ROAD SUBGRADE PROPERTIES OF LOESSAL SOIL IN THE MEMPHIS AREA

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ABSTRACT

The purpose of this paper is to present and discuss the unusually poor engineering properties of a loess. This paper contains the study and assessment of road subgrade made of a loessal soil at a project site in the Memphis area which involves about 64,750 m^2 of Portland cement concrete pavement. The pavement became damaged shortly after construction and resulted in the performance of this geoforensic investigation. The investigation conducted included determining the compactibility and pavement support of the loessal soils in question as engineered fill.

This paper discusses the scope of the investigation taken, the investigation results (which included falling weight deflectometer (FWD) data, level survey results, laboratory testing and soil bearing data), the performance characteristics of the soil, slab damage, and finally a summary and conclusions section.

INTRODUCTION

UPS Facility

As a result of the significant cracking in Portland concrete pavement at the Memphis ~~International Airport~~ a geoforensic investigation was undertaken. Much of the cracking occurred prior to any operational loads on the pavement. This paper discusses the damage investigation. First presented is the scope of the investigation followed by the pavement subbase and subgrade characteristics and then the falling weight deflectometer test results. With this data the performance of soil subgrade as it affected the Portland cement concrete pavement is evaluated.

SCOPE OF INVESTIGATION

The geotechnical investigation consisted of a review of available project documents, a canvass of the pavement damage, a level survey and falling weight deflectometer (FWD) tests on pavement areas as well as drilling and laboratory testing of the immediate soils beneath the pavement. The level survey was performed with a Zeiss NI22 level which measures to the nearest 0.1 mm. The survey was performed at control and expansion joint corners and along cracks in various pavement areas. Also, FWD tests along with ground penetrating radar (GPR) surveys were performed at selected pavement locations. A total of 112 FWD tests were performed across the site on the concrete pavement. From the radar scans the thickness of the concrete was estimated. A total of 19 borings were completed with 17 holes through the pavement and 2 outside of the pavement area and are labeled as B-1 through B-19. There were 14 other borings performed by another party which were also used in the investigation. The tests done on select soil samples consisted of modified and standard proctor tests, unsoaked and soaked California Bearing Ratio (CBR) tests, Atterberg Limits, moisture content tests, in-place soil density determinations, grain size analyses, x-ray diffraction tests and swell tests.

SUBSURFACE PROFILE

The concrete pavement thickness ranged from 10.2 to 25.4 cm. Below the concrete was a gravel subbase which consisted of a crushed limestone. The in-place gravel was dry to wet and appeared tight to loosely packed. For a total of 43 borings the subbase thickness beneath the pavement ranged from 10.2 to 71.1 cm with an average of 21.6 cm.

The subgrade immediately beneath the subbase gravel consisted of a fine-grained soil ranging from a clayey silt to a silty clay (ML to CL). Based on the results of 15 Atterberg Limit tests, the subgrade soils were found to have liquid limits (LL) which ranged from 29.1 to 42.3% and plasticity indices (PI) from 4.2 to 20.6%. Most of these soils have PI values which fall between 7 to 12% with liquid limits of 32 to 35%. The soil moisture contents were generally greater with depth and ranged from about 14 to 30%. Note, however, there was one boring where the immediate subgrade consisted of a loose 36.6 cm layer of gravel with some brown clayey silt underlain by the fine grained soil. In 3 borings the subgrade soils were found to be slightly limey to depths of 17.8 to 27.9 cm apparently due to the stabilization efforts. Also in another hole about the top 43.2 cm of the subgrade was soil cemented.

With hole depths up to 3.4 m no groundwater table was encountered. However, evidence of ponding water in the pavement subbase gravel and on the fine-grained subgrade was found in 3 borings.

LABORATORY RESULTS

CLAY MINEROLOGY

X-ray diffraction (XRD) tests were performed on the < 1 µm fraction of subgrade soils from bulk samples from 8 borings. These test results are summarized in Table 1. The XRD results indicate smectite presence in significant amounts that ranged from 26 to 52% (40% average). Smectites are important minerals in temperate regions due to its large volume change potential. This is attributed to the large ratio of breadth-to-thickness ratios in excess of 100 and specific surfaces as much as 800 m^2/g (Terzaghi et al, 1996). (*1*) Because of their large surface areas and adsorptive properties a significant volume change (shrink/swell) can occur as a result of moisture change.

A smectite sheet is similar to that of micas and vermiculites with a basic building block, shown in Figure 1, of alumina octahedron and silica tetrahedron (Schulze, 1989). (*2*) Al atoms are partially replaced by Mg in the octahedral sheet. Each replacement produces a unit negative charge at the location of the substituted atom, which is balanced by exchangeable cations, such as Ca^{+2}, in which case it is called calcium smectite, or Na^+, in which case it is called sodium smectite. Unlike the case of sodium smectite, the electrostatic attraction of the Ca cations in calcium smectite links the successive sheets together and prevents separation beyond 1 nm resulting in a smaller volume change compared to that of sodium smectite. Among clay minerals, sodium smectite has the smallest and most filmy particles (Terzaghi et al, 1996). (*1*) (Note the 10% vermiculite fraction in one area has these similar properties to smectite.) In addition, the presence of smectite in fine grained soils lowers the strength and greatly exacerbates changes in the soils strength with moisture.

Because of the amount of water absorbed by the smectite particles the overall appearance of the soil as being moist and probably near optimum moisture content camouflages the actual much higher water content of the soil. These silty soils (ML) were classified as moist with in-place moisture contents up to 29%.

Both sodium and sometimes calcium smectite were detected in the XRD tests. The test, however, cannot determine the percentage of each when both sodium and calcium smectite are present in the sample. The calcium smectite may have resulted from cation exchange during construction with the introduction of lime (CaO) in an attempt to stabilize the soil or it could have been present prior to construction. The presence of lime was indicated by the effervescence of some of the soils when tested with acid. The introduction or presence of CaO reduced the amount of water the smectite particles will absorb compared to sodium smectite. However, the percentage of Ca smectite appears insignificant as no difference could be discerned from the swell, compaction or CBR properties of the tested subgrade soils.

COMPACTION PROPERTIES

A total of 6 standard and 3 modified proctor compaction curves were developed for soils in about the first 0.76 m of subgrade across the 64,750 m^2 site. The compaction tests were done on the fine-grained soils with plasticity indices ranging from 8 to 18%. Both the standard and modified compaction results are summarized in Table 2.

Samples prepared for mold compaction were generally moisture conditioned from wet to dry. It should be noted that moisture conditioning required an extraordinary effort and attention because of the silty nature of the soil and the significant amount of smectite. All soils were observed to “ball up” to some degree when drying and required hand grinding in order to insure uniform drying results. Further, because of smectite's attraction to water, the soils were very slow to dry despite taking unusual drying measures.

The modified proctor tests were conducted on 3 bulk samples which had Plastic Indices of 8, 12 and 18%, respectively. These subgrade soils had maximum dry densities ranging from 1,858 to 1,904 kg/m^3 and optimum moisture contents of 12.6 to 14.3% (see Table 2).

Standard Proctors were performed on 6 bulk samples which had plasticity indices that ranged from 8 to 18%. These subgrade soils were found to have a maximum dry density ranging from 1,730-1,794 kg/m^3 with optimum water contents of from 15.3 to 17.0% for all except at one boring location where the measured optimum water content was 19.1%.

A comparison between the modified and standard curves for the same bulk samples showed that a modified compared to a standard compaction effort increased the soils' maximum dry density 6 to 8% and decreased the optimum moisture content 2.5 to 4.4%.

CBR TESTING

A total of 56 CBR tests were performed with 41 of them soaked and the remainder unsoaked. The CBR values at 0.1 penetration are reported herein.

UNSOAKED CBR PROPERTIES

There were 4 bulk samples from which a series of CBR tests were run on unsoaked samples compacted with a standard effort. This CBR data was plotted against moisture. From these results, the unsoaked CBR at optimum as well as at 95% and 98% wet and dry of optimum were determined where possible. Table 3 provides a summary of these results.

For all tests on the unsoaked samples high CBR values between 32 to 66 were obtained dry of optimum. Samples compacted near optimum moisture begin a catastrophic drop in strength and result in negligible CBR values in comparison to the dry side by the time these subgrade soils reach moisture wet of optimum at 95% standard compaction. This is demonstrated in samples for B-9 and B-16/17 which were tested across a range of moisture contents from dry to wet of optimum (see Table 3).

VOLUME CHANGE CHARACTERISTICS

In some of the references and based on the PI values, the project soils would be classified to have low to medium swell potential (Nelson and Miller, 1992). (3). This classification could be deceiving and the impact of swell could be far greater than anticipated depending on the type of structure considered and construction phase (i.e., before, during or after completion). The swell potential should not only be classified as a function of swell pressure as significant damage can still occur from developed swell strain without mobilizing large swell pressure. This is especially true for road pavements constructed on subgrades that have swelling soils. The volumetric strains due to change in moisture can result in significant change in grades. Some of these changes in grades may be detected and corrected after incurring additional expenses due to the delays and grades correction and compaction efforts. If the swelling characteristics of the subgrade are unknown, some of the changes in grades can go unnoticed resulting in less than specified and nonuniform pavement courses.

Because of the smectite content in the subgrade soils, significant swell resulted from soaking. Swell was measured on the 41 soaked CBR tests performed. A total of 15 soaked CBR tests were done with a modified effort. Based on the data collected the swell at optimum, 95 and 98% compaction, and at the peak CBR values were extrapolated. This swell data is summarized in Table 4.

All samples appeared to stabilize within the soaking period, however, the final moisture contents (as stipulated in ASTM D 1883) in the top 2.54 cm of the sample was consistently the greatest for the driest sample compacted and then soaked with values ranging from 22 to 37%. Or in other words, under the same conditions the equilibrated soil moisture after swelling decreased as the soil moisture used prior to soaking increased. This can be explained by breakdown of the soil with drying which results in the soil having greater expansive characteristics.

Also on many of the soaked samples moisture contents were taken not only in the top 2.54 cm of the sample, but in the middle and bottom 2.54 cm. These results showed that free swell did not occur because lower final moisture contents were present in the lower two-thirds of the samples. A plot of this data is in Figure 2. Restrained soil swell resulted because of restrictions of the soil to heave caused by the compaction mold and the compaction energy applied. Consequently the CBR Swell Index (SI) (which is the percent of soil heave to sample length) listed in Table 4 should not be considered the free swell potential of the soil at any dry density-moisture but an indication of swell. The free swell will be up to about 2-3 times the swell index.

As expected, the greatest swell indices are obtained on the dry side of optimum at a modified proctor compactive effort for all soils tested. However, when comparing the initial to final moistures for modified and standard soaked tests it is clear that the samples compacted by the modified procedure had comparatively lower final moisture contents (see Figure 3). In other words, the samples prepared with modified compactive effort indicate greater mold (boundary) restriction to swell. Based on the 3 bulk samples compacted with modified effort, Swell Index Values as high as 7.8% can occur at 95% compaction (see Table 4). At 98% compaction on the dry side of optimum, SI was up to 5.2% averaging 3.6% for the three samples. Even at optimum moisture, an SI value up to 3.0% (1.8% ave.) was achieved.

For the standard proctor tests on the dry side of optimum SI values were obtained between 2.4 to 4.5% at 95% compaction. Even at the soaked peak CBR there was swell with SI values measured between 0.6 and 1.4% with an average of 1.0%. As shown in Table 4, the swell limit is higher than the moisture content for 95% compaction wet of optimum (Standard and Modified) for all the subgrade soils tested.

Without specifically testing for swell, the swell potential of the subgrade soils could remain unsuspected because of the low range of plasticity characteristics. To determine the amount of free swell and associated swell pressure, the restrictions to swell were minimized by only preparing a 3.8 cm thick compacted sample. The subgrade soil was compacted to about 1,826 kg/m^3 dry density at 8% moisture. The resulting swell pressures and strains are given in Figure 2 and were very severe at up to 22% and 206.7 kPa, respectively. Note the plasticity index of the tested smectite loess was only 11%.

Because these smectite soils have the potential to swell, they conversely can shrink if the soil moisture reduces. Note no shrinkage tests were performed.

SOAKED CBR RESULTS

After soaking, CBR testing showed that there was an egregious reduction in strength in the soil as a result of swelling and softening. This can be seen by comparing unsoaked to soaked CBR values for the same subgrade soils. Unsoaked and soaked (standard) CBR curves for B-9 and B-16/17 versus moisture are superimposed in Figures 3 and 4 respectively. As can be seen in these figures, soaking causes the strength to reduce to 3% of the compacted unsoaked strength on the dry side of optimum at 95% compaction. Even when compacted dry of optimum to 98% of maximum dry density the subgrade soil is extremely sensitive to soaking with strength reductions to 5 and 14% of the unsoaked value. Unsoaked and soaked data from the other borings tested indicate subgrade soils with similar moisture sensitivities throughout.

During construction the strength reduction will be even more severe than observed in the lab as the swelling will not be restricted by a compaction mold. For example, if the subgrade soil were compacted dry of optimum and the subgrade soil absorbed water from rain it would swell resulting in an increase in moisture content accompanied by a decrease in density. If a swell of only 3% were realized the dry density could reduce from 98% to 95% compaction and the soil would now be wet of optimum. Unsoaked CBR data from B-9 and B-16/17 indicate that in the field the subgrade soils sensitivity to moisture is egregious with reductions in strength to less than 1% and 5% of the strength prior to swelling and softening when compacted at 98% dry to 95% wet of optimum (see Table 3). Possibly even more dramatic is the drop in unsoaked CBR with maximum dry density. A drop in maximum dry density of less than 16 kg/m^3 going from dry to wet of optimum resulted in a reduction to 2 and 8% of the initial CBR for subgrade soils at B-9 and B-6 respectively. For the B-16/17 soil the CBR similarly decreased to 7% of the CBR value for a dry density only 46.5 kg/m^3 lower.

In Table 5 all the peak and optimum soaked CBR values are summarized as well as the minimums at various levels of compaction. The minimum CBR can be on either side of optimum. Sample plots of the soaked CBR values versus initial moisture contents for all the subgrade soils tested are given in Figures 5 and 6. Also shown on these plots are the 95% and 98% compaction limits. As can be seen from Figures 5 and 6 the resulting soaked CBR values are extremely moisture dependent and are typical of the other soil samples tested. The peaks for all the smectite loess tested ranged from 6.9 to 17.5 (15 ave.) and 7.2 to 76.0 (33 ave.) for standard and modified proctor tests, respectively. The peaks of soaked CBR were found to be from 0 to 1.5% dry of optimum water content except for two samples (i.e. B-2 and B-16/17, modified) where the peaks were 0.9 to 2.2% wet of optimum.

As can be seen in Figures 5 and 6 the CBR values decrease dramatically on both the dry and wet sides of the peak. For samples compacted using a modified effort the soaked CBR values at 98% compaction were 1.2, 1.9 and 5.4 averaging 2.8.

For all the subgrade soils compacted with a standard effort, the minimum soaked CBR values found for 98% compaction were on the wet side of optimum and ranged from 1.4 to 5.0 and averaged 2.8 as did the modified results. At 95% compaction, the minimum soaked CBR decreased further and could exist on either side of optimum with values from 0.8 to 1.5 and averaging 1.2. The reported CBR values agree with Knight, 1961 finding that for most fine-grained soils that are compacted wet of optimum the CBR values will be less than 6 or 7.

FWD (FALLING WEIGHT DEFLECTOMETER) TEST RESULTS

From the FWD deflection data the combined dynamic subbase/subgrade reaction modulus k_{dyn} was backcalculated. The backcalculated k_{dyn} is based on the Westegaard's model for a vertical loading on an infinite elastic layer on elastic springs (i.e., the layer made of concrete with springs representing the soil). Based on previously reported empirical correlations the k_{stat} values for the concrete pavement areas are derived as one-half the dynamic values and ranged from 1.3 to 4.7 kg/cm^3 with an average of 2.6 kg/cm^3. A histogram of this data is plotted on Figure 7. The standard deviation of these k_{stat} values is 0.61 kg/cm^3. Below average k_{stat} values exist throughout the site.

It is important to note that the modulus of subgrade reaction for the soil subgrade, k_s, will be less than the combined subbase-subgrade value. In order to estimate k_s, Figure 8 was used. Based on this figure the subgrade reaction modulus for all the FWD tests on the concrete pavement ranges from 0.58 to 3.04 kg/cm^3 with an average of 1.52 kg/cm^3 when considering the average subbase thickness from drilling records of 21.6 cm.

The k_s data with the associated subgrade soil properties from adjacent borings have been summarized in Table 6. A review of this information indicates that the in-situ reaction modulus appears to generally increase with the in-place dry density and a decrease of moisture. The more plastic soil (i.e., plasticity index of 18-20%) had a different relationship and higher k_s values.

Note for Borings B-4 and B-5, thick gravel subbase are present (i.e., 58.4 and 71.1 cm respectively) and consequently no k_s was determined. In other words, the FWD loading remained essentially in the subbase and the k_{stat} values for the gravel recorded at these hole locations are 1.96 and 2.74 kg/cm^3 for Borings B-4 and B-5, respectively. Because the gravel subbase is expected to have a reaction modulus on the order of 11.07 kg/cm^3, loss of support appears to have resulted as the actual k_{stat} is only 20% of this value. The loss of support appears to indicate soil heave and softening that occurred after the pavement was placed. Further evidence of this is the loose packing of the crushed limestone, the wet stone and subgrade conditions, as well as standing water on the subgrade.

In general, the pattern of k_{stat} and k_s shows a decrease in value in the direction of surface drainage or increased subgrade moisture. The sub-pavement drainage can be to a localized low in the soil subgrade surface (e.g., due to undercutting or grading) or following the overall site grades for the subgrade. Ponded water on the subgrade was found in several borings.

SOIL PERFORMANCE CHARACTERISTICS

CONSTRUCTION

The use of these smectite loess soils as engineered fill is practically impossible. The soil characteristics described above showed that in the normally specified ranges of compaction the soaked CBR value is very poor averaging 1.2 to 2.8 with no difference found if the soil was compacted to 98% standard or modified. Even unsoaked CBR values wet of optimum were found to be comparatively low.

The only recourse is to essentially compact this smectite soil at contents at or close to the CBR peak in order to provide sufficient stability and this might not be practical especially in areas of large rainfall. The complexity of moisture spatial distribution and moisture movement pattern, which is well documented by Dempsey and Elzeftawy (1976) (4) makes this recourse harder to achieve for such types of soils. One practical solution is to stabilize the soil using lime or cement. However, special attention is needed when stabilizing soils with sodium smectite because of its large cation exchange capacity which may require higher than the typically used 3 to 5% of stabilizing agent. This could cause a major increase in the cost of the project. Undercutting is one other practical solution.

Thompson (1996) (5) found from his analyses of equipment sinkage and paving material compaction operations that a minimum in-situ CBR of 6-8 is required. According to Thompson (1996) (5)this requirement should insure a) enough subgrade stability under heavy repeated loading of construction traffic and operations, b) future success of placement and compaction of overlying layers, and c) the longterm performance of the pavement subgrade. Also this should prevent pumping and loss of support problem during construction. Even when compacted on the dry side at high densities and then protected by subbase stone, the exposure of this soil to moisture will cause tremendous subgrade swelling and softening problems either during as well as after construction. In other words, this soil compacted dry of optimum at high densities will appear sufficiently stable and pass proof-rolling, but become egregiously unstable with construction traffic and moisture.

ROAD SUPPORT

As discussed above, because of the nature of this soil one is forced to compact at or dry of optimum at high densities. This would be the condition if the pavement was placed with the subgrade had not degraded and was left uncorrected. This, however, presents post-construction swelling and softening problems for road support due to the much higher potential of absorbing water. Based on the laboratory and field data collected, the k_s for the subgrade soils consisting of clayey silt to clay and silt (ML to CL) can go as low as about 0.55 kg/cm^3 as these soils gain moisture and reach their ultimate state. For the higher plasticity clay and silt to silty clay (CL) with plasticity indices of 18-21%, k_s should not be less than 0.55-0.83 kg/cm^3.

Such low subgrade reaction moduli as reported above are indicative of the low CBR values obtained for these soils in the lab. A CBR of 2 was obtained for all soils despite having dry densities on the order of 1,682 to 1,746 kg/m^3 for both modified and standard compaction efforts. For the clayey silt to clay and silt, a CBR of 2 was obtained when the soil moisture content reached 18 to 21%, while for the higher plastic soil the threshold moisture was 21%. Where additional moisture was added to the soil even lower soaked CBR values were obtained to below 1 under both modified and standard proctor conditions. Also added moisture was required to reach the estimated CBR swell limit at 22 to 26%. It should be noted, however, that although some slight swell may still occur at higher moisture contents it will have a nominal effect as the soil has significantly softened and will "buffer" or "absorb" any resulting volume change against the pavement structure. The low strength and stiffness characteristics as well as the significant swell potential of these soils are indicative of the soil's significant amount of smectite.

Because the subgrade at the site has not reached its ultimate state as of yet, areas which are still significantly compacted and at about optimum moisture or below have significantly higher k_s values than in the softened state. From the FWD tests, combined subbase/subgrades moduli measured 3.82 kg/cm^3 at 2 standard deviations which results in a k_s of 2.49 kg/cm^3 assuming the design subbase thickness of 15.24 cm. Also in areas which were significantly compacted dry of optimum moisture (and have not moistened) the free swell can as much as 22% with swell pressures of 206.7 kPa for nominal strains (see Figure 2).

At optimum, a k_s of 1.94 kg/cm^3 appears reasonable but once the subgrade is exposed to water and swelling begins, free swells of 2-9% are possible until it completely softens equaling 2.08 to 8.33 cm of free heave per meter of compacted soil. Note, however, the actual subgrade heave will of course be less and depends on the pavement and subgrade depth. Nonetheless, this subgrade heave will cause loss of support of the pavement, as discussed above where k_{stat} was only about 20% of what would be expected in the gravel subbase

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Because subgrade areas exist which are still compact and relatively dry, the swelling and softening process is expected to continue at indeterminate rates and locations that are primarily a function of the exposure to water and soil moisture spatial distribution and movement. The amount and timing of swelling and softening will mainly depend on: exposure to water though crack and joint openings as well as sub-pavement drainage, traffic loading, the existing soil conditions, and freeze-thaw action which could possibly further lower k_s. Subgrade moduli as stiff as 2.77 kg/cm^3 can exist in areas which remain dry.

Also because of the volume change potential of the subgrade a reduction in the moisture content of this fine-grained soil can result in shrinkage or subgrade settlement. The most susceptible locations where this could potentially result, of course, would be along exposed edges of the concrete pavement. Soil shrinkage would result if the adjacent unpaved ground dries enough to draw a significant amount of moisture from the subgrade soils. A shrinkage or settlement of several percent may be possible.

PAVEMENT DAMAGE

The pavement damage occurred in designed 15.2 cm concrete. The cracking throughout this 64,750 m^2 site was from hairline to about 0.64 cm wide and hard vertical offsets ranging from none to about 0.64 cm. The cracks formed in more or less linear patterns. One predominantly longitudinal crack extended for a distance of about 137 m.

To assist in determining the pavement subgrade behavior a level survey was performed over various sections of the pavement. Elevations were measured to 0.1 mm at intersections of concrete contraction joints and expansion joints and contraction/expansion joints with asphalt as well as at joint and crack intersections. Elevation profiles were then drawn at damaged slab areas and superimposed onto a profile for the most nearby pavement section which did not show any damage. Based on these plots both heave and settlement up to 2.29 cm and 3.81 cm respectively have occurred. On Figure 9 are four sample profiles. As mentioned earlier, the profiles are drawn from surveyed points at contraction joints and visible cracks. Note, there are a number of remarks made on these profiles where contraction joints probably exist. These cracks are not apparent because they are probably concealed because of the flexible epoxy fillings in the joints.

SUMMARY AND CONCLUSIONS

The project soil discussed in this paper is a loess from the Memphis area. The loess was used as the immediate subgrade support for about 64,750 m^2 of pavement. Upon visual examination, the subgrade soil appeared to be moist and a typical loess soil with significant amounts of silt to be classified as mainly a Clayey Silt or ML. This seemed to be verified with soils exhibiting only a low to marginal range in plasticity with Plasticity Indices of 12% or less except for areas with a PI of 18-20%.

In the lab, however, when the above loessal soils were tested across the typically recommended and specified compaction ranges this soil was found to have extremely detrimental characteristics for pavement support and construction. In order to determine the critical behavior any subgrade soil testing of each sample should at a minimum be done at the limits of the stipulated compaction range. In addition to having low CBR values at both modified and standard Proctor compaction limits there was significant swell.

The significant difficulties during construction, loss of support, and pavement cracking validate the recommendations of Thompson (1996) (5) of a minimum CBR of 6 or 8 for successful pavement construction. The soaked CBR values for this site were always lower than this recommended range. Without testing at the compaction limits of ±95% the results can be deceiving and produce exceedingly high soaked CBR values with no evidence of the soil's swell potential. This was especially the case with regard to the loess discussed herein. From soaked CBR testing it was found that this soil had an egregious sensitivity to the compacted moisture-density where the peak CBR was up to 10.9 fold greater than at the 95% standard compaction limits (9.4 ave.)

The very bad engineering properties of these subgrade soils can be attributed to the significant smectite present in the loess. From X-ray diffraction tests the < 1 μm fraction of the loess was found to have clay consisting of an average 40% of smectite.

The use of the smectite as a subgrade soil resulted in a myriad of construction difficulties. Shortly after the pavement was placed significant damage was evident. The pavement cracking became significantly worse in less than a year's time. Differential surface elevations measured across the concrete pavement demonstrates the effects of the swelling/shrinking and softening process of the pavement subgrade which is accompanied by soil moisture change.

Classification of the swell potential of soils based on plasticity index values could be misleading. The resulting extent of damage is a function of structure type and the construction activities. In fact, for pavement construction even soil classified as low to medium swell potential could have significant impact on the final product through the change of grades and hence the thickness of different pavement courses as well as the future performance of the pavement.

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LIST OF TABLES AND FIGURES

TABLE 1 Summary Of Clay Mineralogy For Subgrade Soils

HOLE	SMECTITE, %	ILLITE, %	KAOLINITE, %	VERMICULITE, %	SODIUM SMECTITE	CALCIUM SMECTITE
B-2	26	54	20	-	YES	YES
B-9	51	36	13	-	YES	NO
B-11/12	36	40	14	10	YES	NO
B-13	37	38	25	-	YES	YES
B-14	35	37	28	-	YES	YES
B-16/17	52	29	19	-	YES	NO

Note: Clay mineral percentage based on < 1 µm fraction.
“-“ indicates negligible amounts.

TABLE 2 Summary Of Compaction Results On Subgrade Soils

HOLE	LL (%)	PI (%)	TEST	γ_d max, kg/m^3	MC opt, %	98% γ_d max, kg/m^3	95% γ_d max, kg/m^3
B-2	34.1	12.3	MOD	1896.6	13.1	1858.1	1802.1
B-2	34.1	12.3	STD	1750.8	16.2	1715.6	1662.7
B-6	33.1	7.7	STD	1770.0	16.8	1734.8	1681.9
B-9	33.5	12.2	STD	1760.4	15.2	1725.2	1672.3
B-11/12	32.4	8.1	MOD	1858.1	14.3	1821.3	-
B-11/12	32.4	8.1	STD	1726.8	16.8	1691.5	1640.3
B-13	35.3	11.0	STD	1730.0	19.1	1694.8	1643.5
B-16/17	39.8	18.2	MOD	1907.6	12.6	1866.2	-
B-16/17	39.8	18.2	STD	1794.1	17.0	1758.8	1704.4

DEFINITIONS:

LL = Liquid Limit
PI = Plasticity Index
STD = Standard Proctor Test (ASTM D698)
MOD = Modified Proctor Test (ASTM D1557)
γ_d max = Maximum dry density
MC opt = Optimum moisture content

TABLE 3 Summary Of CBR Results On Unsoaked Subgrade Soils

HOLE	TEST	DRY SIDE				OPTIMUM	
		95% γ_d max		98% γ_d max		γ_d max	
		CBR	MC,%	CBR	MC,%	CBR	MC,%
B-2	STD	34.0	11.5	32.0	14.0	30.0	16.2
B-6	STD	ND	ND	38	13.9	ND	ND
B-9	STD	42.0	10.5	66.0	12.0	31.0	15.2
B-16/17	STD	54.0	11.3	39.3	13.8	20.0	17.0

HOLE	TEST	WET SIDE			
		98% γ_d max		95% γ_d max	
		CBR	MC,%	CBR	MC,%
B-2	STD	ND	ND	ND	ND
B-6	STD	8	19.0	ND	ND
B-9	STD	6.5	17.7	0	19.4
B-16/17	STD	8.8	18.8	2.0	20.6

DEFINITIONS:

STD = Standard Proctor
MOD = Modified Proctor
MC = Moisture Content
γ_d = Dry Density
CBR = California Bearing Ratio
ND = No Data

TABLE 4 CBR Swell Indices For Soaked Subgrade Soils

HOLE	TEST	DRY SIDE				PEAK CBR	
		95% γ_d max		98% γ_d max		SI, %	MC, %
		SI, %	MC, %	SI, %	MC, %		
B-2	MOD	~5.1	9.0	3.4	10.6	1.3	14.0
B-2	STD	2.4	11.5	1.6	14.0	1.4	14.9
B-6	STD	2.4	11.0	1.2	13.9	0.6	16.5
B-9	STD	4.5	10.5	1.3	12.0	0.8	14.9
B-11/12	MOD	ND	ND	2.2	11.2	0.8	13.8
B-11/12	STD	ND	ND	2.0	13.3	0.8	16.8
B-13	STD	3.9	14.5	1.7	17.2	1.4	18.2
B16/17	MOD	7.8	7.6	5.2	10.0	1.3	14.8
B-16/17	STD	4.4	11.3	2.4	13.8	1.0	16.1

HOLE	TEST	OPTIMUM		WET SIDE			
		SI, %	MC, %	98% γ_d max		95% γ_d max	
				SI, %	MC, %	SI, %	MC, %
B-2	MOD	1.6	13.1	0.8	15.3	0.4	16.8
B-2	STD	1.0	16.2	0.5	18.5	0.4	20.0
B-6	STD	0.6	16.8	0.4	19.0	0.3	21.0
B-9	STD	0.7	15.2	0.4	17.7	0.1	19.4
B-11/12	MOD	0.7	14.3	0.3	16.2	0.1	18.0
B-11/12	STD	0.8	16.8	0.4	19.3	ND	ND
B-13	STD	1.1	19.1	0.7	20.6	0.5	21.7
B16/17	MOD	3.0	12.6	1.2	15.0	0.6	17.2
B-16/17	STD	0.8	17.0	0.5	18.8	0.3	20.6

DEFINITIONS:

STD = Standard Proctor
MOD = Modified Proctor
SI = Swell Index
γ_d = Dry density
MC = Compaction water content
ND = No Data

TABLE 5 Summary Of CBR Results On Soaked Subgrade Soils

HOLE	TEST	PEAK CBR		100% γ_d max		98% γ_d max		95% γ_d max	
		CBR	MC,%	CBR	MC,%	CBR min	MC,%	CBR min	MC,%
B-2	MOD	7.2	14.0	5.0	13.1	1.9	10.6	1.6	~9.0
B-2	STD	6.9	14.9	3.6	16.2	1.4	18.5	0.8	20.0
B-6	STD	14.8	16.5	12.5	16.8	1.9	13.9	1.5	11.0
B-9	STD	14.7	15.0	9.5	15.2	2.4	17.7	1.4	10.5 19.4
B-11/12	MOD	76.0	13.8	42.0	14.3	5.4	16.2	2.2	18.0
B-11/12	STD	17.5	16.8	17.5	16.8	3.0	19.3	ND	ND
B-13	STD	5.8	18.2	5.0	19.1	2.9	20.6	0.8	14.5
B16/17	MOD	16.0	14.8	3.8	12.6	1.2	10.0	0.9	7.6
B-16/17	STD	15.3	16.1	9.3	17.0	5.0	18.8	1.4	11.3

DEFINITIONS:

MOD = Modified test
STD = Standard test
MC = Compaction moisture content
CBR = Minimum soaked CBR either dry or wet of optimum
γ_d max = Maximum dry density
ND = No Data

TABLE 6 Soil Moduli Of Subgrade Reaction And Associated Properties

HOLE/ FWD NO.[1]	SUBGRADE SOIL	k_{stat} kg/cm^3	T_c cm	T_{sub} cm	γ_d[2] kg/m^3	STANDARD COMPACTION, %	MC %[2]	k_s kg/cm^3
B-1 PT 29	silt & clay	3.21	25.30	23.47 (17.8 sli limey)	1795.0 1819.5	101.4 102.8	16.1 16.7	1.88
B-3 PT1-1 (2.1 m E)	clay & silt	1.49	16.46	30.18	----		19.5 19.6	0.55
B-4 PT1-2 (1.8 m E)	silt & clay	1.97	14.63	58.52	1634.8 1719.1	92.3 97.1	21.0 19.4	----
B-5 PT1-4 (0.6 m W)	silt & clay	2.74	18.29	71.93	1605.2 1627.1	90.7 91.9	22.9 22.4	----
B-6 PT2-5	silt & clay	1.30	10.16	24.89	1646.8 1549.3	93.0 87.5	21.8 25.9	0.58
B-7 PT3-2	silt & clay	1.77	14.02	17.98 (43.2 sli limey)	ND	----	21.3	0.83
B-8 PT12-1 (0.6 m N)	silt & clay	1.83	15.85	23.77 (43.2 soil cemented)	ND	----	17.6	0.89
B-10 PT7-1 (2.4 m E)	silt & clay	3.21	15.85	17.68	ND	----	16.7	2.27
B-11 PT7-2 (0.9 m E)	silt & clay	2.49	13.41	18.60 (27.9 sli limey)	1461.4 1513.1	84.6 87.6	21.3 26.8	1.55
B-12 PT7-4 (0.6 m NW)	silt & clay	1.36	16.76	10.80	1620.3 1530.1	93.8 89.6	22.7 27.6	0.50
B-13 PT4-2	clay & silt	1.85	22.56	23.16	1712.5 1691.4	99.0 97.8	17.4 21.8	0.94
B-14 PT25	clayey silt	1.63	24.69	25.00	1568.7 1477.1	88.6 83.4	20.8 20.7	0.75
B-15 PT 46	clay & silt	1.44	21.59	17.78	1572.5 1640.1	90.9 94.8	20.8	0.83
B-16 PT17-1 (0.6 m W)	clay & silt	2.27	16.46	17.78	1555.2	86.7	24.9	1.47
B-17 PT17-2 (1.5 m E)	clay & silt	2.35	17.98	15.54	1672.6 1618.0	93.2 90.2	20.9 23.0	1.94
B-18 PT17-4 (1.2 m N)	clay & silt	ND	13.11	15.88	ND	----	24.5	----
B-19 PT16-2	clay & silt to silty clay	2.24	12.70	35.05	1699.0 1738.8	94.7 96.9	19.2 18.8	~0.83

(TABLE 6)

DEFINITIONS:

k_{stat} = Modulus of subgrade reaction at top of subbase
T_c = Cored thickness of concrete
T_{sub} = Drilled thickness of subbase ground
γ_d = In-place dry density of soil within top foot of subgrade surface
mc = In-place moisture content of soil associated with dry density values
k_s = Estimated modulus of subgrade reaction for soil subgrade

[1]All FWD tests are within about 30 cm of the borehole unless otherwise noted.
[2]Measurements within 30 cm of the top of subgrade except for the second density and moisture values of B-5 which is to a depth of 39.6 cm.

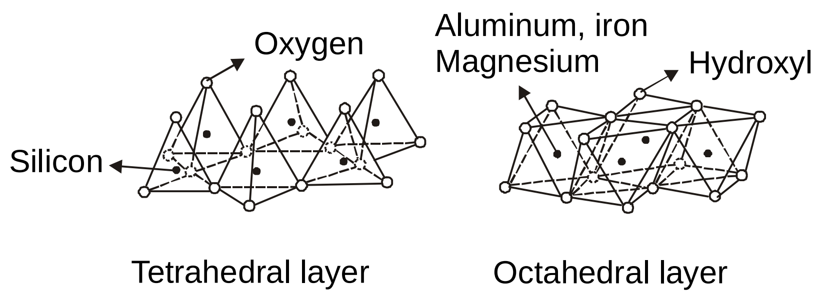

FIGURE 1 Building Blocks Of Clay Minerals (Grim, 1968)

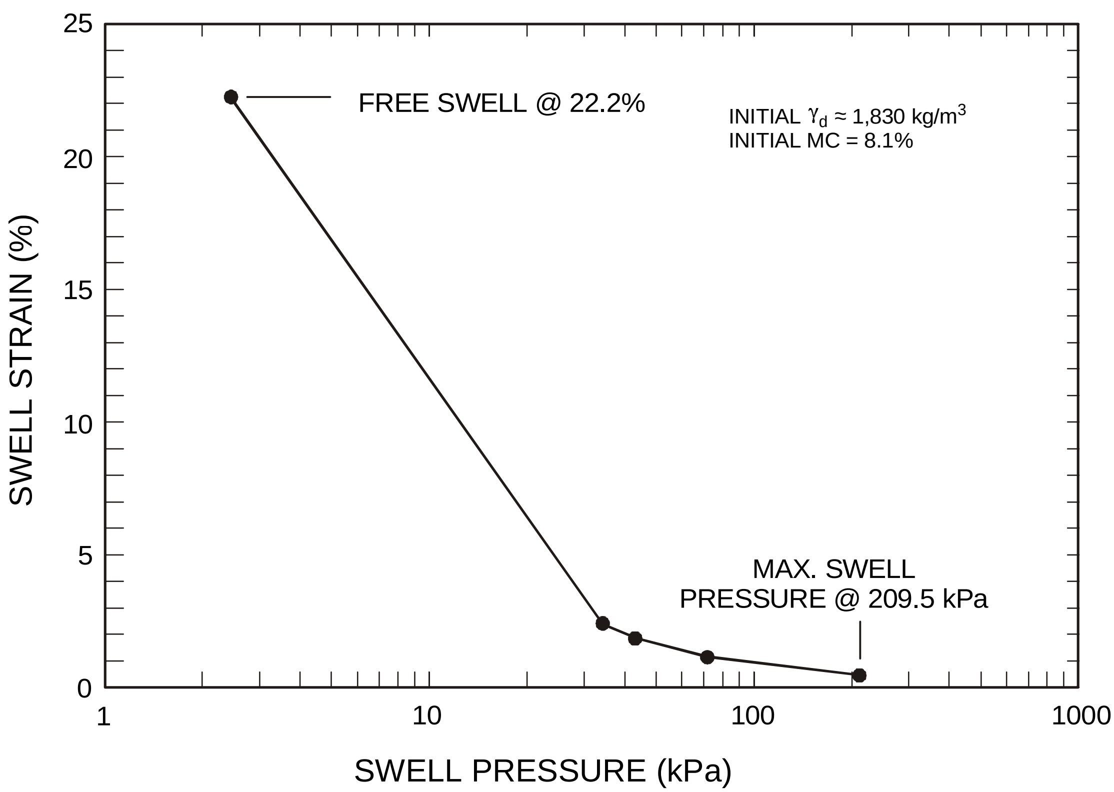

FIGURE 2 Swell Strain Versus Swell Pressure For Subgrade Soil With A PI = 11%

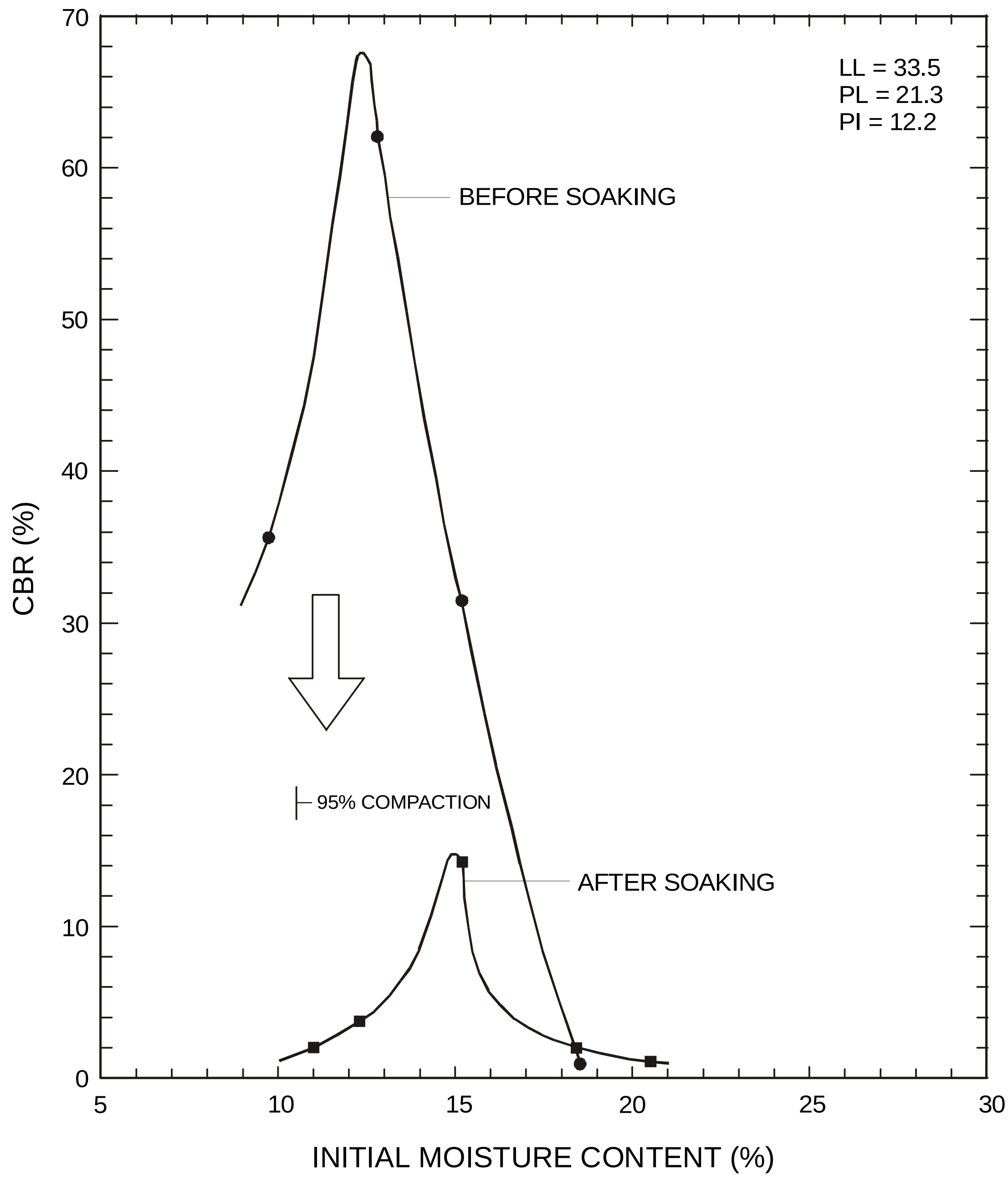

FIGURE 3 Unsoaked And Soaked CBR Values Versus Moisture Content At Standard Compaction For Subgrade Soil At B-9

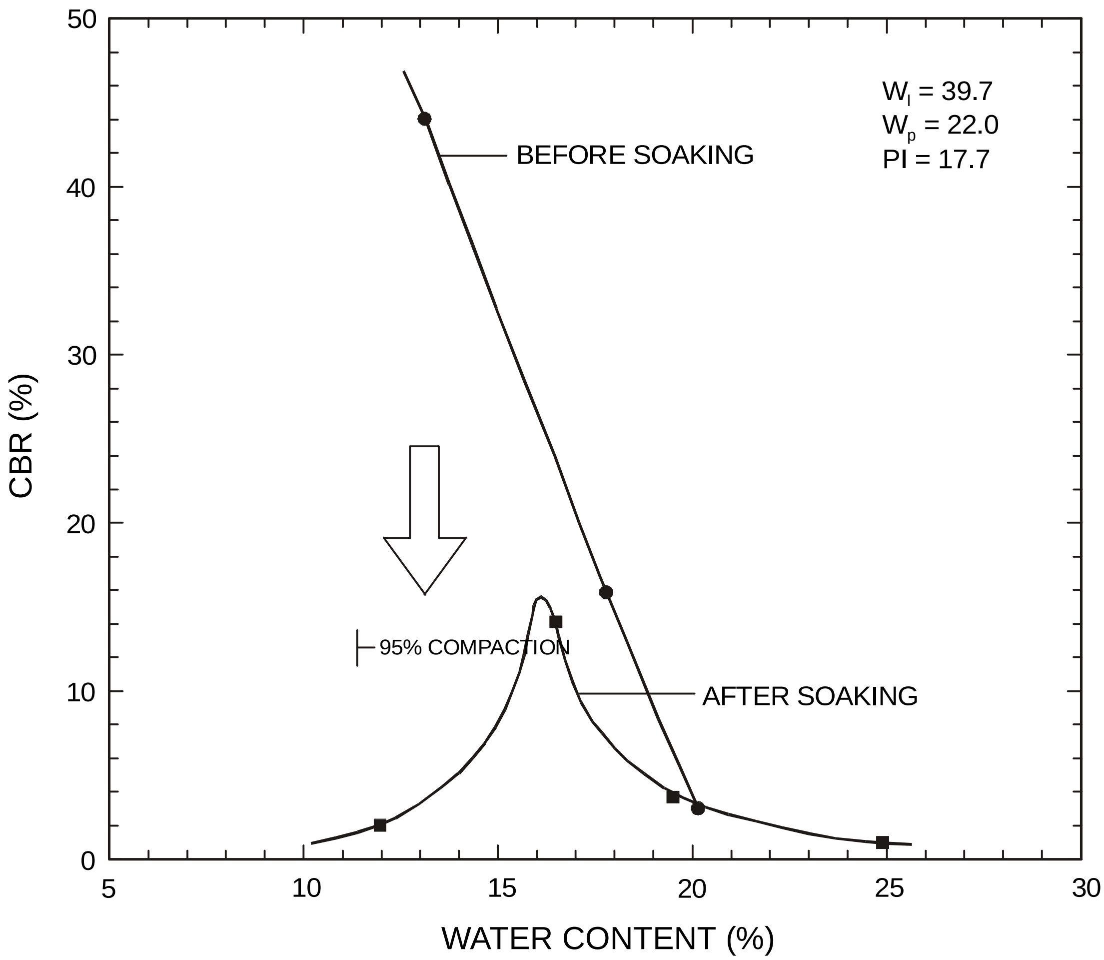

FIGURE 4 Unsoaked And Soaked CBR Values Versus Moisture Content At Standard Compaction For Subgrade Soil At B-16/17

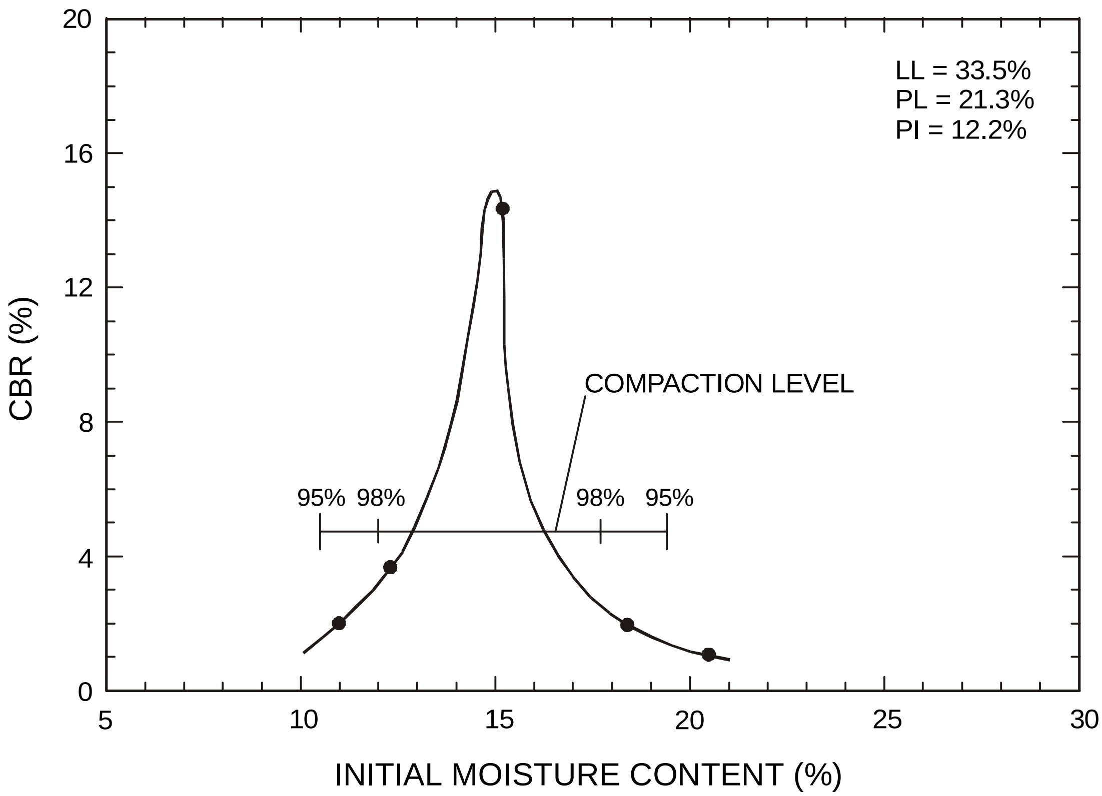

FIGURE 5 Soaked CBR Versus Initial Moisture Content For Standard Compaction Of Subgrade Soil At B-9

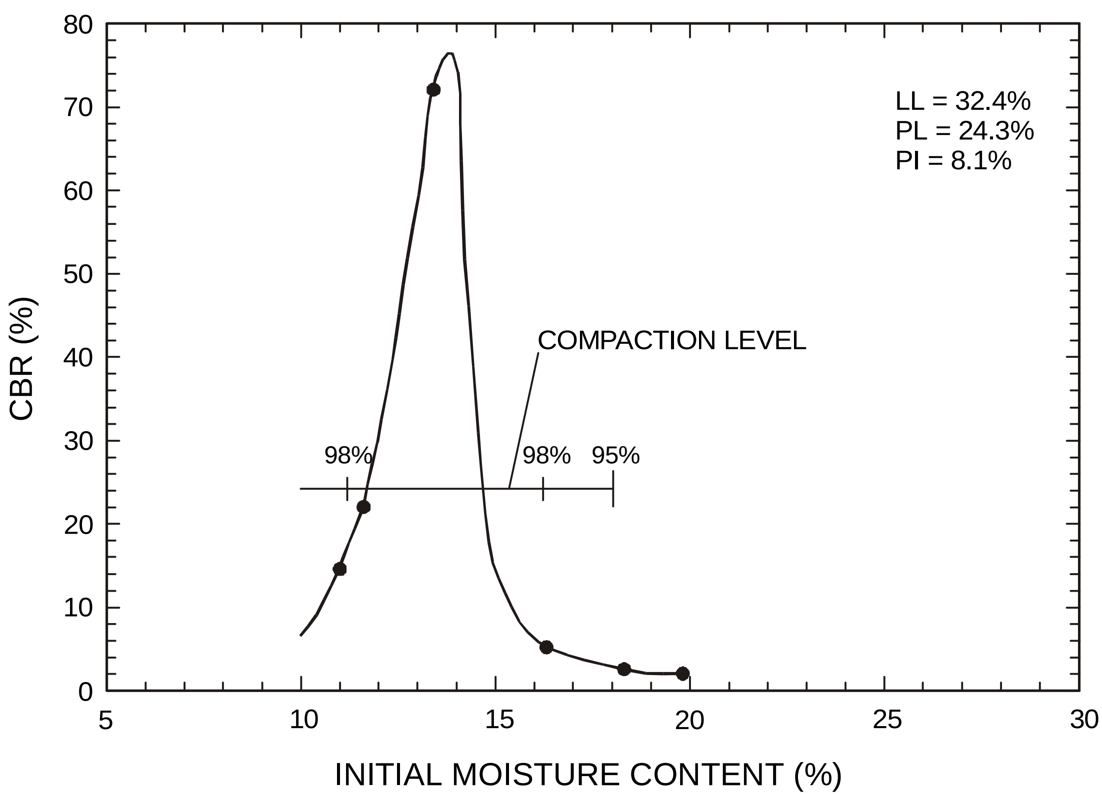

FIGURE 6 Soaked CBR Versus Initial Moisture Content For Modified Compaction Of Subgrade Soil At B-11/12

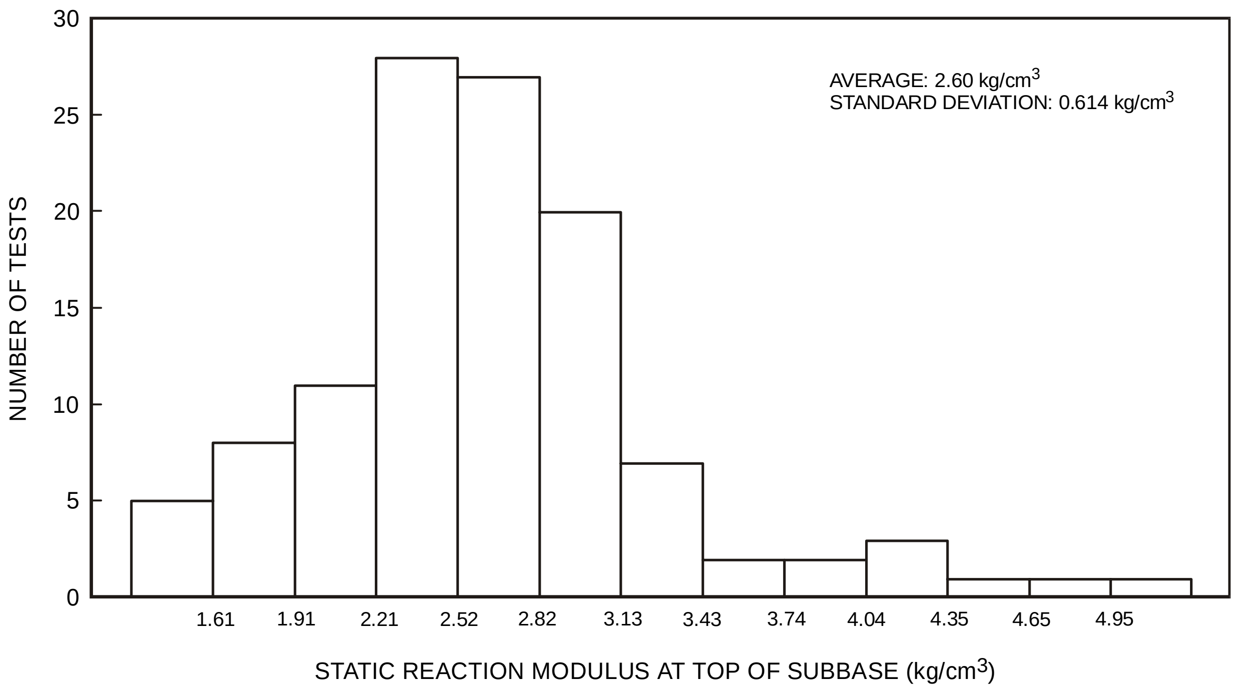

FIGURE 7 Histogram Of Subbase Reaction Moduli Backcalculated From FWD Tests

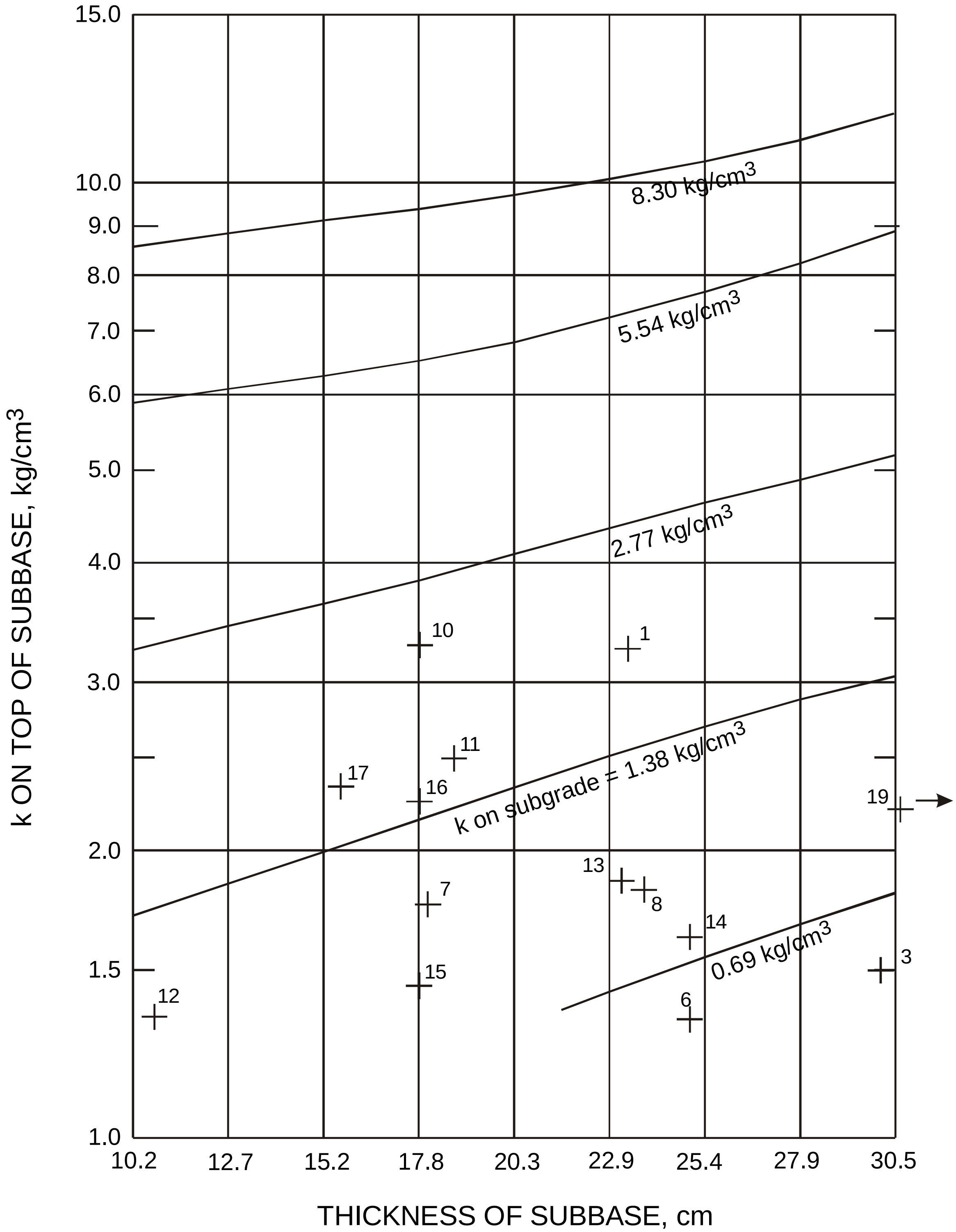

FIGURE 8 Effect Of Granular Subbase Thickness On K-Value (Modified From AASHTO Pavement Design Graph)

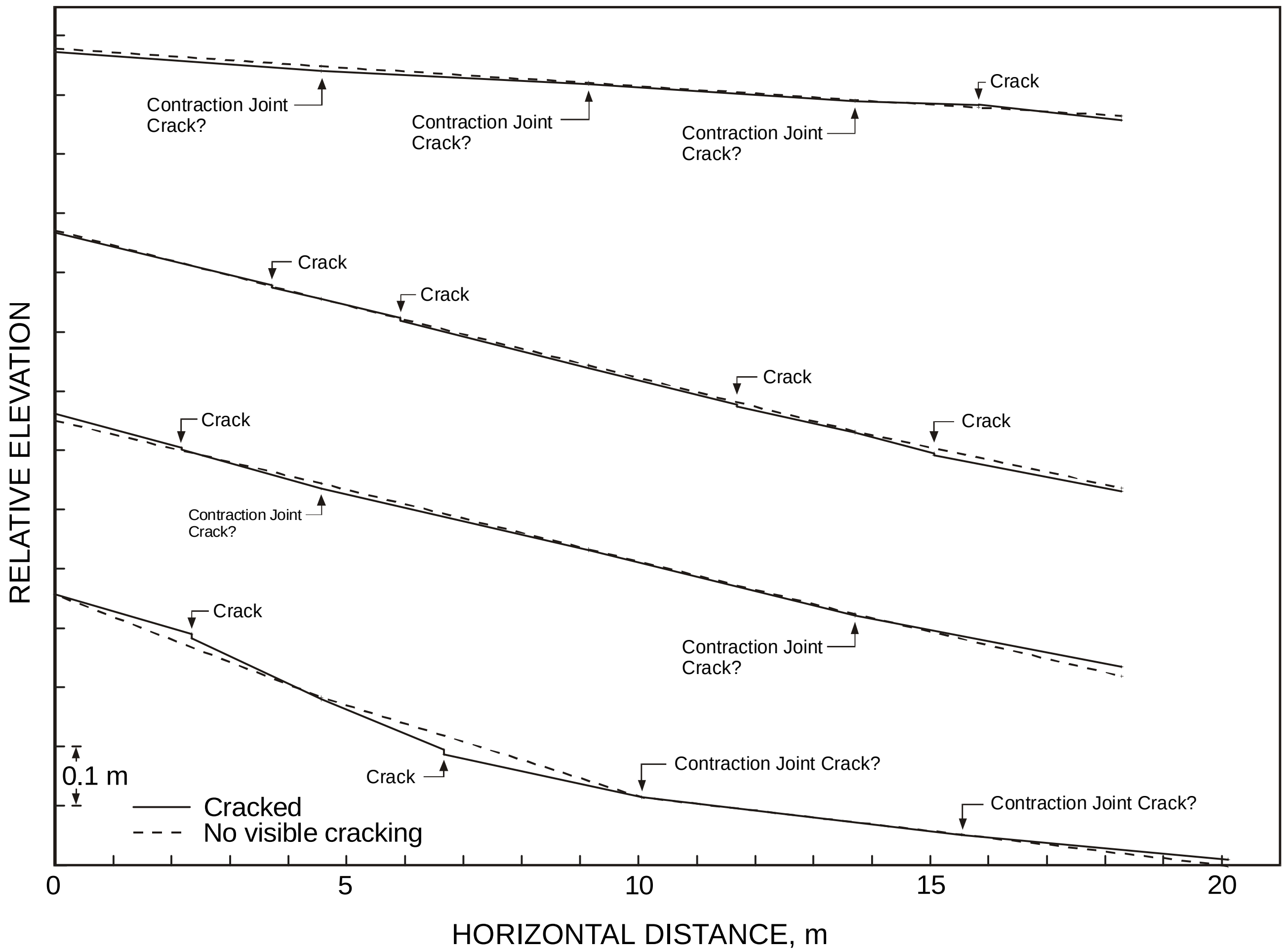

FIGURE 9 Comparison Of Elevation Profiles Across Damaged And Nearby Undamaged Concrete Pavement Sections