when the embankment had a constant height. It seems interesting to note here that in some other soils with a very sensitive structure a softening or collapse can occur during such aging times and settlements continue with little or no pore pressure dissipation. Crawford (1992) noted the above and gave a 30-year example of the collapse behavior. It appears we can have settlement without pore pressure dissipation and dissipation without settlement. This suggests the possibility that in nature the case where the settlement matches the dissipation in accord with currently conventional thinking may be a special case in a wide spectrum of behavior depending on soil structure changes during aging.

SOME NEW THOUGHTS

This section presents some data and concepts relating to aging phenomena that the writer did not include in the TL because of time and length limitations.

Secondary Compression Versus Aging: As briefly discussed in pages 1289 and 1321 of the TL, important uncertainty still exists regarding the equivalence between secondary compression and aging affects. This is perhaps particularly true because of a concept introduced by Bjerrum (1972) and reproduced here as Figure 10. This figure shows the void ratios reducing with a log increase in time for secondary compression using a series of lines parallel to the virgin compression slope C_c. A subsequent load increase produces a higher rate of strain than the current rate and mobilizes increased 'plastic' resistance from the soil structure. This produces an apparent overconsolidation effect. But this overconsolidation results only from the previous secondary compression, and will disappear with continuing secondary compressions. Bjerrum's explanation of the aging preconsolidation affect has received common acceptance and one often finds it repeated in other publications. However, as shown by the explanation of Fig. 1 in the TL and here in Figure 12, described subsequently, it does not properly include aging effects.

The "Bjerrum Model", as well as all spring-dashpot or other mathematical models known to the writer that attempt to simulate the combination of primary and secondary compression affects, predicts an increased compression with increased time of loading. Figure 11, based on Fig. 10, illustrates the void ratio vs. log stress paths for only primary and only secondary compressions -- along the virgin C_c(O-P) and vertical (O-S) paths, resp. Presumably all in between paths (O-PS) represent combinations of primary and secondary. All indicate a decrease in void ratio for a given effective stress change 1-2 with an increase in total compression time. However, Figure 12 shows contrary data.

In Fig. 12 the writer investigated the effect of varying the time for a given normal consolidation effective stress loading increment on the void ratio reduction achieved in duplicate specimens of kaolinite. The many tests plotted show a scatter of results but also a clear trend for decreasing void ratio change with increased time to complete the consolidation increment. The writer also performed a single, 7½-day constant-rate-of-loading test using a natural but extruded residual clay as a check that this behavior would also occur in at least some natural soils. Figure 12 also shows this check.

The data in Figure 12 may demonstrate that some form of aging soil structure stiffening took place simultaneously with the additional loading times used for the same increase in effective stress. In these clays the additional time, presumably mostly 'secondary', produced less void ratio decrease than that expected had such age-stiffening not occurred.

The writer searched the lab test records in detail for other explanations for the decrease in void ratio trends in Figure 12. He found only one possibly creditable: Progressively longer loading times may progressively lessen the relative 'destructuring' of the extruded and stored clay specimens and thus reduce the void ratio change required for equilibrium under the higher effective stress. Direct data does not exist to confirm or deny this possibility for these tests. Indirectly, single load increment, suddenly applied, isotropic consolidation tests over the 0-1 to 0-7.5 ksc range give an essentially constant C_c, rather than the increasing C_c one might expect if the relative destructuring increased with the magnitude of the loading 'shock' to the initial structure. The aforementioned aging explanation seems, to the writer, as more likely correct.

All the less-than-1-day data points in Figure 12 show the void ratio decrease after either approximately a 1 day (■ pts) or a 1 week (▲ pts) secondary. The small average additional void ratio reduction when the secondary compression times increased from one to seven days show the minor effect of secondary compression. These clays had 5- to 8-test average C_α/C_c ratios of only 0.0043 /0.231 = 0.019 for the kaolinite (PI=21%) and 0.0034/0.307 = 0.011 for the Enid residual clay (PI=9%) in isotropic, triaxial consolidation. See Schmertmann (1976) for more information about these clays. Their minor secondary compressions may have allowed the aging effects to dominate and produce higher void ratios for a given consolidation effective stress change despite much longer primary plus secondary consolidation times.

Basic Friction Increase During Creep: Bea (1960) demonstrated that undrained creep for periods of 15-30 days, using extruded Kaolinite and Boston blue clay specimens, changed (aged) their structures in such a way that the basic friction tan ø" (see TL p. 1308) increased during the creep compared to its value at the same strain had the interval of creep (aging) not occurred. Figure 13, simplified from Schmertmann (1976, Fig. 24 and 1981, Fig. 10) compares the mobilization of the basic components c" and ø" at the same axial strain in kaolinite with and without undrained creep. All specimens had been consolidated to 3.50 ksc and held in secondary compression for a total of 1 day prior to undrained creep under constant deviator stress.

Note how the mobilized tan ø" after creep strains of approximately 0.7, 1.7 and 5.0% had increased substantially (Δtan ø" shown by hatched areas) versus the tan ø" mobilization without prior creep. Note also how an additional 1-2% compressive strain tends to destroy the additional creep-aging Δtan ø". The writer proposes that these data offer further evidence that aging affects, even in undrained creep, result from an increase in the frictional component of basic shear strength of the soil. The shear strains of creep presumably tend to destroy structure. But, the time involved tends to age-strengthen structure. We observe the net effect. The aforementioned references also show another example of this behavior from extruded, duplicate specimens of Boston blue clay.

Chemical action: The TL notes on page 1319, in connection with the behavior of the recently discovered locked sands, that geologic aging effects from chemically-induced change in grain shapes produces a large interlocking and friction increase effect. Some recent research suggests that similar effects, of smaller but still significant magnitude, may occur over the much shorter times of interest to engineers.

Many investigators have speculated that the oxidation and solution at the surfaces of silicious particles, possibly developing silica gels, in part might account for engineering aging affects. Some recent research by Nieto and his students (1993) appears to have shown that solution of this type occurs rapidly, proportional to the pressure at the contact points, and continues at a reduced rate. This solution would increase the contact area. This might then give the appearance of a strength gain proportional to pressure (therefore engineering friction) and increasing with time. This is another manifestation, known by Terzaghi a long time ago, that mechanical friction actually results from a physical-chemical bonding effect at contact points, with the bonding proportional to the actual contact area. This additional information about the effects of chemical action on aging friction is of interest because it aids our fundamental understanding of one of the possible contributing factors to soil aging. However, from the point of view of the engineer it behaves according to the engineering definition of friction and therefore can be treated as such.

The aforementioned closure by Charlie _et. al._ notes the possible importance of temperature on the rate of aging improvement of q_c in sand. Their data suggest that this rate increased by a factor of 2.6 when the ground temperature increased from 8° to 10°C. A sensitivity to temperature also suggests chemically related effects.

Aging Time Factor: The writer applauds the research efforts of others to extend the data base and attempt to quantify the effects of aging. The aforementioned work by Nieto gives an example of such research. Athanasopoulos (1993) has recently attempted to show that a quantitative relationship may exist between aging preconsolidation and an aging time factor. Of course, one would expect the effects of aging to increase with aging time. As noted on p. 1325 of the TL, the aging affects appear to increase with the logarithm of aging time, starting from some episode of complete or partial destructuring.

Speeding the Aging Process: Studying aging affects, or even recognizing them, has proven difficult because of the awkward need for the passage of large amounts of time. We need methods to speed up the process if possible. Probably nothing will ever substitute completely for the affects of time. However, some possibilities to speed the aging process include overconsolidation, cyclic loading, vibrations, using the centrifuge, and perhaps increasing temperature.

Figure 14, taken from Long (1980, Fig. B.2), shows the interaction between shear wave velocity V_s and the lab and field confining pressure, aging and OC. In this case a lab "undisturbed" sample will need an aging time of approximately 1.6 years to match the field shear wave velocity, V_s, with N_G = 20%, or an OCR of ≈ 4 and aging of only 1000 minutes. Thus, in this example an OCR of 4 and a lab aging of only 1000 min. would give the same V_s as in the field. OC appears to greatly speed the aging recovery from the effects of sampling. This may in part explain the success of the SHANSEP method (Ladd, 1991) which uses OC to recover the effects of sampling. However, one probably cannot duplicate all the effects of long aging in the field by a simple lab OC procedure.

The TL noted on page 1295 that cyclic loading may speed the aging process. This probably occurs because the cyclic loading helps speed the shifting of particles in a way perhaps generally similar to the particle movements associated with aging. This suggests to the writer that low-intensity vibrations would probably have a similar affect -- however he has no data at present in support.

Using a centrifuge represents another possibility. We know that the times for hydrodynamic consolidation decrease by the factor N^2, where N represents the gravity increase factor induced by the centrifuge. However, researchers have yet to determine the centrifuge scaling factor that applies to aging.

CONCLUSIONS

1. The evidence for aging effects continues to accumulate, perhaps at an increasing rate because of a new awareness. This paper includes some more examples.

2. Possibly because of aging affects, and contrary to primary and secondary consolidation as commonly understood, at least some soils will show a reduced consolidation void ratio change when loaded at a slower rate.

3. Aging effects play a part in explaining creep behavior. Basic friction appears to increase, at least in some soils, during creep as it does during aging without the overall shear strains of creep.

4. Chemical reactions may explain a part of the apparent increase in basic friction during aging.

5. Researchers perhaps now have a renewed emphasis on trying to quantify aging affects. Providing methods for accelerating aging affects in the laboratory would greatly aid this effort and perhaps accelerate its application into practice.

REFERENCES CITED

Athanasopoulos, G. A., (1993), "Reconsolidation Versus Aging Behavior of Kaolinite Clay", Technical Note Journal of Geotechnical Engineering, ASCE, Vol. 119, No. 6, June, p. 1064.

Baez, J. I. and Martin, G. R. (1992), "Liquefaction Observations During Installation of Stone Columns Using the Vibro-Replacement Technique", Geotechnical News, Sep., p. 43.

Bea, R. G. (1960), "An Experimental Study of Cohesion and Friction During Creep in Saturated Clay", Masters Thesis Presented to the University of Florida, Dept. of Civil Engineering, Gainesville, Fl.

Bond, A. J. and Jardine, R. J. (1991), "Affects of Installing Displacement Piles in a High OCR Clay", Geotechnique, Vol. 41, No. 3, p. 355.

Bjerrum, L. (1972), "Embankments on Soft Ground", Performance of Earth and Earth Supported Structures, ASCE, Vol. II, Purdue Univ., Lafayette, Ind., p. 20.

Bjerrum, L. (1972), "Embankments on Soft Ground", Performance of Earth and Earth Supported Structures, ASCE, Vol. II, Purdue Univ., Lafayette, Ind., p. 21.

Charlie, W. A., Rwebyogo, M. F. J. and Doehring, D. O. (1992), "Time-Dependent Cone Penetration Resistance due to Blasting", Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 8, Aug., p. 1211.

Crawford, C. P. (1992), Discussion of Schmertmann (1991), Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 12, Dec., p. 2011.

Duncan, J. M., Javet, D. F. and Stark, T. D. (1991), "The Importance of a Desiccated Crust on Clay Settlements", Soils and Foundations, Vol. 31, No. 3, p. 83.

Ellstein, A. (1992), Discussion of Schmertmann (1991), Journal of Geotechnical Engineering, ASCE, Vol. 118, No. 12, Dec., p. 2012.

Jefferies, M. G. (1991), "Explosive Compaction", Geotechnical News, Vol. 9, No. 2, Jun., Figure 1, p. 30.

Jones, John S. (1993), John S. Jones & Assoc., Purcellville, VA, personal communication.

Koutsoftas, D. C. and Ladd, C. C. (1985), "Design Strengths for Off Shore Clay", Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 3, Mar., p. 343.

Ladd, C. C. (1991), "Stability Evaluation During Staged Construction", 22nd Terzaghi Lecture, Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 4, Apr., p. 568.

Long, L. G. (1980), "Comparison of field and laboratory dynamic soil properties", thesis presented to the University of Texas, at Austin, Tex., in partial fulfillment of the requirements for the degree of Master of Science, 223 pp.

Mayne, P. W. and Stewart, H. E. (1990), Authors' Closure, Journal of Geotechnical Engineering, ASCE, Sep., p. 1440.

Nieto, A. (1993), Professor, Department of Geology, Univ. of Illinois, Urbana, Ill., Personal Communication and Contribution to this Symposium.

Riker, R. E. and Fellenius, P. H. (1992), "A Comparison of Static and Dynamic Test Results", Preprint, Fourth International Conference on the Application of Stress-Wave Theory to Piles, Den Haag, Sep..

Schmertmann, J. H. (1976), "The Shear Behavior of Soil at Constant Structure", Bjerrum Memorial Volume, Norwegian Geotechnical Institute, Oslo, Norway, pp. 65-98.

Schmertmann, J. H. (1981), "A General Time-Related Soil Friction Increase Phenomenon", SPT 740, ASTM, Philadelphia, PA., pp. 456-484.

Schmertmann, J.H. (1991), "The Mechanical Aging of Soils", 25th Terzaghi Lecture, Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 9, Sept., pp. 1288-1330.

Seidel, G. P., Haustorfer, I. J. and Plesiotis, S. (1988), "Comparison of Dynamic and Static Testing for Piles Founded into Limestone", Proceedings of the Third International Conference on the Application of Stress-Wave Theory to Piles, Ottowa, May 25-27, pp. 717-723.

Stokoe, K. (1993), Professor Environmental and Civil Engineering, Univ. of Texas, Austin, Personal Communication.

Thomann, T. G. and Hryciw, R. D. (1992), "Stiffness and Strength Changes in Cohesional Soils due to Disturbance", Canadian Geotechnical Journal, Vol. 29, No. 5, Oct., p. 859.

Troncoso, J. H., Tshihara, K. and Verdugo, R. (1988), "Aging Effects on Cyclic Shear Strength of Tailings Materials", Proceedings IX World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan.

Troncoso, J. H. (1990), "Failure Risks of Abandoned Tailing Dams", Proceedings, International Symposium on Safety and Rehabilitation of Tailing Dams, Sydney, Australia, pp. 34-47, Sponsored by International Commission on Large Dams and Australian National Committee on Large Dams.

Wolski, W., Larsson, R., Szymanski, A., Hartlen, J., Lechowicz, Z. and Bergtahl, U. (1989), Swedish Geotechnical Institute Report Number 36, Linkoping, p. 33.

Wu, T. H., Hsu, J. R. and Ali, E. M. (1983), "Plastic Deformation of an Embankment on Clay", Canadian Geotechnical Journal, Vol. 20, No. 3, Aug., p. 449.

TABLE 1 - SUMMARY OF EXAMPLES

Effect	Approx. % improvement	Lab	Field
p_c, M	40-100	X	X
Liquef.	50-100	X	X
G_o	50-200	X	X
E	50-100	X	
s_u	50-100	X	X
q_c, N	30-140		X
Pile Cap.	25-60+		X (1 mo.)
Δu, A	40	X	X

$\Sigma \simeq 50\text{-}100\%$

TABLE 2

ENGR. SOIL AGING: CONCLUSIONS

- General 50-100% effect that increases with log time
- May decrease Δu
- All/Mostly increased soil friction due to grain slips/fabric arching
- Mechanical & usable in practice

TABLE 3

SOME PRACTICAL APPLICATIONS

- Look for aging preconsolidation effect in so-called NC soils
- Correct lab data for aging
- Correct field data for aging after ground improvement
- Expect faster Δu dissipation in near-saturated soils
- Have design include aging effects when appropriate (lower initial F.S.?)

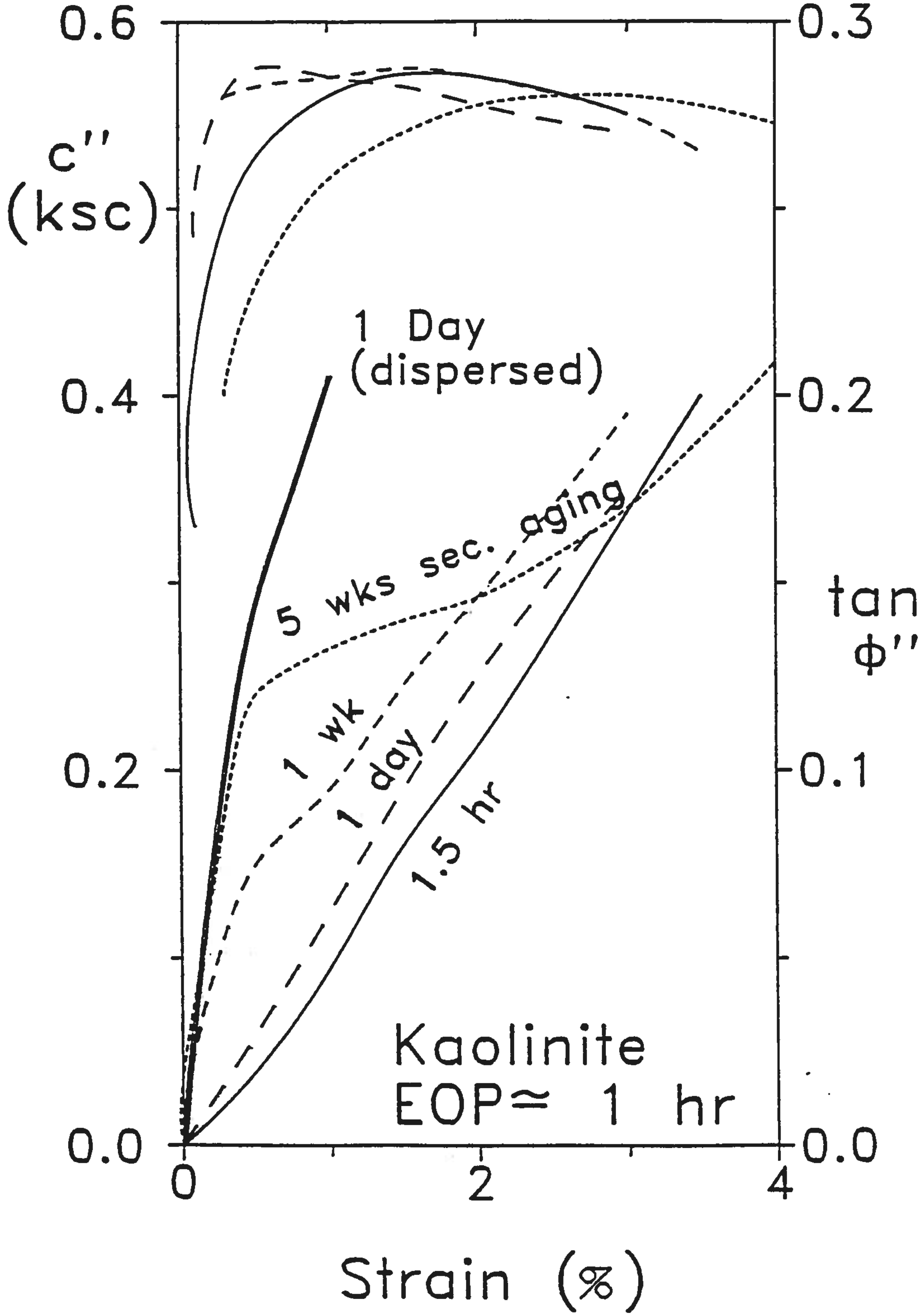

FIGURE 1 COMPARING TAN ø" MOBILIZATION OF DISPERSED KALONITE WITH THAT OF RELATIVELY UNDISPERSED KALONITE SUBJECTED TO VARIOUS TIMES IN SECONDARY COMPRESSION AGING

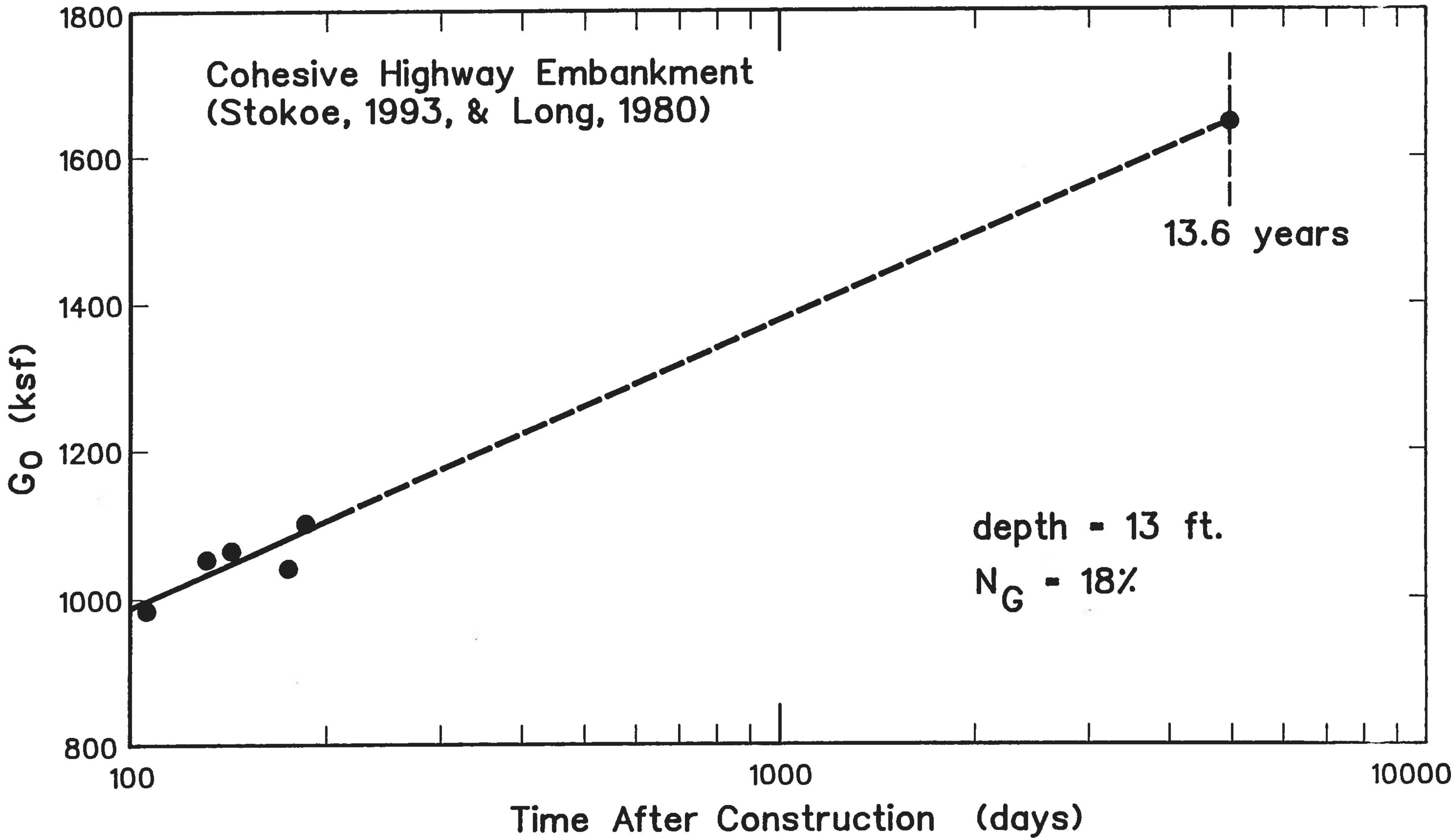

FIGURE 2 - UPDATED EXAMPLE OF SMALL STRAIN MODULUS INCREASE IN FIELD

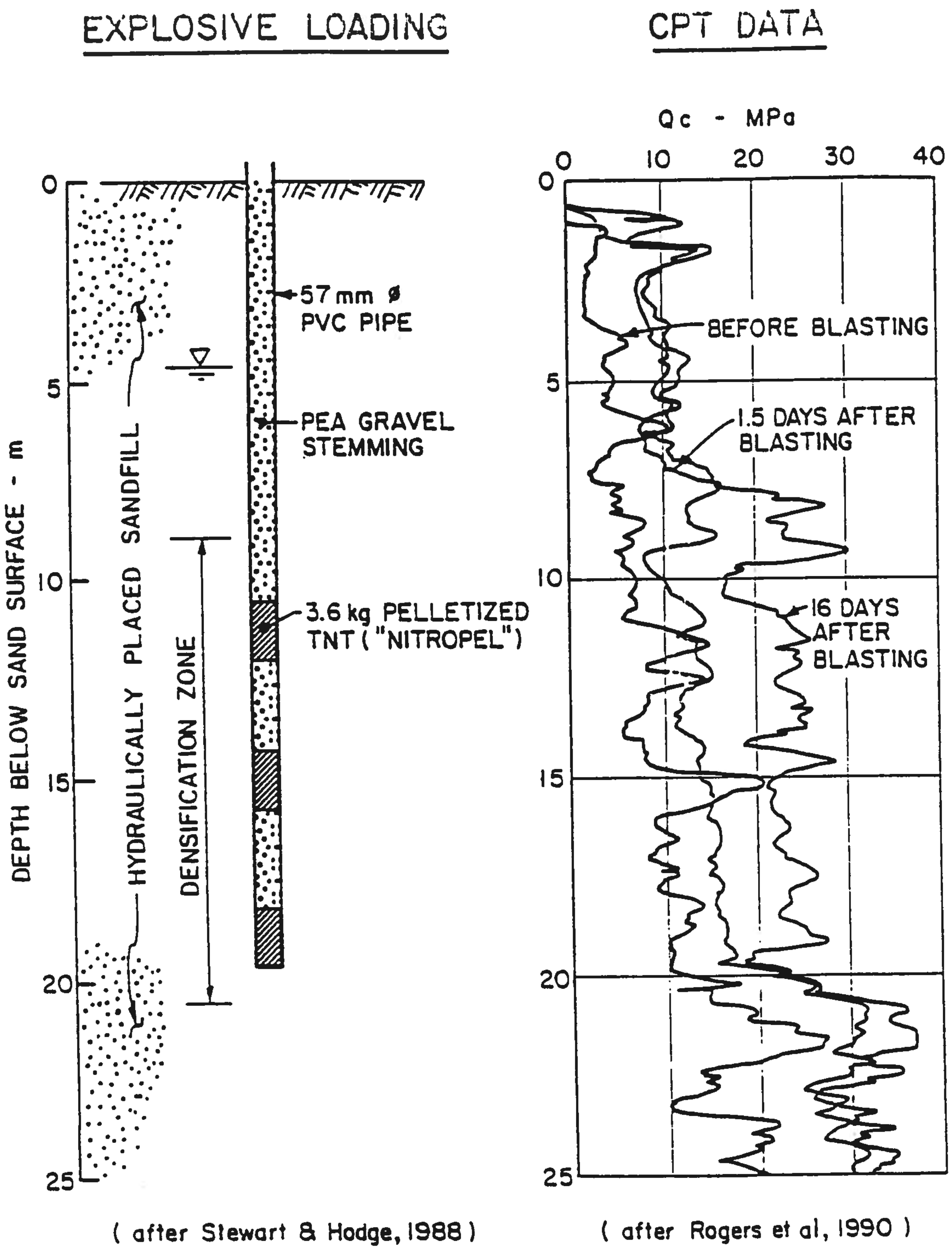

FIGURE 3 - EXAMPLE OF AGING EFFECTS AFTER EXPLOSIVE COMPACTION

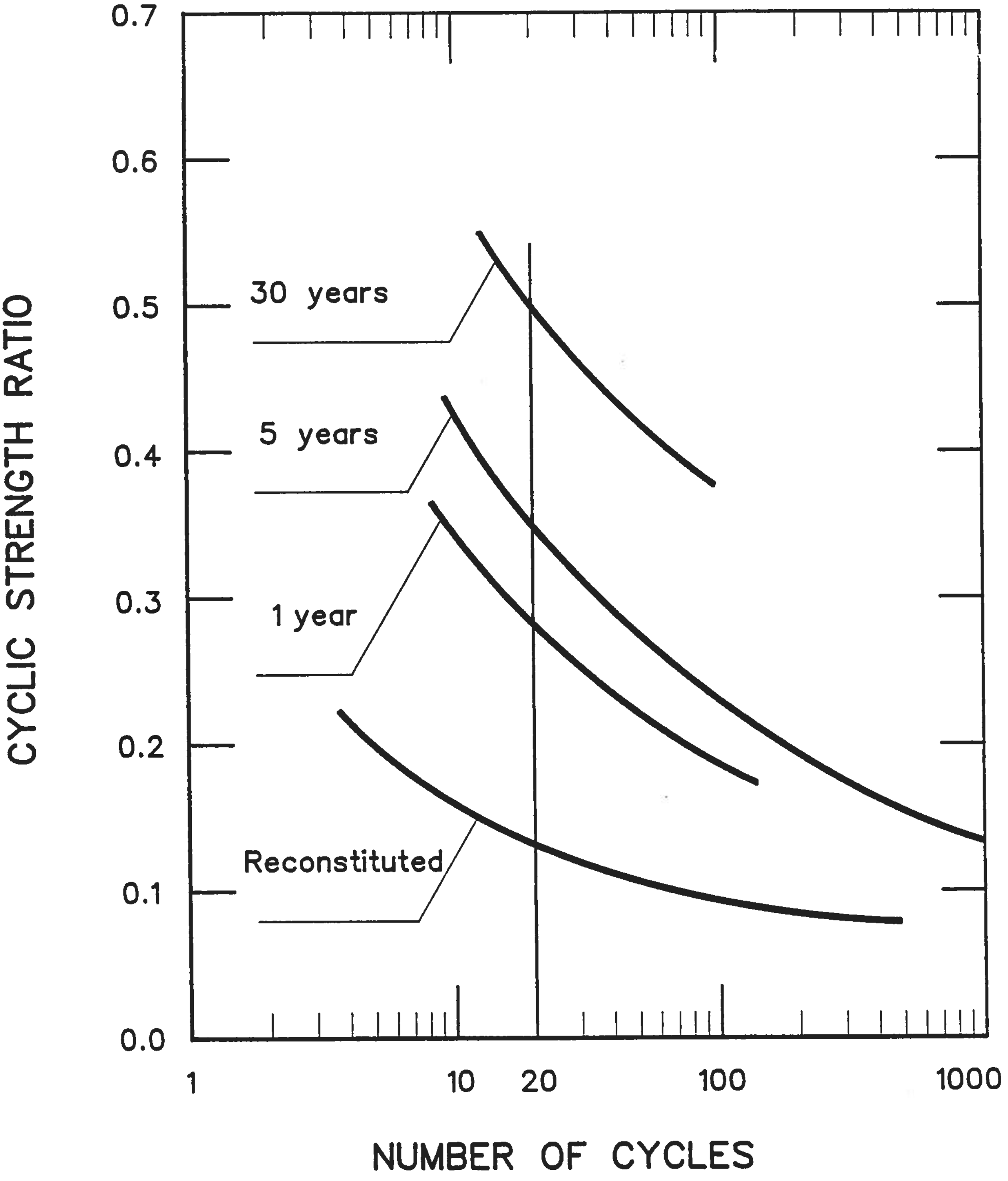

FIGURE 4 - EFFECT OF AGE IN SILTY SANDS FROM TAILINGS DAMS IN CENTRAL CHILE

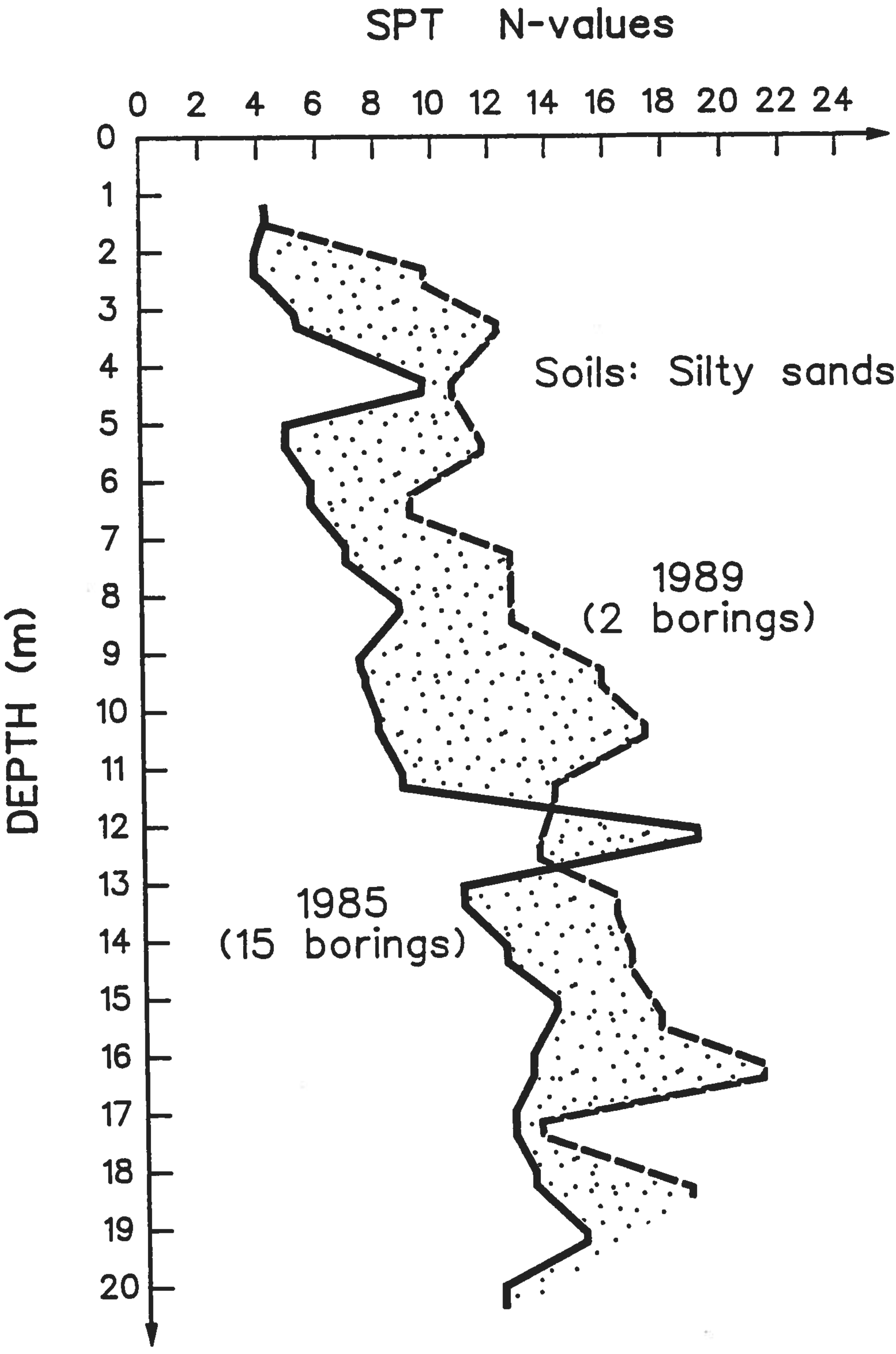

FIGURE 5 - EFFECTS OF AGING ON STANDARD PENETRATION RESISTANCE, EL COBRE #3 TAILINGS DAM (Troncoso, 1990)

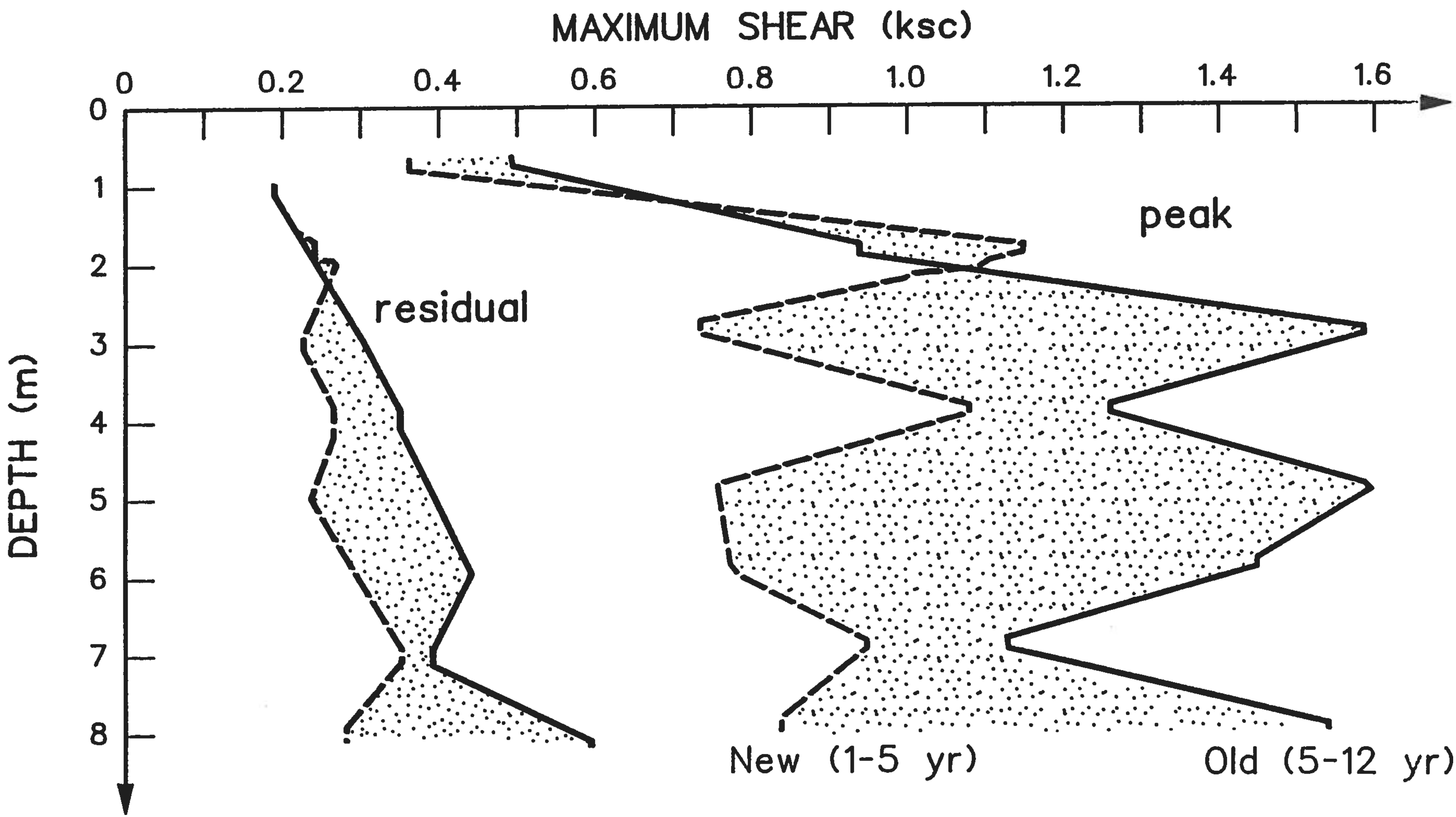

FIGURE 6 - VANE STRENGTHS IN OLD AND NEW TAILINGS DAMS

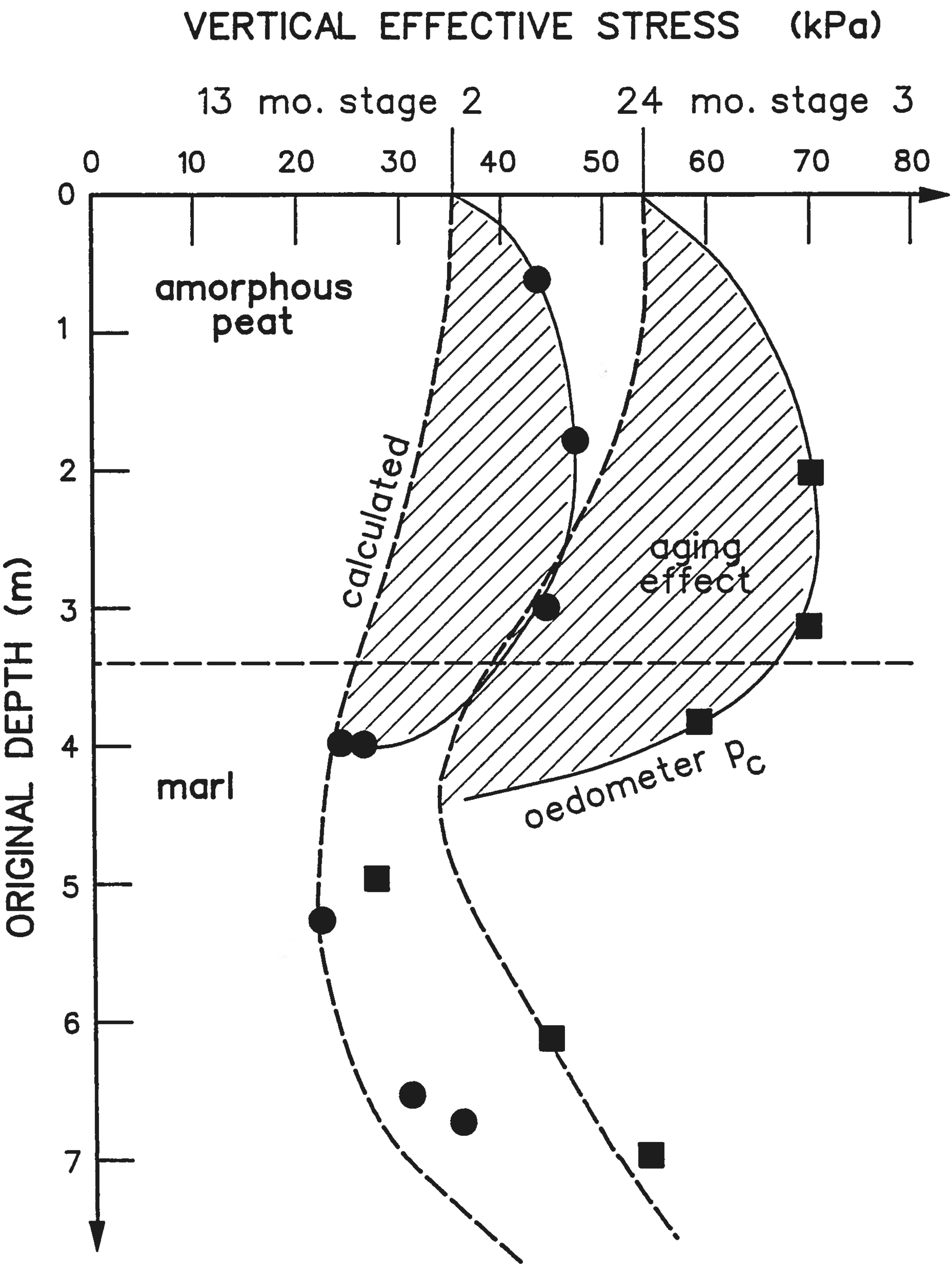

FIGURE 7 - AGING PRECONSOLIDATION IN PEAT UNDER TEST EMBANKMENT (Wolski and Larsson, 1989)

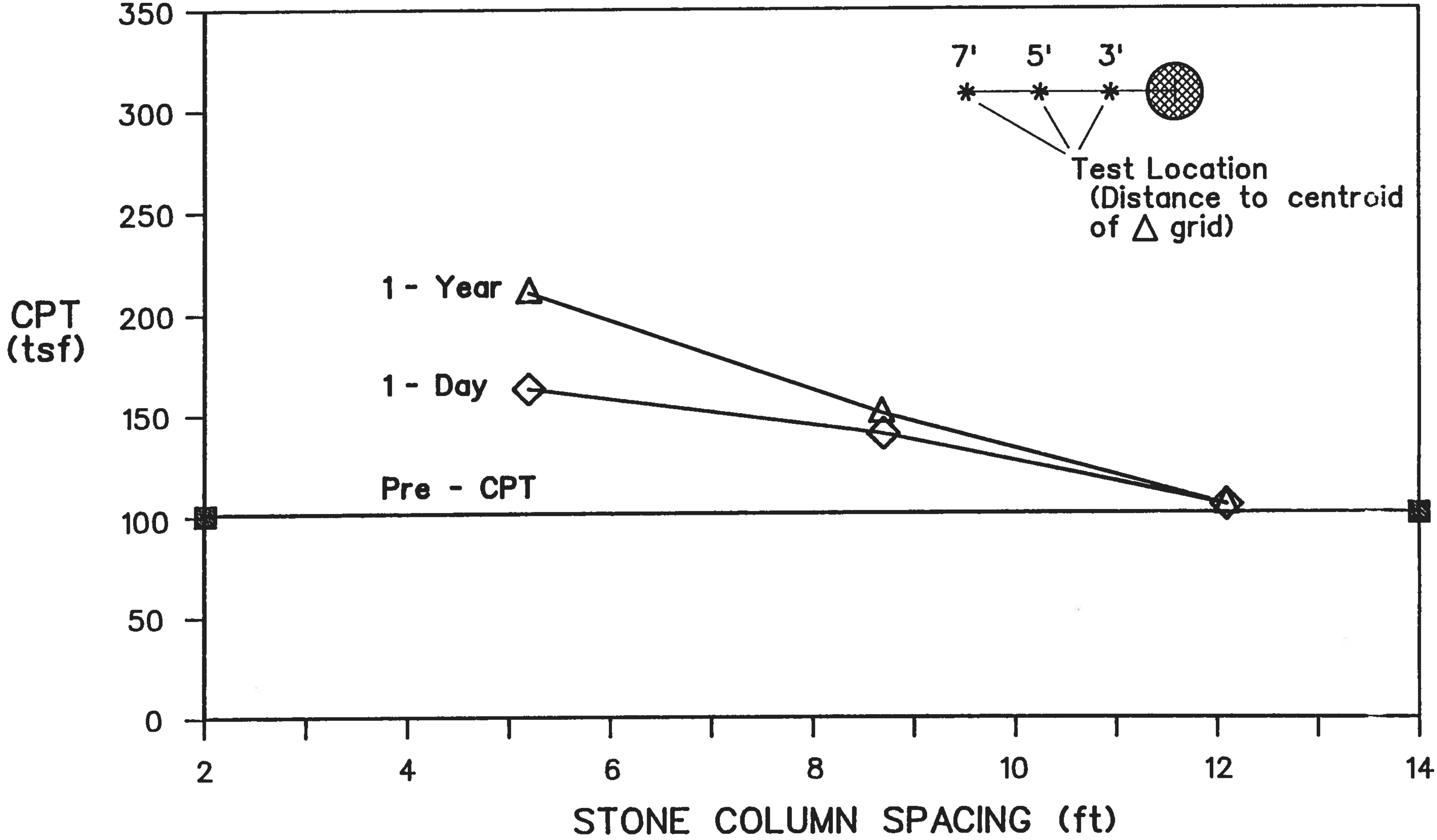

FIGURE 8 - TIME EFFECT ON PENETRATION RESISTANCE
(Baez and Martin, 1992)

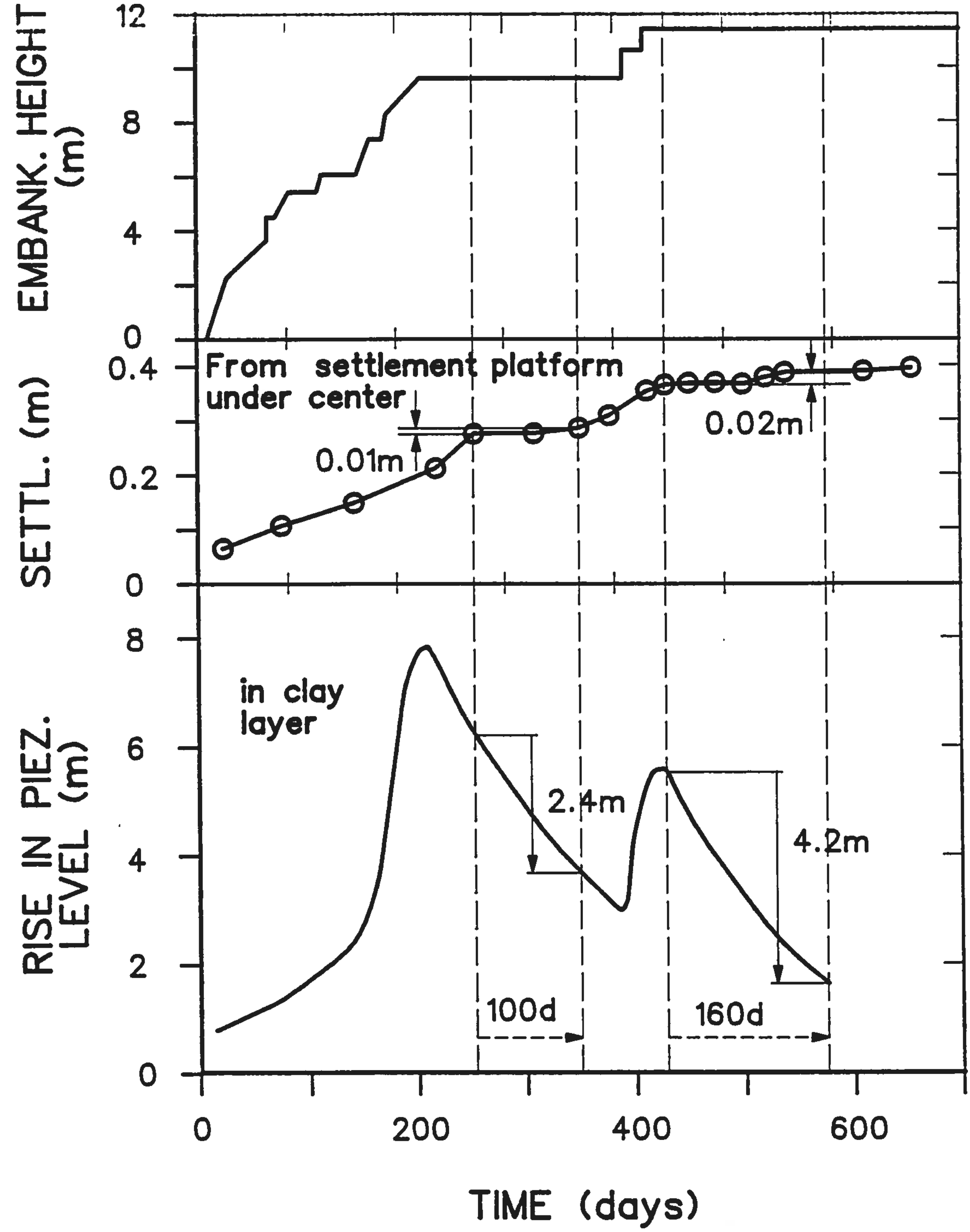

FIGURE 9 - FILL HEIGHT, SETTLEMENT, AND PORE PRESSURE (Wu et. al., 1983)

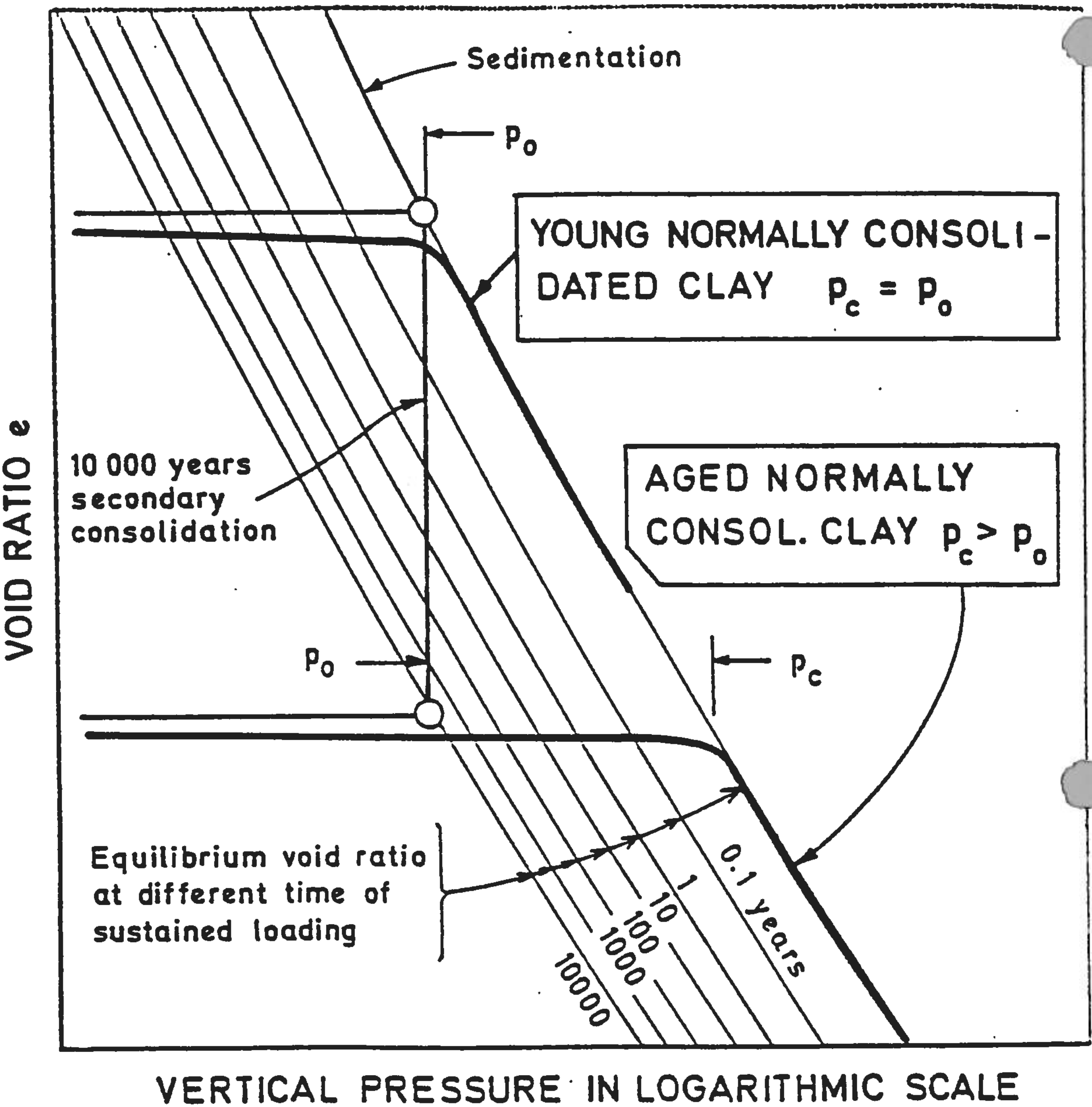

FIGURE 10 - BJERRUM MODEL (1972)

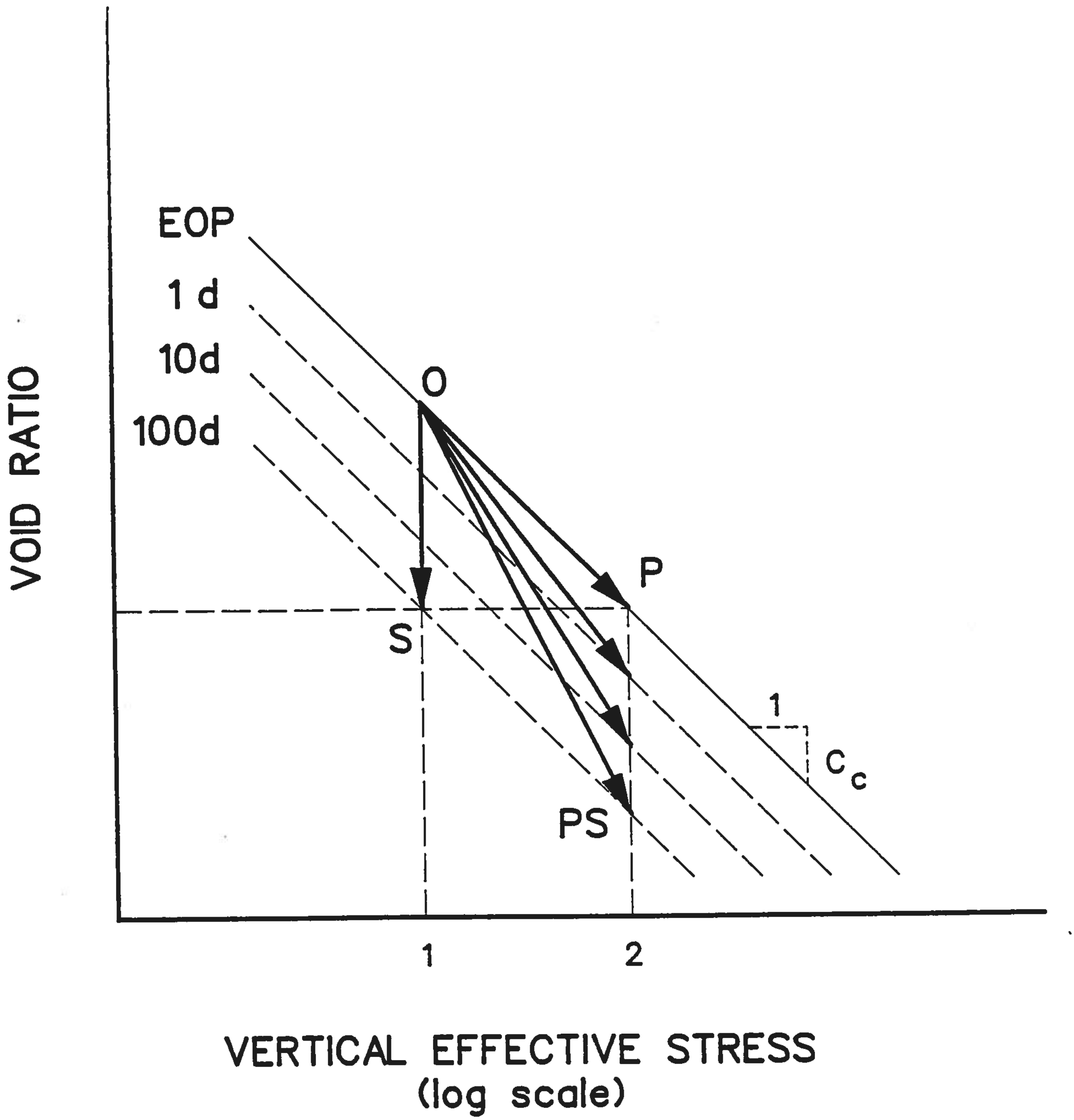

FIGURE 11 - INCREASING PRIMARY+SECONDARY CONSOLIDATION TIME REDUCES VOID RATIO IN BJERRUM MODEL

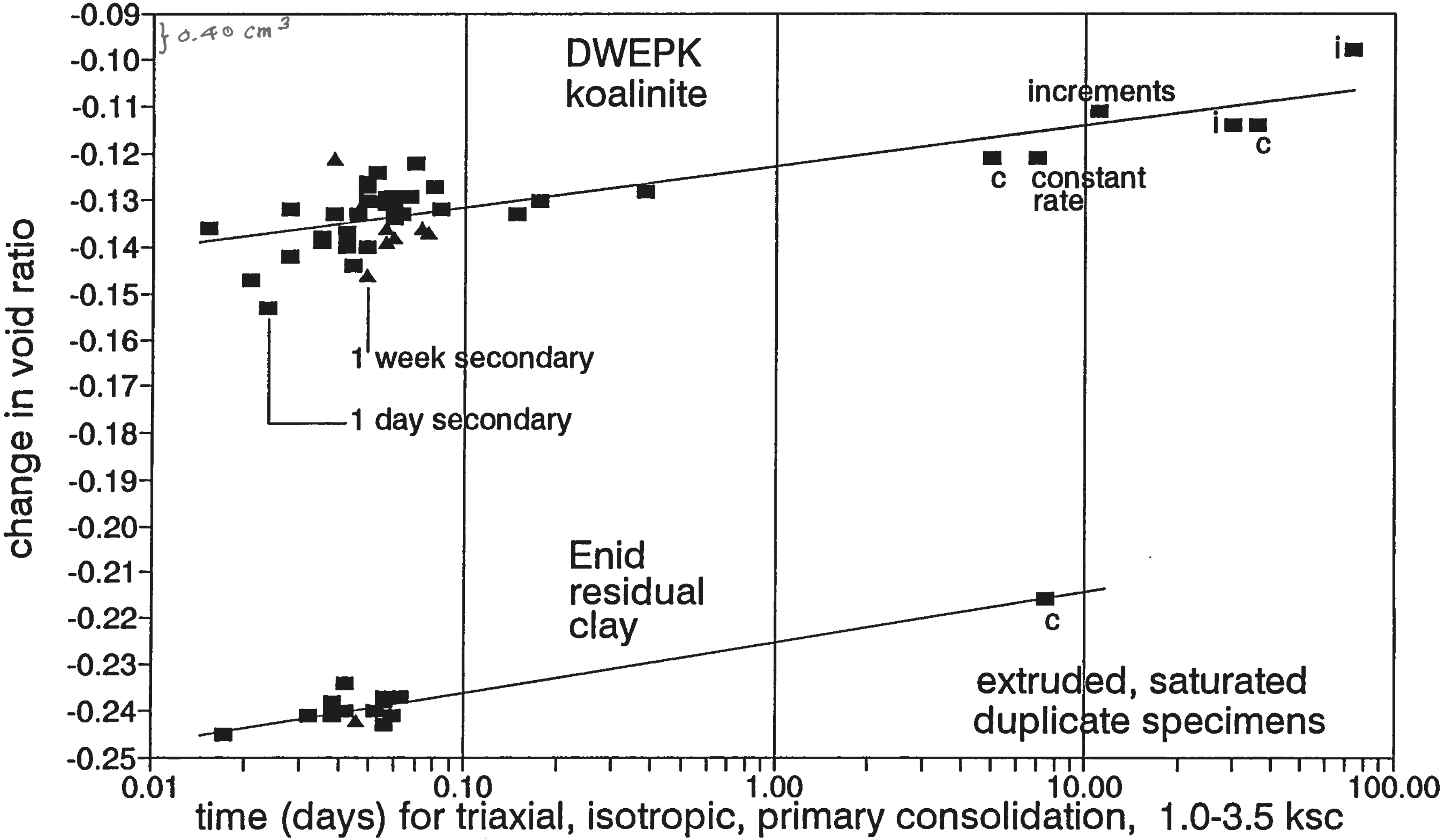

FIGURE 12 - EXAMPLES OF DECREASING VOLUME CHANGE BY INCREASING TIMES TO REACH FINAL STRESS

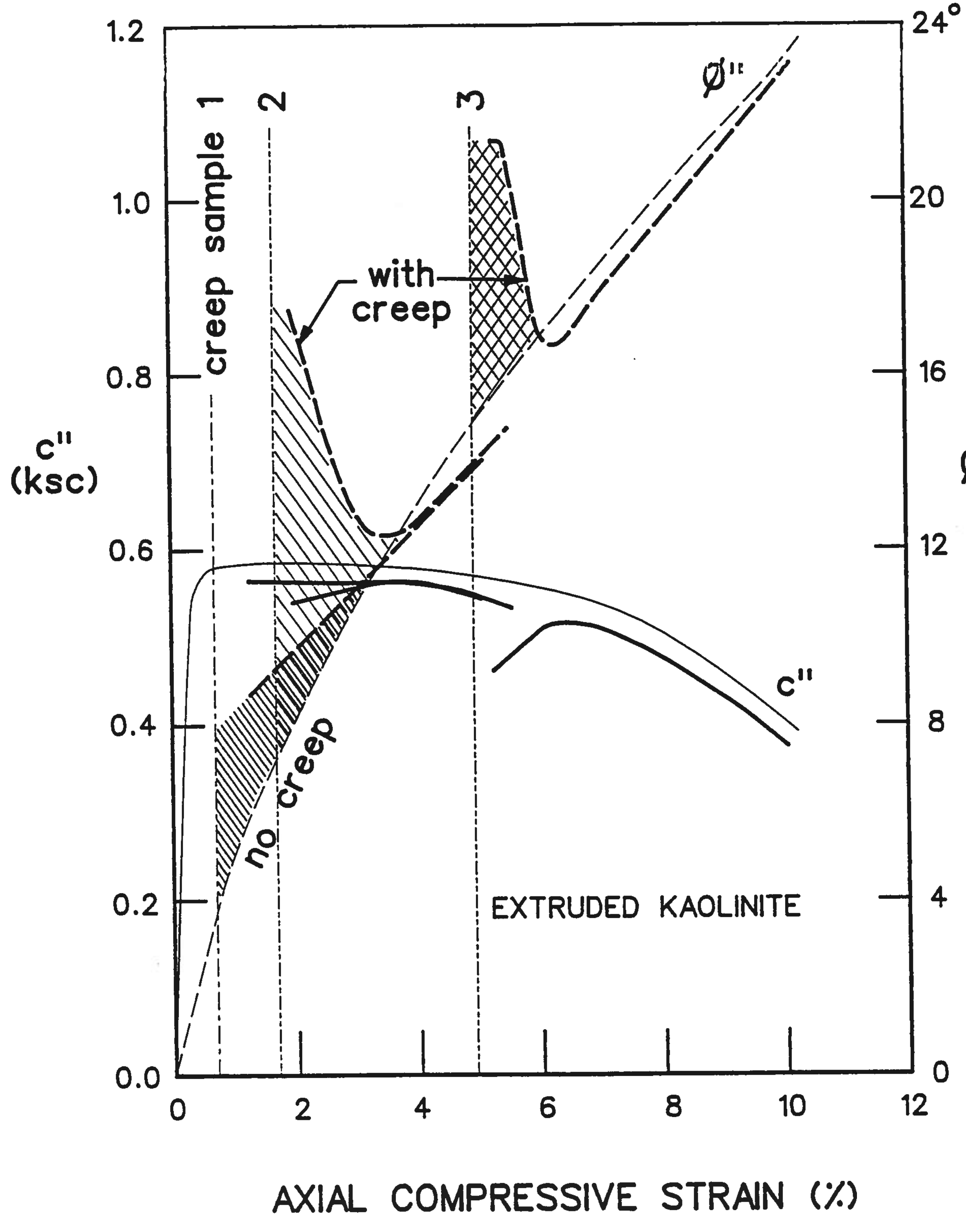

FIGURE 13 - FRICTION INCREASE DURING UNDRAINED CREEP

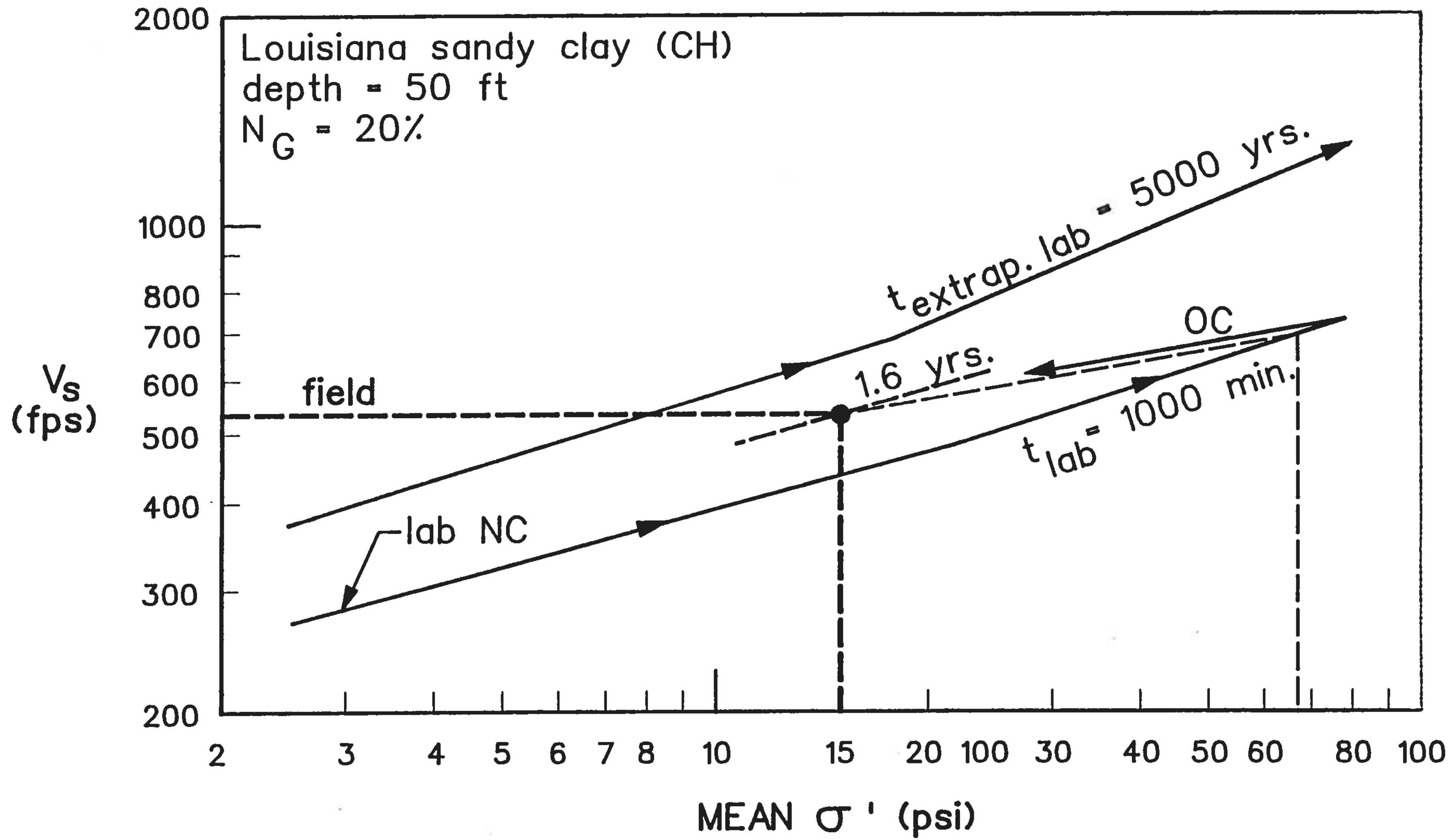

FIGURE 14 - EXAMPLE OF OVERCONSOLIDATION SHORTENING AGING TIME (Long, 1980)