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JEFFERSON FREEWAY INVESTIGATION (WESTBOUND LANES)

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SUMMARY

On October 7, 1985, a Caterpillar Model 992C front-end loader was operated over paved portions, including two bridges, of the Jefferson Freeway in Jefferson County. This section of the freeway had been opened recently to traffic. The empty gross weight of that vehicle is 207,578 pounds -- 120,395 pounds front axleload and 87,183 pounds rear axleload. Section 105.13 of the Standard Specifications for Road and Bridge Construction designates that hauling in conjunction with construction of a project be performed so as not to violate limitations provided by law or regulation. Axleloads allowed by statute are limited to 20,000 pounds; gross vehicular weights are limited to 80,000 pounds. The axleloads and gross weight of the Model 992C exceed those permitted by law. The front-end loader traveled on steel cleats, also not allowed under the statutes. No permit to move the loader over the highway was obtained; and the manner in which it was moved (on cleats) was not permissible even under permit.

The Kentucky Transportation Reseach Program (KTRP) was retained to perform an investigation for the Kentucky Department of Highways to assess damage that may be attributed to passage of the Model 992C over portions of the Jefferson Freeway. KTRP investigators conducted visual surveys, performed Road Rater deflection tests, did in-place California Bearing Ratio (CBR) testing on subgrade materials, obtained pavement cores, and conducted laboratory tests and analyses.

It appeared the pavement was damaged by the front-end loader. No evidence of damage to the bridges attributable to the front-end loader was detected. Damage to the pavement attributable to passage of the front-end loader was assessed in the amount of $322,074.50.

It is important to note that compressive strengths of the portland cement concrete, as determined from core specimens, were considerably higher than would normally be expected for two-year old concrete that was specified to meet a 28-day compressive strength of 3,500 pounds per square inch (psi). Division of Materials' records for the project indicate the average 28-day compressive strength was 6,130 psi. It is almost certain that damage caused by the front-end loader would have been considerably more extensive if the portland cement concrete placed on the project only met or slightly exceeded specification requirements.

Visual observations of the relevant sections of the Jefferson Freeway did reveal physical damage to joints in the pavement. This damage could be associated with the passage of the front-end loader because of markings of the pavement surface by the cleats of the vehicle. This damage should be repaired to restore the joints to an as-constructed condition so as not to accelerate and aggravate damage under future traffic loadings and environmental conditions.

Both field and laboratory testing of the subgrade and cores from the portland cement concrete slabs indicate similar characteristics of the materials. Properties back-calculated from surface deflections (obtained with a Road Rater) also verify results of field and laboratory testing and evaluations.

Analyses do show that the loading due to the passage of the front-end loader "consumed" only about one percent of the fatigue (service) life of the mainline slabs. Even though this seems to be only a nominal amount, it is much more than can be attributed to a single pass of any "normal" vehicle allowed on highways. To illustrate, one pass of the front-end loader was equivalent to 35 million automobiles or approximately 53,000 legally loaded five-axle semi-trailer trucks. The single pass of the front-end loader did utilize all of the service life of the shoulder slabs. It was not possible to attribute conclusively the initiation of horizontal cracking at approximately middepth in the portland cement concrete slabs near load-transfer assemblies. However, the analysis of load-transfer efficiencies at joints did suggest the likelihood that the cracking was aggravated and propagated by this unusual loading event.

It should be noted that the cost of restoration is not based on the most expensive strategies. The recommended strategies of rehabilitation were selected on the basis of what was considered to be the most effective and practical.

INTRODUCTION

The mainline pavement on the Jefferson Freeway was constructed as 10 inches of nonreinforced portland cement concrete having slab lengths of 12, 13, 17, and 18 feet and transverse joints were sawed on a skew. Load transfer assemblies were placed at locations of sawed transverse joints. The mainline pavement was tied to 6-inch portland cement concrete shoulders using a keyed joint and tie bars. No load transfer assemblies were used between slabs on the shoulders.

On October 7, 1985, a Model 992C front-end loader, manufactured by Caterpillar, traveled directly over three sections of pavement (see Appendix A), according to District 5 engineering personnel who observed the event. The first section was the exit ramp off Preston Highway to the eastbound ramp onto the unpaved portion of the Jefferson Freeway. From there, the loader traveled west on the subgrade of the Jefferson Freeway to the beginning of the 10-inch portland cement concrete pavement between the Preston Street and I-65 interchanges. The loader proceeded across the lanes of the Jefferson Freeway to the right side of the roadway where the typical path placed the right wheels on the shoulder and the left wheels on the outside lane. While traveling on the Jefferson Freeway, the loader crossed two bridges. The loader exited the Jefferson Freeway at the northbound exit ramp to I-65 where it eventually had all four wheels on the 10-inch portland cement concrete pavement. The loader traveled north on I-65, having the right side on the subgrade and the left side on the temporary asphaltic concrete pavement. The loader exited I-65 on the eastbound exit ramp to the Outer Loop with the right wheels on an asphaltic concrete shoulder and the left wheels on the 10-inch portland cement concrete pavement until eventually all wheels contacted the concrete pavement. The loader proceeded to the Outer Loop where it crossed to the subgrade on the north side, then onto the subgrade to the railroad property where it crossed beneath I-65 and then traveled to a quarry.

A visual survey to determine surface defects was performed on October 17, 18, 21, 23, and November 12, 1985. Road Rater deflection tests were performed on October 17, 18, and 21, 1985. Concrete pavement cores were obtained and in-place CBR tests were performed on the subgrade on October 17, 18, and 21 and November 13, 14, 15, 1985. Two bridges were inspected on October 23, 1985.

The Model 992C is currently the largest front-end loader manufactured by Caterpillar. Steering is accomplished by a hinged joint in the middle of the frame in lieu of pivoting wheels. The bucket has a rated capacity of 12.5 cubic yards. The loader is mounted on four tires, each of which is surrounded by a series of connecting metal plates having a cleat on each longitudinal side of the plate as shown in Figure 1. Such steel cleats are not permitted on highways. These cleats provide a gripping action similar to that of tracks used on bulldozers, etc. According to Caterpillar, this configuration has an empty gross weight of 207,578 pounds. Allowable gross vehicular weights under the statues is 80,000 pounds. Fifty-eight percent of the load is carried by the front axle for an axleload of 120,395 pounds, or 60,198 pounds per tire. According to the statutes, axleloads are limited to 20,000 pounds. The rear axle carries 87,183 pounds, or 43,591 pounds per tire. Inspection of the loader indicated that no more than six of the 1-inch wide cleats may be in contact with a flat pavement at any instant. Scar marks on the surface of the concrete pavement typically measured 29 inches in length. The cleats are 43 inches in length. Thus, scar marks indicate that the entire length of the cleats apparently were not in contact with the pavement. Under such a loading configuration, the calculated bearing pressure for the front axle is 345 psi. The calculated bearing pressure for the rear axle is 251 psi. Typical tire inflation pressures of trucks are on the order of 70 to 80 psi. Unusually high pressures being observed recently range from 110 to 150 psi. Damage to pavements is directly related to the tire pressures -- the greater the pressure, the more damaging the load.

INSPECTIONS OF WESTBOUND BRIDGES

Two bridges (over Blue Lick Road and Freedom Way Road) were inspected on Wednesday, October 23, 1985. The inspection consisted of 1) visual observations of the bridge decks, 2) visual observations of the bridge superstructures, 3) visual observations of the bridge substructures, and 4) eddy-current nondestructive testing on portions of the Freedom Way overpass. Since the front-end loader wheel paths were evident in the right traffic and curb lanes of both bridges, the inspections focused on those portions of the bridges.

For clarity, the following notation is used: 1) the direction of traffic is east to west on each bridge; 2) the westernmost abutment is referred to as Bent 1, the adjacent pier is Pier 2 etc.; and 3) facing east, the leftmost girder is termed G1, the girder adjacent to the right of G1 is termed G2, etc.

The decks were the first elements of the bridges inspected. The cleats on the vehicle had scored the decks in a manner similar to scoring on the pavement. The loader's wheel paths were clearly identifiable. No damage other than surficial chalkiness (cleat marks) was observed.

Cracks in the top side of the deck of the Blue Lick bridge are diagrammed in Figure 2. This pattern of cracking has been observed recently to be typical of this style bridge. It has not been determined whether the cracks are directly over the beams or whether they occur in the maximum shear zone alongside the beams. The cracks were not visible on the underside of the deck. Inasmuch as the cracks are typical of the style of bridge (observed elsewhere), they are not considered to be associated with the single-load occurrence under investigation and reported herein.

The crack pattern in the deck of Freedom Way bridge is diagrammed in Figure 3. The cracks on the topside are the same as the cracks underneath. On the bottom, they have effloresced and are most obvious. No evidences of new cracks were observed (cracks without efflorescence). The cracks are considered as being unrelated to the loading in question -- but are suspected as being caused by the difference in thermal expansion of the steel and concrete.

Since both bridges were of deck-girder configuration, the substructures and superstructures were inspected concurrently. Close access to elevated portions of the bridges were facilitated by a lift-bucket truck provided by KYDOH District 5. Inspection of both bridges centered on structural elements on the right side of the structure.

The Blue Lick Road westbound overpass was inspected first. The two-lane structure is a three-span, continuous, prestressed girder bridge. Inspection of Bents 1 and 4 (abutments) revealed no signs of settlement or load-induced cracking. The neoprene bearing pads between the abutments and girders showed no evidence of crushing or undue compression. There were no signs of uneven compression between the outer bearing pads on the left and right sides of the bridge. The berm headwalls showed no signs of displacement or cracking.

Inspections of the piers revealed no signs of settlement or cracking. The exposed portions of the pier-cap faces on the right side of the bridge showed no signs of cracking.

The neoprene bearings between the girders and the pier caps exhibited no signs of distress. There were no signs of uneven compression between the outer bearing pads on the left and right sides of the bridge.

Using the lift-truck, the center span superstructure was inspected from the underlying roadway. Seven prestressed, precast concrete girders supported the deck. The three girders on the right portion of the bridge (G5, G6, and G7) were closely inspected on both web faces and on the lower flange faces. No signs of cracking were evident. The joints between the precast upper flanges of the girders and the bridge deck appeared as being tight.

Three transverse cast-in-place diaphragms interconnected the girders. Those were located at mid-span and over Piers 2 and 3. The diaphragms showed no signs of cracking. The joints between the diaphragms and most of the girders appeared tight, except at the outer (fascia) Girder G1. There a gap was observed in the web area of G7 ranging from 3/16 inch near the upper flange to 1/32 inch near the lower flange. Inspection of the joint between the mid-span diaphragm and the left outer girder, G1, revealed the same type separation. A brief inspection of a similar location (G7 to mid-span joint) on the eastbound bridge revealed a similar type gap had been sealed. It is presumed those separations were construction-related and maybe the sealing had been omitted from the westbound bridge.

No visible cracks were detected on the undersides of the deck on the center span between the piers. A fence between the piers and the roadways prevented close access to the side spans, except near the abutments. No cracking was observed on the underside of the bridge deck at those locations. Cracks observed on the deck may have occurred at the edges of the girders.

The westbound overpass for Freedom Way Road is a two-lane, simple-span steel girder bridge. The structure has eight rolled, 36-inch, I-beam girders having six horizontal stiffeners fillet-welded to the beams. The girders are interconnected with 14-inch channels bolted to the horizontal stiffeners.

Inspection of Bents 1 and 2 (abutments) and concrete retaining walls revealed no signs of disturbance or cracking. There were no signs of cracking on the bearing caps supporting the three rightmost girders (G6, G7, and G8). The lead sheet under the bearing pads showed some signs of possible slippage. However, inspection of bearing locations on the left side of the bridge revealed a similar disturbance. This observation is believed to be unrelated to the overload under investigation.

The three rightmost girders (G6, G7, and G8) were visually inspected at the abutments. The webs showed no signs of buckling at those locations. The stiffener-to-web fillet welds were visually inspected in the tension areas. No signs of weld fracture were evident.

Inspection of the underside of the bridge deck revealed transverse cracks in the concrete at approximately 6- to 8-foot intervals. The cracks had effloresced, indicating their occurrence prior to the loading under investigation. Close inspection revealed no cracks within the efflorescent deposits or between the deposits and the concrete. No cracks without efflorensence were detected. Joints between the bridge deck and Girders G6, G7, and G8 were tight.

The tension areas of the stiffener-to-girder fillet welds in four locations (near Bent 1) on Girders G7 and G8 were nondestructively inspected using the eddy-current method. A Hocking Model AV-10 eddy-current tester was used with a special 100 kHz cross-axis coil probe. The fillet welds were inspected along the lowest 12 inches to the termination of the fillet welds (at the stiffener copes). No flaws were detected. However, the testing had to be based on differential signals due to lack of a fillet-weld crack standard.

It does not appear the two bridges suffered significant, detectable damage due to the overload. Also, the bridges appear to be in good condition.

PAVEMENT

FIELD TESTING

Prior to field testing, three sites were chosen and designated as Sites A, B, and C. These sites were selected representing typical pavement conditions and considering safety of traffic and work crews. Site A began at Slab 72, Site B began at Slab 216, and Site C began at Slab 304 as shown in Appendix A. Rows were marked at each site. Row A was on the shoulder 2 feet from the outer edge. Row B was on the shoulder, 2 feet from the longitudinal joint between the shoulder and mainline. Row C was on the outer lane of the mainline, 2 feet from the longitudinal joint with the shoulder. Row D was on the centerline of the outer lane of the mainline. Row E was on the outer lane of the mainline, 2 feet from the longitudinal joint between that lane and the adjacent mainline lane. Field sampling and testing for materials characterization performed on October 17, 18, and 21, 1985, included coring the rigid pavement in the outer lane and shoulder and performing in-place California Bearing Ratio (CBR) tests on the subgrade at selected locations. The field testing program is described and results are summarized in Table 1.

Seventeen cores were obtained using a portable core drill. Locations of sites cored (for compressive strength, elastic moduli, and subgrade CBR´s) are illustrated on the visual survey forms in Appendix A. Cores were marked for the purpose of identification for laboratory testing. Seven cores were obtained from Site A, with cores taken in Row A at Slabs 72 and 73, and cores were obtained from all rows at Slab 83. Five cores were obtained at Site B, with cores obtained in each row at

Slab 225. When the core in Row A at Slab 225 was removed, a void about 3 inches in depth was noted between the bottom of the pavement slab and the top of the aggregate base. Five cores were obtained at Site C, with cores taken in each row at Slab 307. The water table appeared to be high at this location. When the cores were removed, water rose in the core holes to a level within 6 to 9 inches of the pavement surface. The holes were pumped dry and the water level returned to within 6 to 9 inches of the pavement surface within 4 to 5 minutes.

In conjunction with the coring, five in-place CBR tests were attempted, and three were completed. The penetrations and calculations for in-place CBR tests were performed in accordance with ASTM D 1883-73, except tests were performed on the soil in its actual in situ condition. The first in-place CBR test was attempted in Row A at Slab 72. After removal of the pavement core, the aggregate base was removed to expose the subgrade soil for testing. A large rock at the subgrade surface prevented completion of the test. An in-place CBR test was then completed in Row A of Slab 73. At this location, the subgrade material was composed of a high percentage of large gravel (diameter about 3 inches) in a soil matrix. To insure that the in-place CBR test was representative of the underlying material, about 12 inches of the soil-rock subgrade was removed. The resistance to penetration (in-place CBR = 56) indicated a very strong supporting layer at this location. Another in-place CBR test was performed in Row C at Slab 83. As with the previous test, large gravel particles were encountered, and about 12 inches of material were removed. The resistance to penetration (in-place CBR = 43) indicated a very strong supporting layer. An in-place

CBR test was completed in Row B at Slab 225. The subgrade at this location appeared to be a silty clay. The large gravel previously noted was not detected at this location. The resistance to penetration (in-place CBR = 7) was much lower than at the locations previously noted. An in-place CBR test was attempted in Row B at Slab 307. Water that rose in the hole (as previously noted) caused problems in preparing a representative surface for the testing, and the test was abandoned.

LABORATORY TESTING

Four soil samples placed in containers, which were then sealed, were returned to the laboratory for moisture content determinations in accordance with ASTM D 2216-80. The soil samples were obtained from in-place CBR locations. Results of the moisture content determinations are included in Table 1.

Seventeen pavement cores were returned to the laboratory for further testing. The locations of these cores were described previously. All cores were measured for length in accordance with ASTM C 42-84a, which requires that cores used for length determination have a nominal diameter of 4 inches. Additional testing of the cores included determinations for unit weight, compressive strength, and modulus of elasticity. Sonic and static moduli of the concrete represent approximate upper and lower bounds, respectively, of the concrete modulus of elasticity. To best simulate field conditions, the cores were tested in the dry condition, which is permitted by ASTM C 42-84a. Three cores were sawed into rectangular prisms and tested for modulus of elasticity by fundamental transverse frequency (sonic modulus) in accordance with ASTM C 215-83. The prisms were cored to a nominal diameter of 2 inches for further testing. The 4-inch cores obtained from the shoulder also were cored to a nominal diameter of 2 inches to achieve a length-to-diameter ratio of 2. The remaining mainline cores were tested at a diameter of 4 inches. All cores were sawed to a length-to-diameter ratio of 2 and tested for unit weight, compressive strength, and static-chord modulus of elasticity (static modulus) in accordance with ASTM C 469-83. Compressive strength tests under this standard also must comply with the requirements of ASTM C 39-84 and ASTM C 42-84a. ASTM C 42-84a requires that the diameter of cores tested for compressive strength be at least twice the maximum nominal size of the coarse aggregate used in the concrete. The design nominal maximum size was 1 inch. Although the nominal diameter of all cores was 2 or 4 inches, the actual diameter of the nominal 2-inch diameter cores was slightly less than 2 inches (about 1.9 inches). This was not considered a major deviation from the standard, since a visual inspection of the cores before and after breaking did not reveal any aggregate particles greater than 3/4 inch. To further verify the validity of the compressive strength results, a comparison was made between the average compressive strength of the 2-inch diameter cores from the mainline and the 4-inch diameter cores from the mainline. There was a difference of approximately five percent, with a lower standard deviation for the 2-inch diameter cores. All results from the laboratory testing of cores are summarized in Table 1.

The urgency of the problem required concurrent analyses for determining in-place conditions and evaluating the load induced stresses by several methods. A "rule of thumb" empirical method to determine the modulus of elasticity for normal-weight concrete is

$$\text{Modulus of Elasticity} = 57{,}000\ (f'_c)^{0.5} \qquad 1$$

where f'_c = compressive strength in psi.

The average compressive strength from laboratory tests was 7,210 psi. Using Equation 1, this yields a modulus of elasticity of 4.8 million psi. Equation 1 (1-3) (numbers in parenthesis refer to those in the References) was derived from

$$\text{Modulus of Elasticity} = 33\ (\gamma)^{3.5}\ \times\ (f'_c)^{0.5} \qquad 2$$

where γ = weight in pounds per cubic foot (pcf).
Assuming a unit weight of 144 pcf in Equation 2 (2, 3) yields the constant "57,000" in Equation 1. The minimum unit weight of cores obtained from the Jefferson Freeway was 145 pcf.

A review of available literature (4) provided the relationship between compressive strength and modulus of rupture:

$$\text{Modulus of Rupture} = k(f'_c)^{0.5} \qquad 3$$

where k = a constant varying from 8 to 10.
Data published by the Portland Cement Association (4) provide the following relationship:

$$\log\ (E) = 6.2340368 + 0.0005634989 \times MR \qquad 4$$

where E = modulus of elasticity for portland cement concrete and
MR = modulus of rupture for portland cement concrete.
Figure 4 illustrates the relationships described by Equations 3 and 4. Figure 5 combines Equations 3 and 4 to indicate that, for a unit weight

of 144 pcf and compressive strength of 7,200 psi, k would be approximately 9.6. Equation 1 was used to estimate the modulus of elasticity as a function of the compressive strength. The mean of the compressive strengths yielded a modulus of elasticity of 4.8 million psi. The relationship in Equation 2 provided a modulus of 5.1 million psi. The difference of 0.3 million psi produces insignificant changes in the response data.

The modulus of rupture and modulus of elasticity were calculated as a function of compressive strength and unit weight. Unit weights varied from 145 to 150 pounds per cubic foot and the mean was 147. Compressive strength data in Table 1 were divided into two groups — those from the wheel tracks of the loader and those from outside the wheel tracks. The mean associated with the loader's wheel tracks was 6,968 psi while the mean outside the wheel track was 8,070 psi. The mean of the compressive strengths in the wheel track was 86 percent of the mean outside the wheel track.

Sorting the compressive strength data by slab thickness indicated that the mean of the 6-inch shoulder slabs in the wheel track was 81 percent of the mean of the compressive strengths outside the wheel track. For 10-inch mainline slabs, the mean of the compressive strengths from the wheel track was 88 percent of the compressive strengths outside the wheel track.

It is important to note that compressive strengths of the portland cement concrete, as determined from core specimens, were considerably higher than normally would be expected for two-year old concrete, which was specified to meet a 28-day compressive strength of 3,500 psi.

Division of Materials' records for the project indicate the average 28-day compressive strength was 6,130 psi. It is almost certain that damage caused by passage of the front-end loader would have been considerably more extensive in the event portland cement concrete placed on the project only met or slightly exceeded specification requirements.

DEFLECTION TESTING FOR VERIFICATION OF ELASTIC MODULI

Dynamic deflection measurements were used to determine in-place structural conditions of the rigid pavements. Additionally, deflection measurements were used for comparison of conditions for one site relative to another. Comparisons of one site versus another may involve comparisons of relative magnitudes of measured deflections. More sophisticated evaluations involve comparisons of conditions expressed as the "back-calculated" effective layer moduli or as load-transfer efficiencies.

Data and information previously presented describe methods for testing pavement cores and the subgrade to determine strength and load-carrying capacities of the various components of the pavement structure. The "static chord method" of ASTM C 469 is a destructive test wherein stress and strain measurements are determined for a wide range of loadings until the pavement core or laboratory sample is failed in compression. The compressive strength of the sample also is determined from this test. Typically, the resulting elastic modulus determined from this procedure is representative of the weakest expected modulus of elasticity or stiffness for the material. Moduli of elasticity also may be estimated using results of nondestructive measurements. Analyses involving the determination of the fundamental frequency (transverse or longitudinal) for a concrete prism or core and the use of these measurements to estimate the elastic modulus are presented in ASTM C 215. Wave propagation analyses described in ASTM C 215 are typically representative of the strongest or stiffest expected elastic modulus for the material. Deflection measurements also have been used to "back calculate" the effective elastic modulus for the various component layers of a pavement structure. Procedures developed in Kentucky relating to the use of dynamic deflections for back calculation of effective pavement conditions are described in Reference 5.

Finally, empirical relationships have been developed relating more easily and commonly determined factors with material stiffness or modulus of elasticity. Two relationships that may be used to select design parameters are 1) for concrete, Equation 2 may be used; and 2) for subgrade soil (6),

$$E_{Subgrade} = 1500 \times CBR \qquad 5$$

where $E_{Subgrade}$ = elastic modulus for subgrade soil (psi) and

CBR = California Bearing Ratio of the subgrade material.

Deflection measurements were obtained at the three sites. Deflection testing at Site A consisted of measurements (using the Road Rater) for twelve consecutive slabs in the outside shoulder and the three adjacent lanes. Deflection measurements were obtained on a grid pattern involving the joint, third points, and midpoints of the slabs; centerline and wheel paths for each lane; and the wheel paths for the shoulder. Deflection testing of eleven slabs at Site B was conducted on a similar grid pattern, except third-point testing was eliminated.

Deflection testing at Site C was conducted on a similar grid pattern, but consisted of deflection testing only in the shoulder and outside lanes and only at joints and midslab locations. Test locations at all sites are shown on the data forms in Appendix A.

Deflection measurements at midslab and third-point locations were used to "back calculate" the effective condition of the portland cement concrete and subgrade material. Deflection measurements at joints and midslab locations were used to estimate the load-transfer efficiency for each joint. Generally, the procedure (5) for estimation of effective pavement condition involves the theoretical simulation of deflection measurements (for the Road Rater loading configuration) for a matrix of combinations of layer thicknesses and elastic layer moduli for the portland cement concrete pavement, crushed stone base, and subgrade soil.

The elastic modulus for the crushed stone base is expressed in terms of the elastic moduli of the supporting and overlying layers (subgrade and portland cement concrete). The effective pavement and subgrade conditions are expressed in terms of a combination of layer thicknesses and elastic moduli for the portland cement concrete and subgrade resulting in a theoretical deflection bowl that matches the measured deflection bowl. There are a number of combinations of elastic moduli for the subgrade and portland cement concrete pavement that potentially result in a theoretical deflection bowl matching the measured deflection bowl. The design layer thicknesses for the mainline and ramp pavements (10 inches portland cement concrete and 6 inches dense-graded limestone aggregate) and for the shoulder pavement (6 inches portland cement concrete and 10 inches dense-graded limestone aggregate) were used for back-calculation of the effective subgrade moduli, assuming an elastic modulus of the portland cement concrete as discussed previously. Actual layer thicknesses varied by a small degree relative to the nomimal or design layer thicknesses used for the analysis. However, these variations were not considered a significant influencing factor.

Three specific elastic moduli were assumed in an attempt to "bracket" the effective subgrade conditions. These analyses provide additional information describing in-place material properties for use in determining damage associated with one pass of the front-end loader. The three assumptions for the elastic moduli of the portland cement concrete were 2.0 million psi, 4.8 million psi, and 6.0 million psi.

A preliminary review of data in Table 1 indicated a representative elastic modulus by the static chord method (ASTM C 469) of 2.0 million psi. Similarly, a representative elastic modulus of 6.0 million psi was determined by the fundamental (transverse) frequency method (ASTM C 215). Using the compressive strength data and assuming a unit weight of 144 pcf in Equation 2 indicated a representative elastic modulus of 4.8 million psi. It was necessary to make these assumptions on the basis of preliminary reviews of partial data samples in order to proceed with data analyses in an efficient manner and at the same time meet a demanding schedule.

A more detailed summary of compressive strengths and measured deflections is presented in Table 2. Back-calculated elastic moduli associated with deflection measurements and core sites only and the three assumptions for the elastic modulus of the portland cement concrete are presented in Table 3. Table 4 summarizes the results of analyses for adjustment of back-calculated subgrade moduli on the basis of compressive strength analyses of cores. A review of data presented in Tables 2 and 3 indicates typical ranges of minimum subgrade elastic moduli of 8,344 psi to 12,007 psi for mainline and ramp pavements and 7,871 psi to 10,695 psi for shoulder pavements based upon the three assumed elastic moduli for the portland cement concrete pavement. Adjustment of those same data on the basis of actual elastic moduli determined by ASTM C 469, ASTM C 215, and Equation 2 indicates only a small change in the effective back-calculated elastic moduli (9,000 psi to 12,750 psi) for the mainline pavement section. Similar variations were observed for the other comparisons of back-calculated elastic moduli values for the assumed elastic moduli for the portland cement concrete and the elastic moduli determined by field and laboratory tests.

Data presented in Tables 5 through 9 summarize average back-calculated elastic moduli of the subgrade for deflection measurements obtained for each test site. Figures in Appendix B illustrate the distribution of back-calculated elastic moduli for the various test sites.

Results of in-place California Bearing Ratio (CBR) tests reported in Table 1 indicated a minimum CBR of 7. The elastic modulus computed using Equation 2 associated with a CBR 7 subgrade is 10,500 psi. A review of the figures in Appendix B indicates a similar typical minimum elastic modulus for the subgrade. Therefore, it was decided to use a minimum CBR of 7 ($E_{Subgrade}$ = 10,500 psi) for the bearing capacity (or elastic modulus) of the subgrade for purposes of fatigue and other damage-related assessments.

The compressive strength of a concrete specimen is typically a common measure of the quality of the concrete. Therefore, it was further decided the elastic modulus determined on the basis of unit weight and compressive strength (Equation 2) would be used to characterize the portland cement concrete for purposes of fatigue analyses and damage assessments.

EFFICIENCY OF LOAD TRANSFER AT JOINTS

Theoretically, a dowel at a joint is considered 100 percent efficient when the dowel transfers one-half of the applied load from one slab to an adjoining slab. This is true in the event each slab at the joint deflects an equal amount and each assumes one-half of the applied load (7). Empirical procedures have been developed in Kentucky (5) wherein dynamic deflection measurements are used to estimate the efficiency of load transfer for joints in portland cement concrete pavements. More specifically, the procedure involves a comparison of deflection measurements at midslab to deflection measurements at a joint or crack. The deflection bowl at midslab is determined for some constant dynamic load. For the same load, the deflection at the joint is determined by placing the load feet of the Road Rater on one side of the joint (or crack) and positioning the joint (or crack) between the second and third sensors. The efficiency of load transfer is estimated as the ratio of the difference between deflection for the second and third sensors at midslab compared with the similar differences between deflections for the deflection bowl at the joint.

This method of evaluation of load transfer is currently experimental from the perspective of qualitative analyses. However, comparisons of one site relative to another may be presented. The first three figures in Appendix C are three-dimensional representations of load-transfer efficiencies for each slab of each test site. Similar representations also are presented for averages of load-transfer efficiency by lane for each test site. The larger the "tower", the greater is the estimated efficiency of load transfer for each joint as was determined on the basis of dynamic deflection measurements.

A review of Appendix C indicates considerable scatter by test site. The vehicle in question generally followed wheel paths D and B for each of the three sites. There does not appear to be a significant relationship between the path of the0017vehicle in question and load transfer efficiency except for Site A, where there are lower load-transfer efficiencies for Rows C and D. Even though the analyses of load transfer efficiencies are somewhat inconclusive, they do support the results of other analyses and observations that indicate premature distress in the wheeltracks of the front-end loader.

The relatively high degree of scatter for estimated efficiencies of load transfer was cause for concern. Therefore, additional coring was performed within the area of joints and dowel-bar assemblies. The positions of those additional cores as well as other coring for elastic modulus and CBR determinations are shown on figures in Appendix C. Cores were obtained from areas in the path of the front-end loader as well as from areas well isolated from the path of the loader. Cores were obtained in areas where there was evidence of surface distress as well as in areas where no surface distress was observed. Nine of ten cores indicated a horizontal crack in the concrete on either the top or bottom, or both, of the dowel bar regardless of the core location. An attempt was made to determine the extent of cracking by coring approximately 6 inches from cores made through the dowel. Only one of six cores indicated any evidence of horizontal cracking at that distance from the dowels. A more detailed description of the findings for the evaluation of this set of cores is contained in Table 10.

On the basis of available information and data, an explanation for the observed cracking has not been developed at this time. It may be hypothesized that the observed cracking is a result of the pressure exerted on a dowel by an external load. An illustration of a typical load-pressure diagram for a dowel is presented in Figure 3.10 of Reference 7.

The observed horizontal cracking also may be attributed to stresses created by temperature changes -- associated expansion and contraction of the portland cement concrete. Somewhat similar observations for other pavement sections have been reported (8). Additional study is necessary to adequately determine the extent and cause of the observed horizontal cracking. However, at this time, there is no direct evidence that the passage of the front-end loader is solely responsible for the observed horizontal cracks; but it is likely that the passage of the front-end loader accelerated deterioration by contributing to the propagation of the cracks.

ASSESSMENT OF DAMAGE

VISUAL CRACK INSPECTION

Three sections of pavement were inspected visually by walking and closely inspecting each slab of the mainline and shoulder in the wheel tracks of the front-end loader. The wheel tracks were obvious due to the scuff marks caused by the cleats. A few diagonal cracks were observed on the mainline pavement. None of those cracks were deemed to have been caused by the loader.

All cracks and spalled areas of crushed concrete at joints were noted by location and slab number on field sheets. Appendix A illustrates findings of the visual survey.

The loader traveled over 928 of the 10-inch thick slabs of mainline and ramp pavement and 367 of the 6-inch thick slabs of paved shoulders. Use and analysis of this information is presented elsewhere in this report.

The Jefferson Freeway was revisited on November 18, 1985, to confirm details. While inspecting Site C, four additional shoulder slabs were observed as being cracked in the wheel tracks of the front-end loader. Those cracks were not evident during the original survey performed on October 17 and 18, 1985. Apparently, the additional cracking was evident during the November 18 inspection because the slabs had cooled and contracted. Microcracks formed earlier had widened and become visible as the slab and air temperatures decreased. Additional cracking observed on November 18 also is included in Appendix A. Crack surveys were made during times when the pavement surfaces were dry. Experience has shown that cracks are more evident when the pavement has been wetted and then begins to dry. Additional cracks might have been observed if the pavement had been wetted.

FATIGUE AND THICKNESS DESIGN METHODS

Pavement thickness design methods have been developed on the assumption that failure occurs through fatigue. Fatigue theory assumes the pavement is flexed by a few large loads, or more repetitions of a lesser load. The portland cement concrete thickness design system (9, 10) developed and used in Kentucky is a merger of the Portland Cement Association (11) and the AASHTO (12) methods coupled with Kentucky experience. The Portland Cement Association method was developed from data obtained on light-to-medium volume roads (by today´s conditions) while the AASHTO method was developed from data taken at the AASHO Road Test that simulated high truck volumes and heavy axleloads.

The Portland Cement Association method is based on a relationship beginning with one repetition of a catastrophic load producing a ratio of stress to modulus of rupture of 1.0. As the number of repetitions increases, the magnitude of load must decrease, producing lesser stresses corresponding to decreasing values of stress ratios. In addition, this method is supported by laboratory tests of beams tested in flexure. Laboratory tests are performed on specimens of limited size, permitting measurements of stresses in only one direction (tangential). Thus, the stress-ratio system used in this method could not account for stresses that might be almost as large but occurring in other directions (radial, shear, and vertical). The AASHTO method may be expressed as a ratio of stresses defined as a function of the desired number of repetitions. This method also is supported by laboratory fatigue test data. However, the mathematical expressions of the AASHTO and Portland Cement Association methods differ. The two systems merged in the range of two million repetitions of an 18,000 pound single axleload (9).

Strain energy density is defined as the work at a given point within a body caused by a force applied to the outside of the body (9). The equation for strain energy density (9) accounts for all stresses acting in all directions. Figure 6 was developed to illustrate the relationship used to convert magnitude of stress in one direction to an equivalent value of strain energy density. The fatigue criterion relationship permits converting a given ratio of stress corresponding to an assigned level of fatigue to the ratio of strain energy density as shown in Figure 7. The Chevron N-layer computer program was used to calculate all the one-direction stresses and including them in one equation to calculate a corresponding value of strain energy density. The Kentucky thickness design criterion is expressed as ratios of strain energy density as a function of repetitions (Figure 7). The program assumes that all layers extend infinitely in a horizontal direction. Thus, the Chevron N-layer program may be used only to analyze a load applied at the center of a slab and may not be used to analyze a joint or edge condition. However for practical purposes, calculated stresses and strains of portland cement concrete slabs are negligible beyond 6 feet from the applied load.

EVALUATION OF STRESSES IN SLABS UNDER LOADS

To evaluate the fatigue of the slabs over which the front-end loader had traversed, stresses induced in both the 10-inch mainline slabs and the 6-inch shoulder slabs were evaluated using three independent methods (see Appendix D) -- the Chevron N-layer elastic computer program, influence charts presented by the Portland Cement Association (13), and the ILLI-SLAB finite element computer program (see Appendix E). After determining stresses, they were compared to the design or allowable stresses to obtain stress ratios. Stress ratios of 1.0 or greater indicate the fatigue life of a pavement slab has been equaled or exceeded. By all three methods, the stress ratios in the 10-inch mainline slabs were less than one, indicating the front-end loader had not "consumed" all of the fatigue life of the slab. Analysis of fatigue indicated only nominal loss of service life (approximately 1.1 percent). However, in the case of the 6-inch shoulder slabs, the stress ratio was significantly higher than one. This indicated the fatigue life of the shoulder slabs had been completely consumed by the one passage of the front-end loader.

It also should be noted that the stresses induced in slabs may be influenced by the support provided by the underlying foundation or subgrade materials. Elastic moduli of the subgrade were back-calculated based on deflection data. The distribution of back-calculated elastic moduli are summarized in Appendix B for both shoulder and mainline pavement sections. A review of these distributions indicated 89 percent of the back-calculated subgrade moduli were less than 30,000 psi. Further review indicated 100 percent of back-calculated elastic moduli were less than or equal to 40,000 psi. Figure 8 indicates a stress ratio of 0.96 corresponding to 40,000 psi subgrade modulus of elasticity. For a modulus of 30,000 psi, the stress ratio is 1.10. By interpolation, it was estimated 98 percent of back-calculated subgrade moduli were less than 38,000 psi. This value was then used as a critical value -- any subgrade modulus less than 38,000 psi would result in stress ratios greater than 1.0 and thus a "consumption"of the total fatigue life of the slab with one passage of the front-end loader.

ECONOMIC ANALYSES

An economic assessment of damage was made in two phases. The first phase of the analyses related to an assessment of repair costs associated with the findings of the visual inspection previously discussed and based on observable distress or damage attributed to the passage of the front-end loader in question (Appendix A). The second phase involved an economic assessment of damage associated with the loss of the service life associated with the acceleration of fatigue in the pavement structure and/or near catastrophic loading applied to the pavement structure.

PHASE I -- ASSESSMENT OF REPAIR COSTS

Findings of the visual distress survey are summarized in Table 11. Distresses are subdivided by location of occurrence (10 inches mainline and/or ramp pavement and 6 inches shoulder pavement), types of observed distresses (powdered or shattered concrete at the transverse joint areas but not apparently a deep-seated failure, corner or diagonal cracking, transverse cracking, longitudinal cracking, powdered or shattered concrete at longitudinal joint areas but not apparently a deep-seated failure), and survey section.

A repair strategy relating to each observed distress was formulated. The estimated cost per unit was then determined for use in determining the costs to repair observable distresses resulting from passage of the front-end loader. A summary of each repair strategy and documentation for the unit cost for each repair strategy follows

A. Repair strategy for powdered or shattered edges of joints

1. The same strategy shall apply to both transverse and longitudinal joints.

2. The purpose for conducting this repair activity is to restore the integrity of the joint and to seal the joint to minimize future damage associated with moisture intrusion and freezing and thawing.

3. Repair strategy: Use latex concrete or epoxy concrete for patching. Estimated quantities are determined on the basis of the volume of a 1-foot long x 1/3-foot wide x 1/3-foot deep cavity (0.11 cubic feet of patching material).

4. The estimated unit cost for latex patching material was estimated on the basis of unit bid costs associated with three repair and patching projects (data provided by the Division of Design):

a. I-24 PCC Pavement Repair:
latex patching -- $106/cubic foot

b. I-64 Montgomery-Bath-Rowan Counties PCC Pavement Repair:
latex patching -- $200/cubic foot

c. I-64 Franklin County PCC Pavement Repair:
latex patching -- $75/cubic foot

d. Average bid cost for latex concrete patching -- $127/cubic foot

5. The same repair strategy was applied to both 10-inch pavement sections (mainline and ramp sections) and 6-inch pavement sections (shoulder sections).

6. The cost for repair per lineal foot of powdered or shattered longitudinal and transverse joints -- 0.11 cubic foot/lineal foot x $127/cubic foot = $13.97/lineal foot

7. The total lineal feet of powdered or shattered longitudinal and transverse joints were obtained from Table 11:

a. Transverse joints:

Mainline/Ramp -- 591 feet x $13.97/lineal foot = $8,256.27

Shoulder -- 168 feet x $13.97/lineal foot = $2,346.96

b. Longitudinal joints:

Mainline/Ramp -- 420 feet x $13.97 = $5,867.40

Shoulder = 14 feet x $13.97 = $195.58

c. TOTAL = $16,666.21

B. Repair strategy for corner/diagonal cracking for mainline, ramp, and shoulder pavement sections

1. The same repair strategy will be applied to all pavement sections. The repair quantities will vary because of the varied pavement thickness.

2. The repair strategy should consist of full-depth, partial-width patching for each area where corner cracking was observed as presented in Appendix A.

3. The purpose for the repair activity is restoration of the structural integrity for the damaged pavement section.

4. Repair quantities are based on an assumed full-depth patch area of 4 feet x 4 feet = 16 square feet or 1.78 square yards.

5. The estimated unit cost for repair will vary dependent upon the slab thickness (10 inches for mainline and ramp pavements and 6 inches for shoulder pavement sections).

a. Unit costs for removal and replacement of PCC pavement and other incidental construction costs were obtained from the Division of Design.

b. I-24 PCC Pavement Repair: Removal and replacement -- 10-inch Pavement @ $50.95/square yard

1-inch diameter transverse tie bars @ $10.50 each

1 1/4-inch dowel bars @ $11.30 each

c. I-64 Montgomery-Bath-Rowan Counties PCC Pavement Repair: Removal and replacement -- 10-inch Pavement @ $55.00/square yard

d. I-64 Franklin County -- PCC Pavement Repair Removal and replacement -- 10-inch PCC Pavement @ $45.50/square yard

e. Average Costs: Removal and replacement -- 10-inch PCC Pavement @ $50.48/square yard

1-inch diameter transverse tie bars @ $10.50 each

1 1/4-inch dowel bars @ $11.30 each

f. No unit costs were available for removal and replacement of 6-inch PCC shoulder pavement. Therefore, the unit cost was estimated on the basis of the ratio of the cost for new construction of 6-inch PCC shoulders over the cost for new construction of 10-inch PCC pavement and applied to the average cost for removal and replacement of 10-inch PCC pavement. The unit costs associated with new pavement and or shoulder construction were determined for a recently awarded section of the Jefferson Freeway near the Beulah Church Road in Jefferson County and were supplied by the Division of Design:

6-inch PCC Shoulder @ $15.95/square yard

10-inch PCC Pavement @ $18.58/square yard

Ratio = 0.86

(0.86)($50.48 square yard) = $43.41/square yard for removal and replacement of 6-inch PCC shoulder pavement

g. Unit cost for each patch:

i -- 10-inch PCC mainline section

$50.48/square yard x 1.78 square yards = $89.85 per patch (removal/replacement)

4 tie bars @ $10.50 each = $42.00

4 load transfer dowel bars @ $11.30 per patch = $45.20

TOTAL COST PER PATCH = $177.05

ii -- 6-inch PCC shoulder Section

$43.41/square yard x 1.78 square yards = $77.27 per patch (removal/replacement)

4 tie bars @ $10.50 each = $42.00

TOTAL COST PER PATCH = $119.27

6. The total number of areas requiring this type repair were obtained from Table 11:

a. Mainline/ramp pavement sections

69 areas x $177.05 per patch = $12,216.45

b. Shoulder pavement sections

24 areas x $119.27 per patch = $2,862.48

c. TOTAL = $15,078.93

C. Repair strategy for transverse cracking

1. Due to the short slab lengths (average 15 feet), it was determined that an appropriate repair strategy would be total slab replacement in lieu of partial replacement.

2. The purpose of this repair is restoration of the structural integrity for the damaged pavement sections.

3. Repair quantities for ramp and mainline pavement sections (10-inch pavement sections) are based on slab dimensions 12 feet by 15 feet (180 square feet or 20 square yards).

4. The width of shoulder sections varied from 6 feet to 10 feet depending upon whether they were adjacent to ramp or mainline sections or were in transition zones.

a. A review of data presented on the visual survey data sheets in Appendix A combined with field inspections indicated the front-end loader traveled over 367 slabs. Of those, 106 slabs were 6 feet wide, 12 were in a transition zone from 6 feet to 10 feet in width (assume 8-foot width), 133 slabs were 10 feet wide, 15 were in a transition zone from 10 to 8 feet (assume 9-foot width), and the remaining 101 were 8 feet wide.

b. The weighted average for the width of shoulders traversed by the front-end loader is 8.2 feet. Therefore, 8 feet was used for the width of shoulders for determination of quantities for analyses of repair strategies.

c. Repair quantities for shoulder pavement sections (6-inch pavement sections) are based on slab dimensions 8 feet by 15 feet (120 square feet or 13.33 square yards).

5. The unit cost for removal, replacement, and reconstruction of a 10-inch PCC pavement slab:

a. 20 square yards x $50.48/square yard/slab = $1,009.60

b. Twelve 1 1/4-inch dowel bars @ $11.30 each = $135.60

c. Six 1-inch transverse tie bars @ $10.50 each = $63.00

d. TOTAL = $1,208.20

6. The cost for removal, replacement, and reconstruction of a 6-inch PCC pavement slab:

a. 13.33 square yards x $43.41/square yard = $578.66

b. 6 1-inch transverse tie bars @ $10.50 each = $63.00

c. TOTAL = $641.66

7. The number of areas having transverse cracking requiring repair is presented in Appendix A. Transverse cracking was observed at only one location, but affected both a mainline slab and a shoulder slab. The cost of repair is the cost of total replacement for one mainline slab ($1,208.20) and one shoulder slab ($641.66) for a total replacement cost of $1,849.86.

D. Repair strategy for longitudinal cracking

1. The repair strategy should consist of full-depth total slab replacement for all areas where longitudinal cracking has been observed as presented in Appendix A.

2. The purpose for the repair activity is restoration of the structural integrity for the damaged pavement section.

3. Repair quantities for total slab replacement for both mainline and ramp pavements and also shoulder pavements have been previously determined:

a. $1,208.20 per 10-inch PCC pavement slab

b. $ 641.66 per 6-inch PCC shoulder pavement slab

4. Visual inspection results presented in Appendix A indicate the need for total replacement of 56 shoulder slabs. The cost associated with this repair activity is 56 slabs x $641.66/slab = $35,932.96.

E. Summary of costs associated with repair of distresses and damages as illustrated in Appendix A:

1. Powdered or shattered joints (longitudinal and transverse)	$16,666.21
2. Corner/diagonal cracking (mainline, ramp, and shoulder sections)	$15,078.93
3. Transverse slab cracking	$ 1,849.86
4. Longitudinal slab cracking	$35,932.96
5. TOTAL COSTS FOR REPAIR OF OBSERVABLE DISTRESSES	$69,527.96

PHASE II -- ASSESSMENT FOR LOSS OF SERVICE LIFE

The second phase of the economic analysis involves an assessment of damages associated with the acceleration of pavement fatigue and/or the damage to the pavement structure resulting from the near catastrophic overloading by the front-end loader. Such damage is likely to significantly reduce the effective service life of the pavement section. Procedures and findings of detailed laboratory and field materials characterization activities have been previously documented. Results of these analyses were used in the analysis of fatigue associated with the passage of the front-end loader.

Fatigue analyses of the 10-inch pavement for mainline and ramp sections were based upon a CBR 7 subgrade material (determined from minimum in-place CBR tests and supported by back calculation from deflection testing analyses) and an elastic modulus of 5.0×10^6 psi for the portland cement concrete (determined by laboratory analyses for compressive strength, density, static chord elastic modulus, and sonic modulus by transverse fundamental frequency and also indirectly supported by deflection measurements). The analysis of fatigue indicated only nominal loss of service life (1.1 percent) as the result the passage of the front-end loader.

Figure 9 illustrates a straight-line deterioration relationship (on a per slab basis) for the 10-inch pavement section. Initial pavement construction cost data (Table 12) were supplied by the State Highway Engineer´s office. The salvage value was estimated as the effective worth of the pavement structure if total deterioration rendered the pavement section the equivalent of a crushed stone base. This assumption is not unfounded since current pavement thickness practices for design of asphaltic concrete over broken and seated portland cement concrete pavement (in-place recycled portland cement concrete pavement) are based on the assumption the broken concrete will behave no worse than a crushed stone base having a thickness equal to the total thickness of the existing pavement section (portland cement concrete pavement plus dense-graded limestone aggregate). The deterioration schedule in Figure 9 is used to determine the current worth of the pavement section in 1983 dollars (the pavement was completed in 1983). This value may then be converted to 1985 dollars using the single payment compound amount factor (13):

$$SPCAF = (1 + i)^n \quad 6$$

where i = interest rate expressed as a decimal and

n = time period (years).

An interest rate of eight percent was assumed for this analysis. The fatigue loss per slab (approximately 1.1 percent) is applied to this value (current value in 1985 dollars) plus the cost of the asphaltic concrete overlay required to complete salvage of the pavement section. The cost associated with a 5-inch asphaltic concrete overlay is then added to the salvaged value (1985 dollars) of the slab. A unit cost of $25.00 per ton was used for asphaltic concrete. Figure 9 illustrates these calculations. An estimated loss of service life associated with fatigue of the 10-inch PCC pavement may then be determined.

An alternate method of estimating the economic impact of the loss of service life is the product of the percentage fatigue loss (1.1 percent) times the current removal and replacement costs. The calculation of initial construction costs, salvage values, and current replacement costs are presented in Table 12. The Division of Design indicated a 20-year expected design life. The 1985 loss per slab may then be multiplied by the total number of slabs traversed by the front-end loader. Figure 9 also illustrates these calculations.

A review of the results of the visual inspections in Appendix A indicated variable widths for some of the pavement slabs because of tapers and transition zones at or near ramp areas. The inspection indicated a total of 833 of the 12-foot wide lane slabs were loaded by the front-end loader. Seventeen slabs were located in taper areas. Addtionally, 78 slabs having an 8-foot width were traversed by at least one wheel of the front-end loader. Therefore, a total of 928 slabs were affected by passage of the front-end loader. For estimation purposes, the total number of slabs was converted to an equivalent number of 12-foot wide slabs by weighting on the basis of slab width. The total number of equivalent 12-foot wide slabs was determined to be 898. The calculation of total pavement loss of economic service life is presented and illustrated on Figure 9.

In summary, analyses on the basis of initial costs and projected salvage values and an assumed 5 inches asphaltic concrete repair strategy indicated a total loss of service life of $4,624.70 in 1985 dollars. On the basis of current removal and replacement costs, the total loss of service life was valued at $10,643.99. Analyses on the basis of the salvage life deterioration curve and asphaltic concrete overlay (and thus the $4,625 cost indicated above) are not totally realistic since it is not possible to overlay small and isolated portions of a roadway (single lane or individual slabs). Calculations presented herein are based on overlay of a small proportion of slabs only, which is not feasible or practical from a construction point of view. The front-end loader passed over only approximately one-third of the roadway. The costs to repair the entire pavement may be estimated from the product of $472.23 per slab (see Alternate A, Item 2.c in Figure 9) and 2,694 slabs (3 x 898 slabs). This results in a total repair cost of $1,272,188. Assuming the approximately 1.1 percent loss of fatigue life, the loss of service is estimated as $13,867 ($1,272,188 x 0.011), which is somewhat greater than the $10,644 estimated on the basis of removal and replacement only of deteriorated slabs. Therefore, it is recommended that damage assessments be determined on the basis of removal and replacement costs, the least costly of the alternatives.

Fatigue analyses of the 6-inch shoulder pavement also were analyzed using a CBR 7 subgrade and an elastic modulus of 5.0×10^6 psi for the portland cement concrete. Results of these analyses are summarized in Appendix D dealing with fatigue analyses. The theoretical stress ratio for the portland cement concrete was in the order of 1.6 for the CBR 7 subgrade. Information in Figure 8 illustrates how stress ratio varies as a function of subgrade strength for a constant modulus of elasticity for the portland cement concrete. Pavement damage in the form of cracking and/or punching of the portland cement concrete may be expected when the stress ratio exceeds 1.0.

Since the stress ratio exceeded 1.0 for analyses on the basis of a CBR 7 subgrade, it was not possible to use the same approach as was used to assess damage for the mainline pavement sections. The fatigue capacity of the shoulder sections has been theoretically exceeded when the stress ratio exceeds 1.0 for the "design condition." Therefore, it was decided to estimate the extent of damage to the pavement shoulders on the basis of the proportion of estimated subgrade strengths (from deflection measurements) exceeding the subgrade modulus of elasticity corresponding to a stress ratio of 1.0 or a subgrade elastic modulus of 38,000 psi or less as determined from Figure 8.

Figure 10 illustrates a straight-line deterioration curve (on a per slab basis) for the 6-inch shoulder pavement section. Initial pavement construction cost data were supplied by the State Highway Engineer's office (Table 13). The salvage value was estimated as the effective worth of the pavement structure if total deterioration rendered the pavement section the equivalent of a crushed stone base. The basis for this assumption has been discussed earlier. The same procedures used for evaluation of 10-inch sections were used for the 6-inch sections excepting for the method used to estimate percent deterioration. These analyses are summarized in Figure 10.

An alternate method to estimate the economic impact associated with a reduction of service life makes use of the product of the percent deterioration or damage and the current removal and replacement costs. The calculation of initial construction costs, salvage value, and current removal and replacement for 6-inch shoulder pavement sections are presented in Table 13.

The distribution of shoulder widths and number of shoulder slabs traversed by the front-end loader have been summarized earlier. A total of 367 shoulder slabs were determined to have been traversed by the front-end loader. A weighted slab width of 8 feet was determined. Each slab was assumed to have a nominal length of 15 feet.

On the basis of current removal and replacement costs (the least costly and at the same time practical strategy, as discussed previously), the total loss of service life was estimated at

$207,702.55. Adding the estimated cost of overlaying the mainline lanes ($1,272,188) to the cost to overlay the shoulders ($259.15 per slab times 367 slabs) results in a total worth for the existing section plus overlay of $1,367,296. The cost of asphaltic concrete overlay alone is $404,056.88 (2,694 mainline slabs times $137.50 per slab plus 367 shoulder slabs times $91.64 per slab). This is considerably greater than the $207,702 estimated for removal and replacement. Therefore, it is recommended that damage assessments be determined on the basis of removal and replacement costs.

CONCLUSIONS

The results of all economic analyses are summarized below:

TOTAL REPAIR COSTS	$69,527.96
LOSS OF SERVICE (1985 DOLLARS)	
Mainline	$10,643.99
Shoulders	$207,702.55
ANALYSES COST	
Kentucky Transportation Research Program	$ 29,200.00
Kentucky Transportation Cabinet	$ 5,000.00
TOTAL	$322,074.50

Visual observations of the relevant sections of the Jefferson Freeway did reveal physical damage to joints in the pavement. This damage could be associated with the passage of the front-end loader because of markings of the pavement surface by the cleats of the vehicle. This damage should be repaired to restore the joints to an as-constructed condition so as not to accelerate and aggravate damage under future traffic loadings and environmental conditions.

Both field and laboratory testing of the subgrade and cores from the portland cement concrete slabs indicate similar characteristics of the materials. Properties back-calculated from surface deflections (obtained with a Road Rater) also verify results of field and laboratory testing and evaluations.

Analyses do show that the loading due to the passage of the front-end loader "consumed" only about one percent of the fatigue (service) life of the mainline slabs. Even though this seems to be only a nominal amount, it is much more than can be attributed to a single pass of any "normal" vehicle allowed on highways. To illustrate, one pass of the front-end loader was equivalent to 35 million automobiles or approximately 53,000 legally loaded five-axle semi-trailer trucks. The single pass of the front-end loader did utilize all of the service life of the shoulder slabs. It was not possible to attribute conclusively the initiation of horizontal cracking at approximately middepth in the portland cement concrete slabs near load-transfer assemblies. However, the analysis of load-transfer efficiencies at joints did suggest the likelihood that the cracking was aggravated and propagated by this unusual loading event.

It should be noted that the cost of restoration is not based on the most expensive strategies. The recommended strategies of rehabilitation were selected on the basis of what was considered to be the most effective and practical.

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LIST OF FIGURES

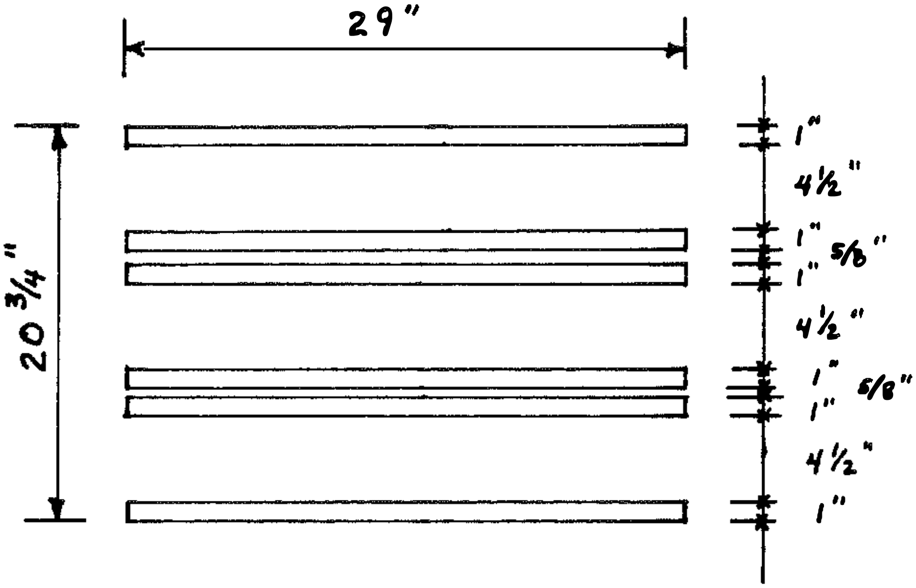
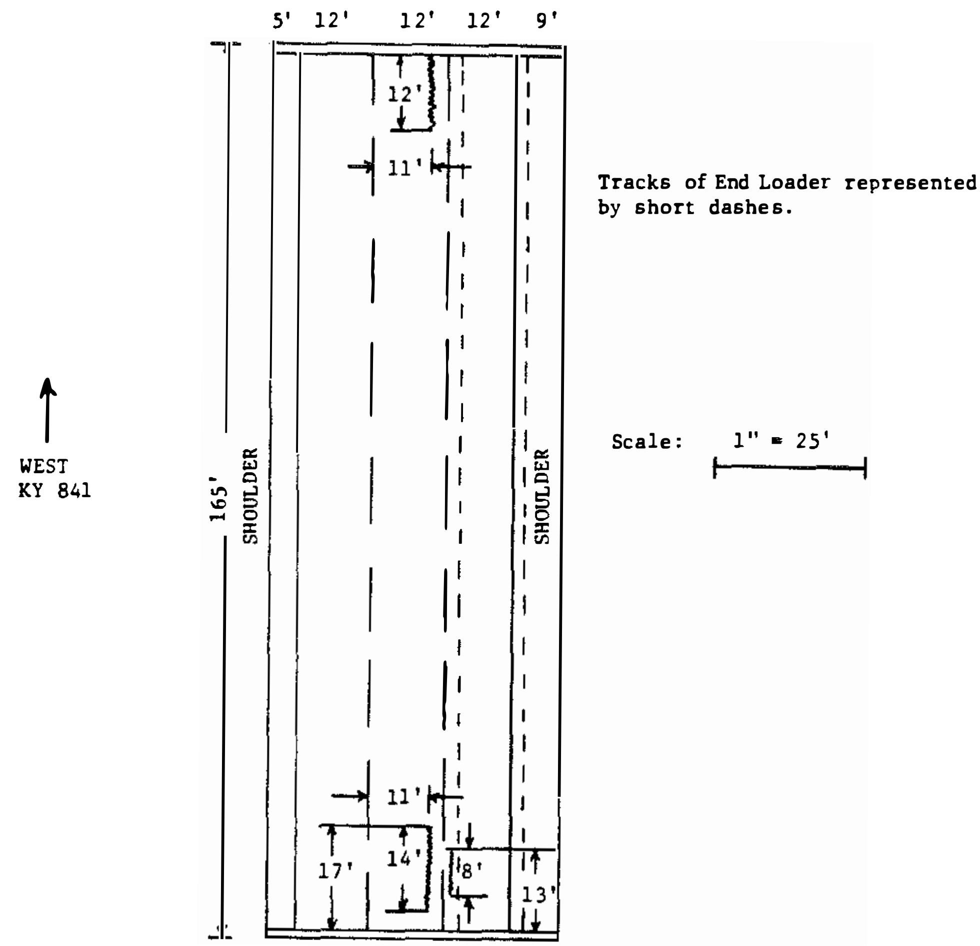


FIGURE 1. Sketch of Traction Cleates Used on Front-End Loader.

BRIDGE: Jefferson Freeway over Blue Lick Drive

TYPE: Three-Span Continuous, Precast, Prestressed Concrete Girder Bridge, Constructed 1980.



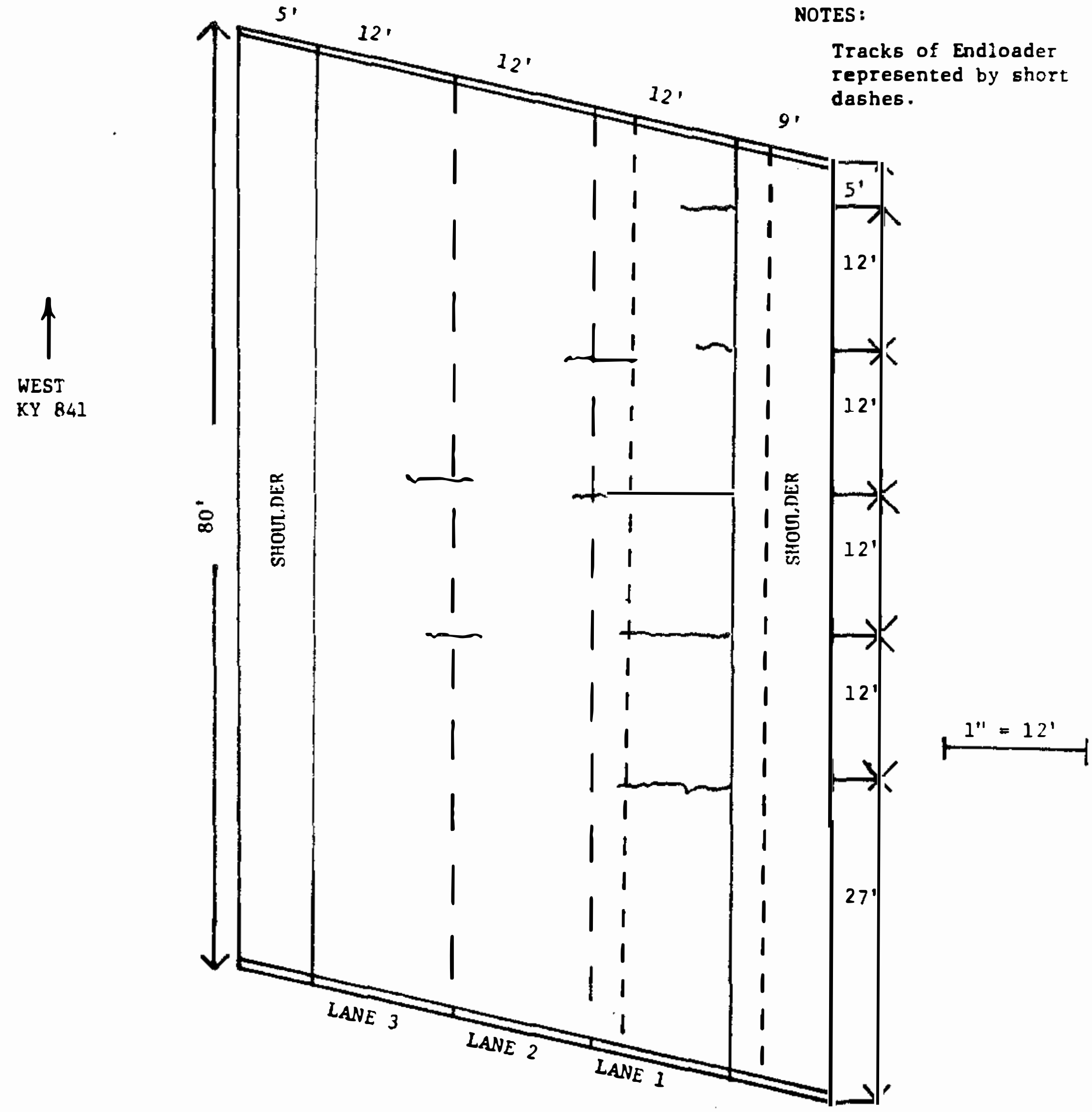
OBSERVATIONS:

Three longitudinal cracks were found on the surface. No transverse cracks were observed. No damage was found at the armoured edges. No new cracks were found to be caused by the Front-End Loader.

FIGURE 2. Field Notes for Deck of Blue Lick Bridge on Jefferson Freeway.

BRIDGE: Jefferson Freeway over Freedom Way

TYPE: Single-Span Steel Girder Bridge on 30° Skew, Constructed 1980



OBSERVATIONS:

No longitudinal cracks were observed. Transverse cracking evident at approximately 12 foot intervals (not parallel with skew). This pattern is consistent with other crack patterns observed on bridges of similar construction. No damage was found at the armoured edges. No new cracks were found to be caused by the Front End-Loader.

FIGURE 3. Field Notes for Deck of Freedom Way Bridge on Jefferson Freeway.

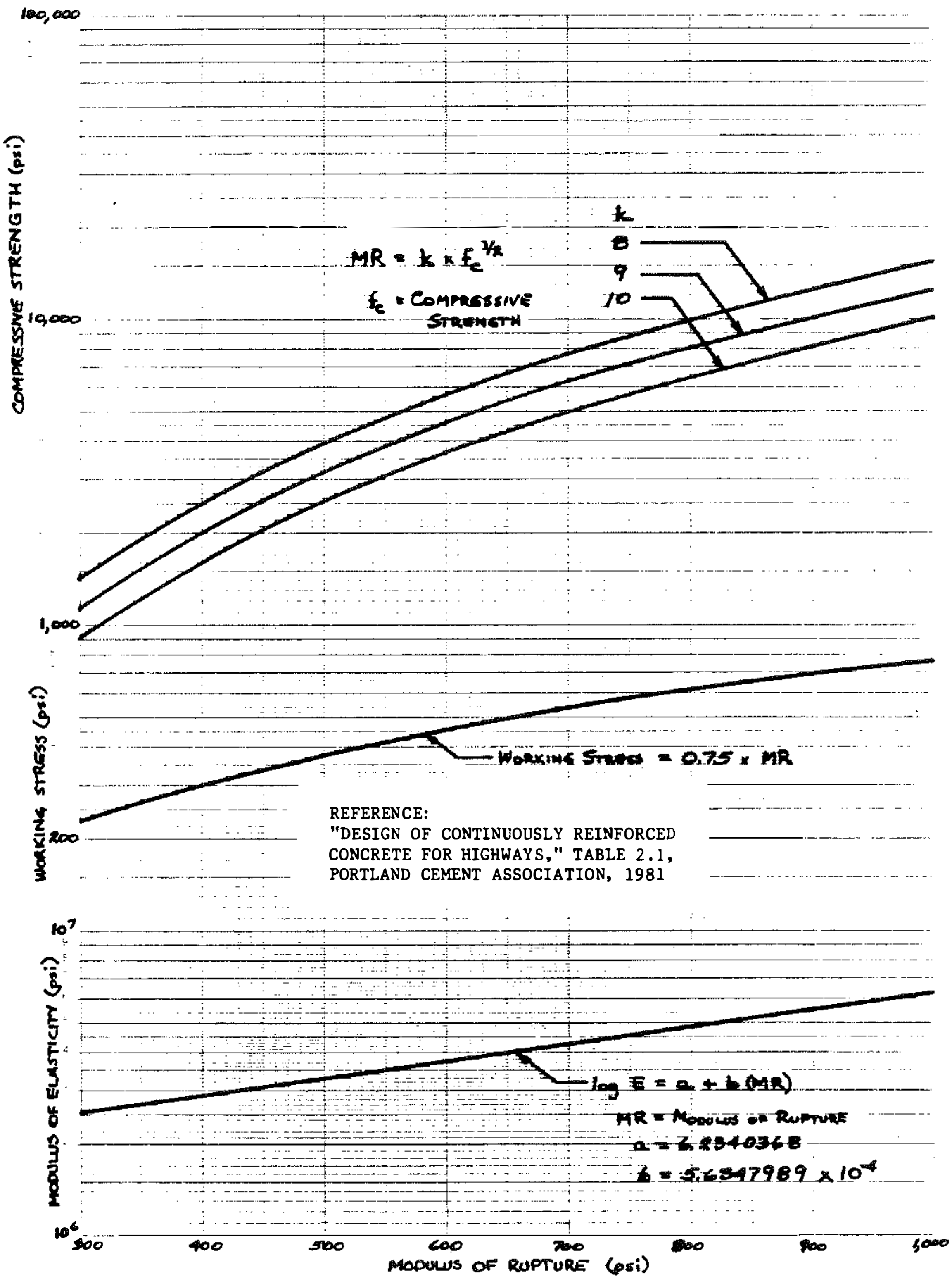


FIGURE 4. Relationships between Modulus of Rupture, Modulus of Elasticity, Working Stress, and Compressive Strength.

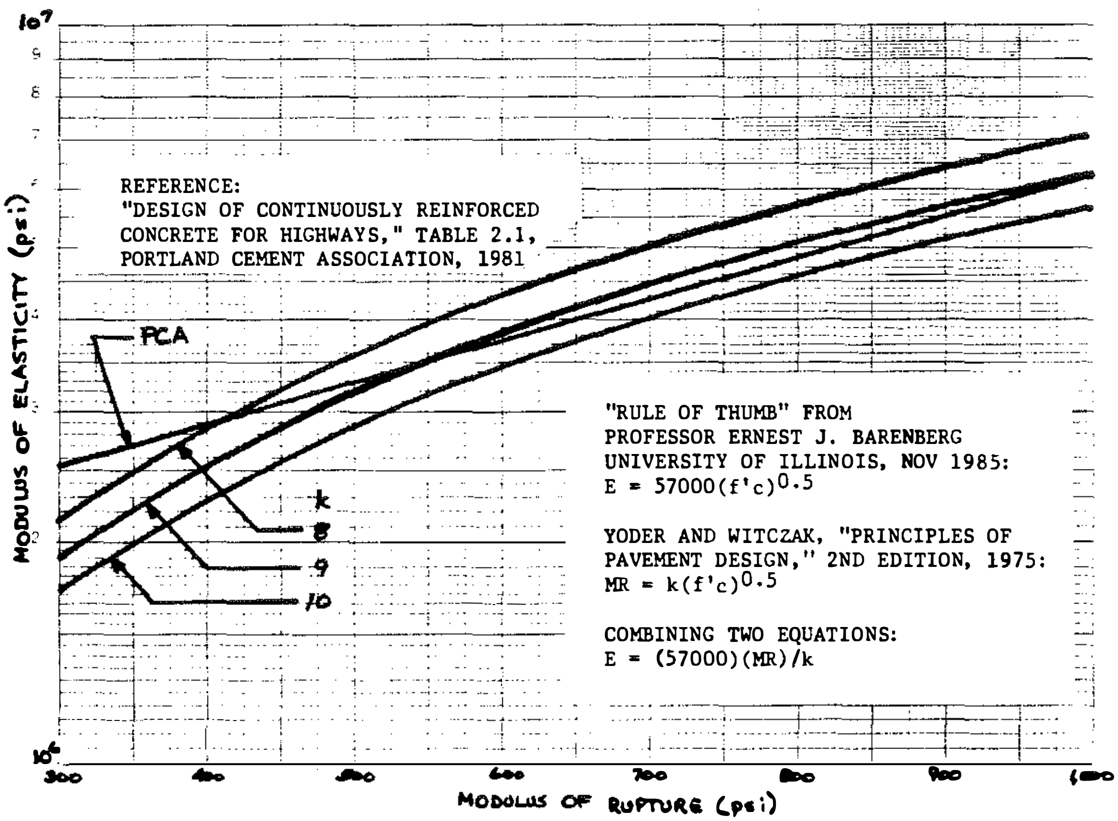


FIGURE 5. Comparison of Two Relationships between Modulus of Rupture and Modulus of Elasticity.

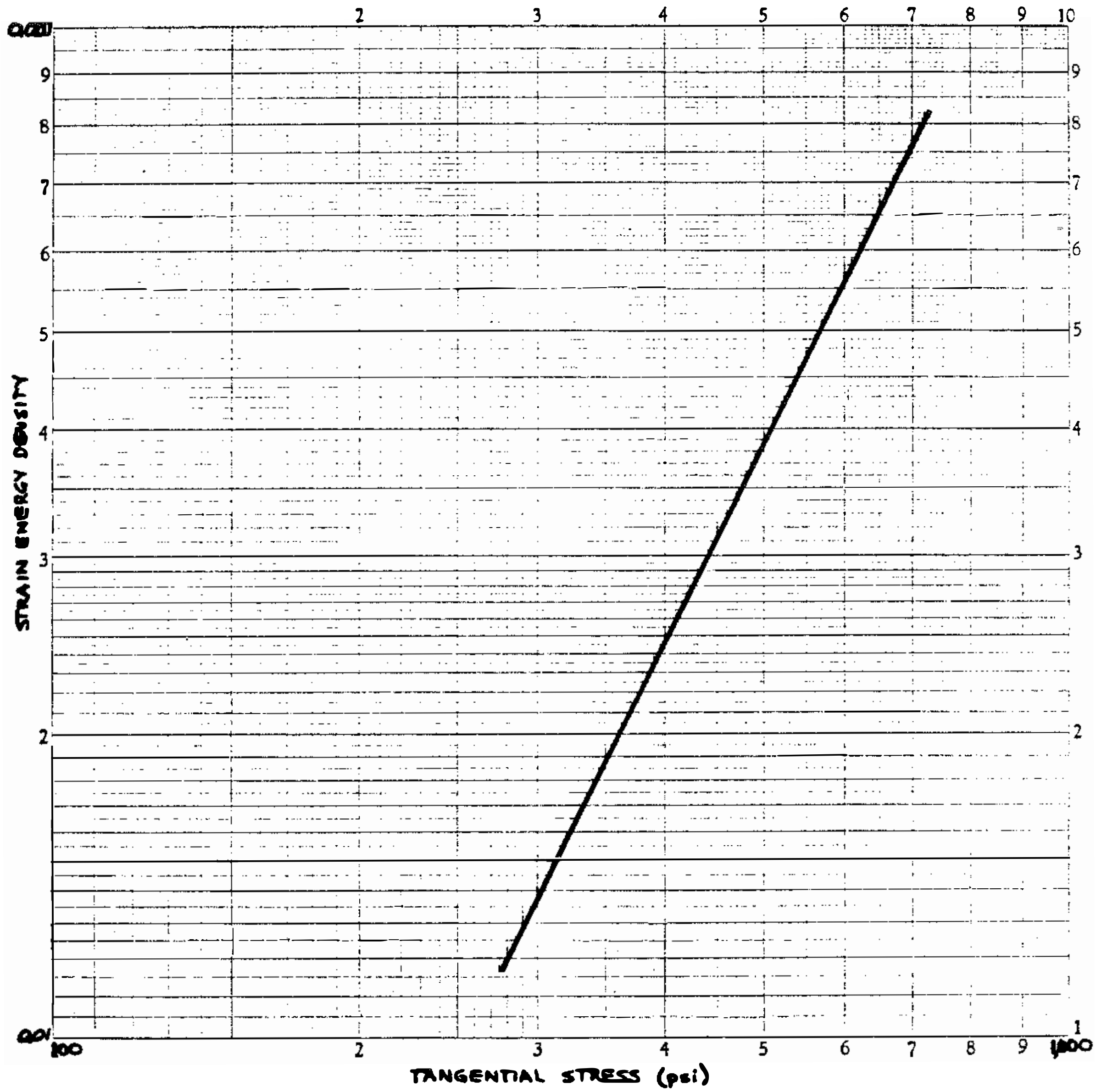


FIGURE 6. Relationship between Tangential Stress and Strain Energy Density.

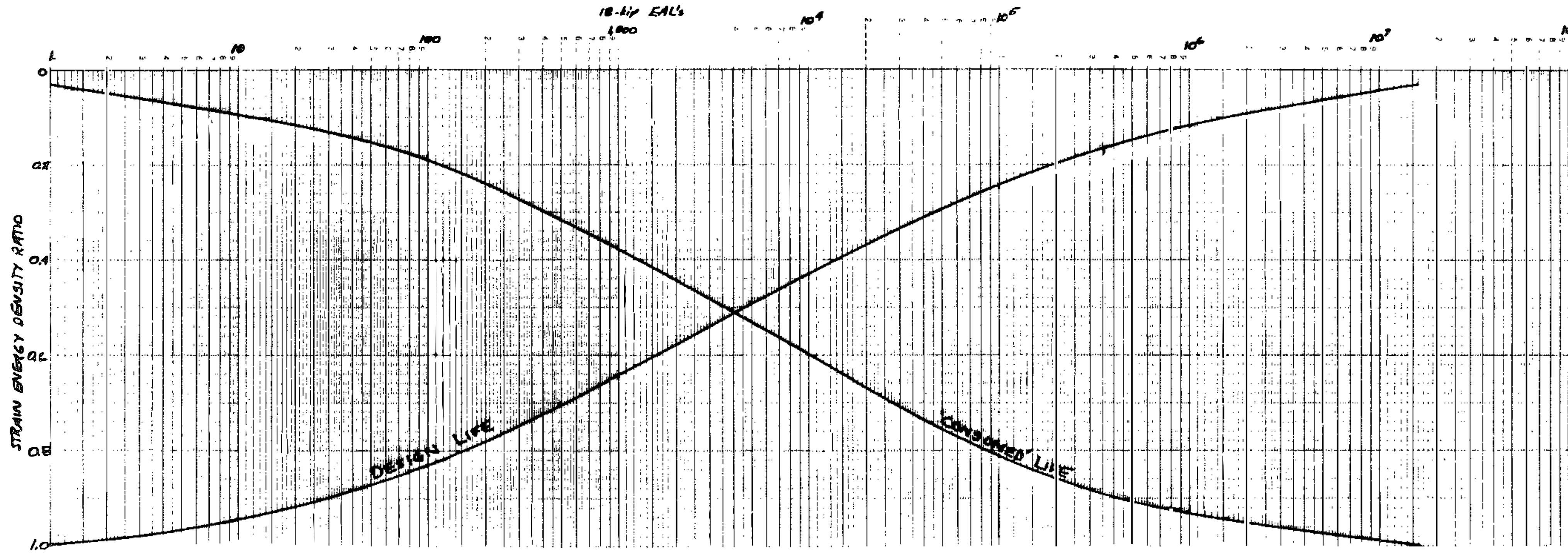


FIGURE 7. Repetitions of 18-kip Equivalent Axleloads as a Function of Strain Energy Density Ratio for Both Design Life and Consumed Life.

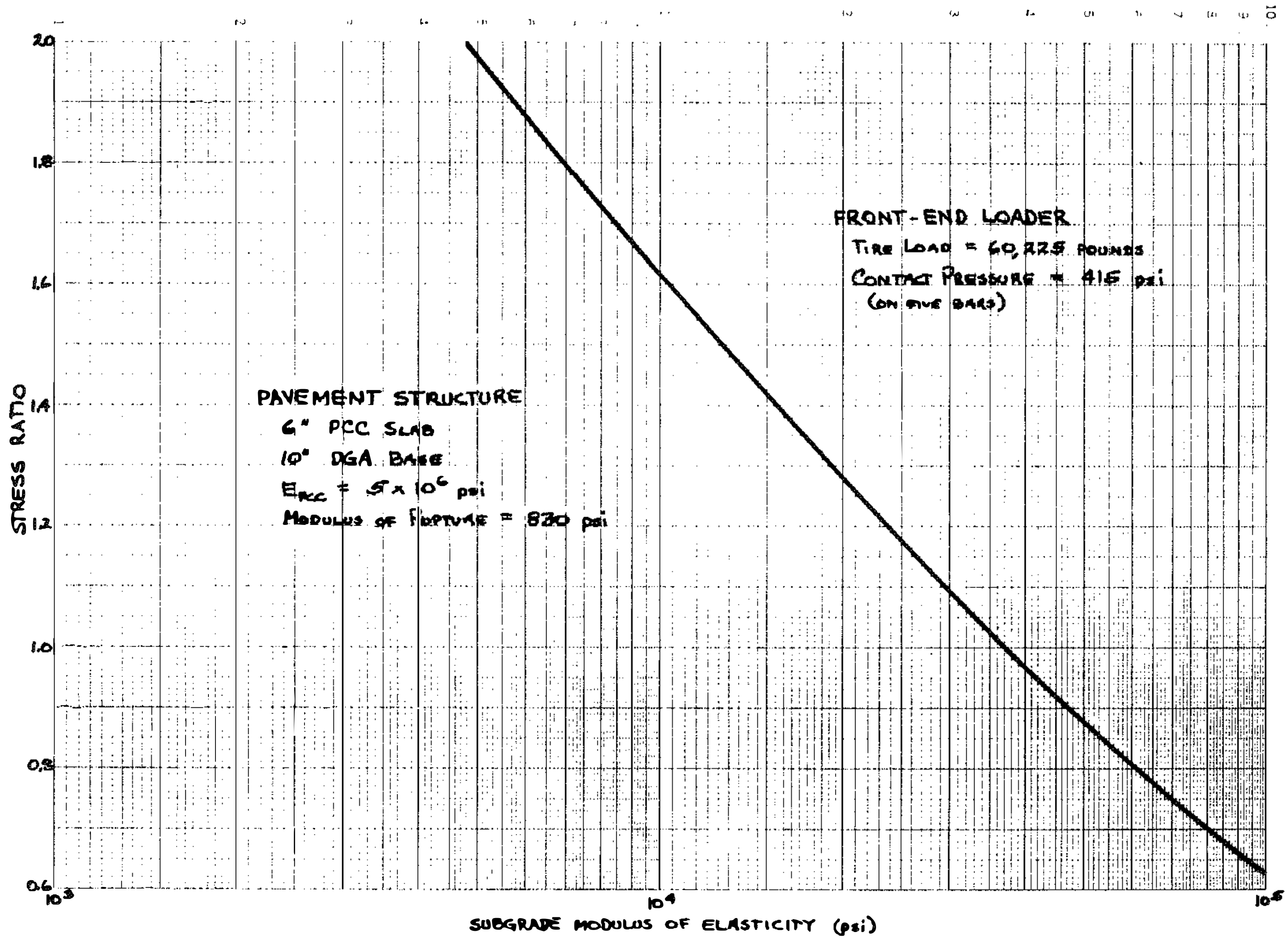


FIGURE 8. Expansion of 5-Million psi Curve from Figure 12.

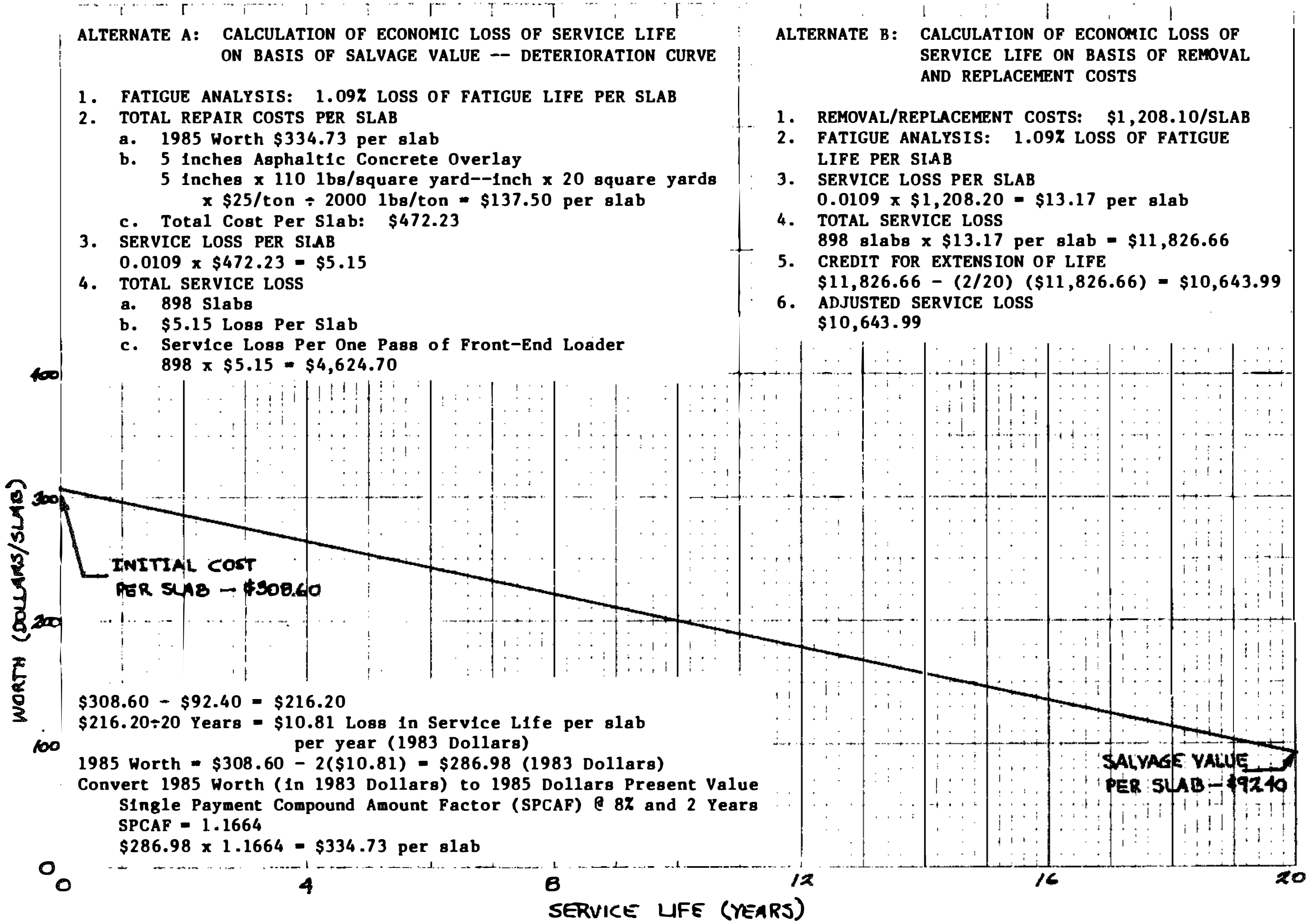


FIGURE 9. Relationships between Worth and Service Life for 10-Inch Portland Cement Concrete Mainline Pavement.

ALTERNATE A: CALCULATION OF ECONOMIC LOSS OF SERVICE LIFE ON BASIS OF SALVAGE VALUE -- DETERIORATION CURVE

1. FATIGUE ANALYSIS: 98% LOSS OF FATIGUE LIFE PER SLAB
2. TOTAL REPAIR COSTS PER SLAB
   a. 1985 Worth $167.51 per slab
   b. 5 inches Asphaltic Concrete Overlay
      5 inches x 110 lbs/square yard--inch x 13.3 square yards x $25/ton ÷ 2000 lbs/ton = $91.67 per slab
   c. Total Cost Per Slab: $259.15
3. SERVICE LOSS PER SLAB
   0.98 x $259.15 = $253.97
4. TOTAL SERVICE LOSS
   a. 367 Slabs
   b. $253.97 Loss Per Slab
   c. Service Loss Per One Pass of Front-End Loader
      367 x $253.97 = $93,206.99

$152.73 - $61.58 = $91.15
$91.15÷20 Years = $4.56 Loss in Service Life per slab per year (1983 Dollars)
1985 Worth = $152.73 - 2($4.56) = $143.61 (1983 Dollars)
Convert 1985 Worth (in 1983 Dollars) to 1985 Dollars Present Value
Single Payment Compound Amount Factor (SPCAF) @ 8% and 2 Years
SPCAF = 1.1664
$143.61 x 1.1664 = $167.51 per slab

ALTERNATE B: CALCULATION OF ECONOMIC LOSS OF SERVICE LIFE ON BASIS OF REMOVAL AND REPLACEMENT COSTS

1. REMOVAL/REPLACEMENT COSTS: $641.66/SLAB
2. FATIGUE ANALYSIS: 98% LOSS OF FATIGUE LIFE PER SLAB
3. SERVICE LOSS PER SLAB
   0.98 x $641.66 = $628.83 per slab
4. TOTAL SERVICE LOSS
   367 slabs x $628.83 per slab = $230,780.61
5. CREDIT FOR EXTENSION OF LIFE
   $230,780.61 - (2/20) ($230,780.61) = $207,702.55
6. ADJUSTED SERVICE LOSS
   $207,702.55

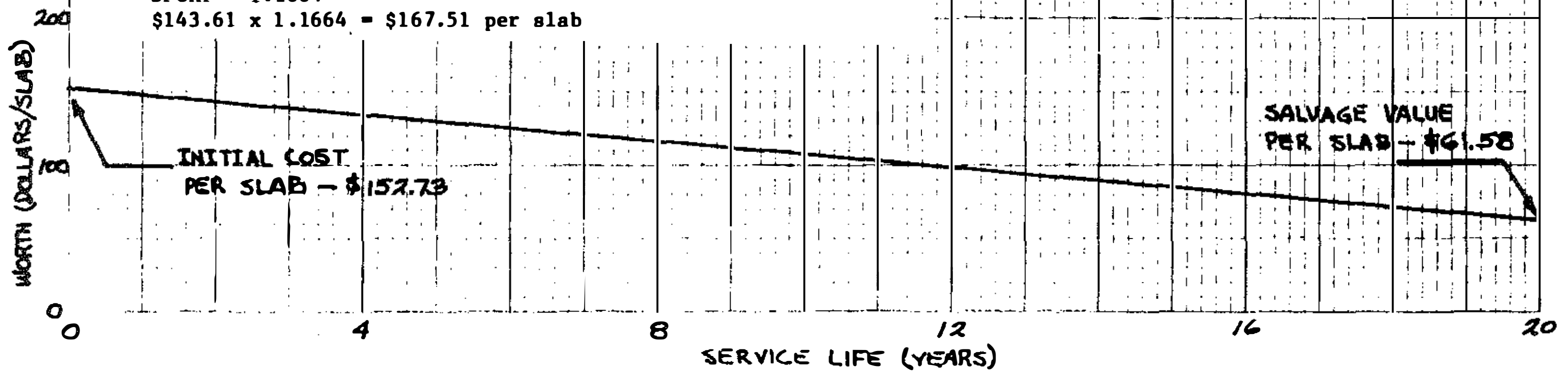


FIGURE 10. Relationship between Worth and Service Life for 6-Inch Portland Cement Concrete Shoulder Pavement.

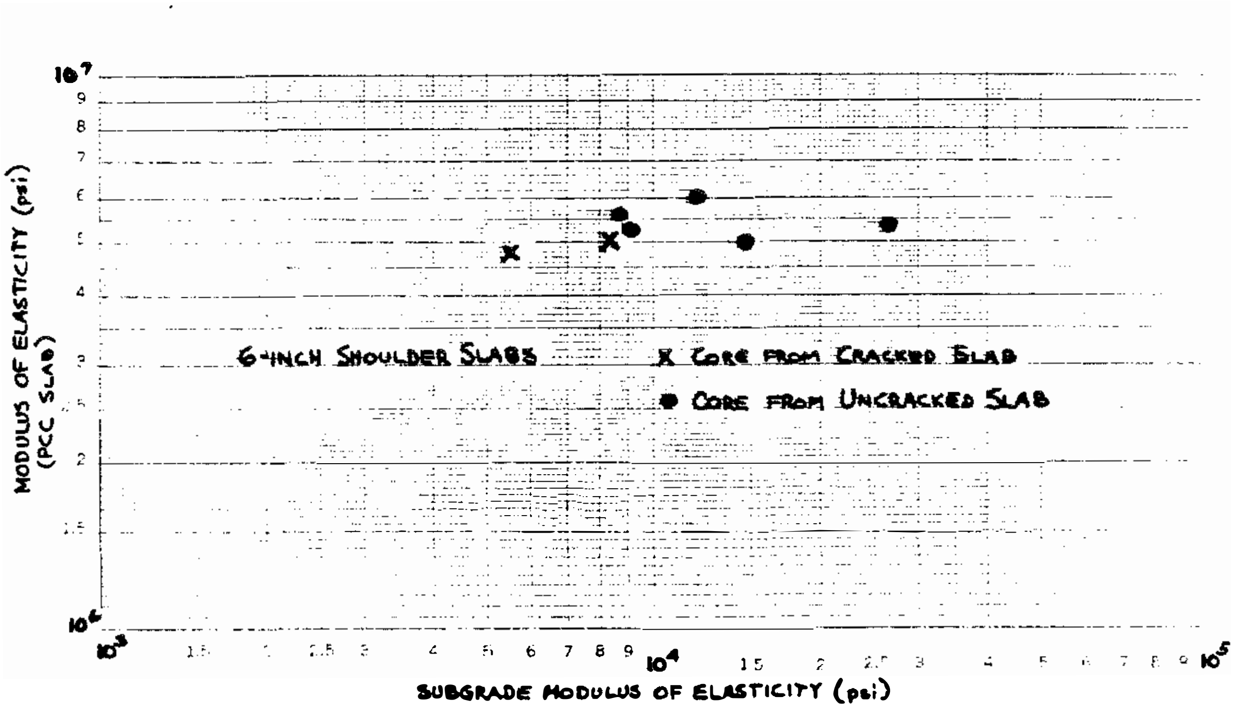


FIGURE 11. Relationships between Moduli of Elasticity for Subgrade and Portland Cement Concrete.

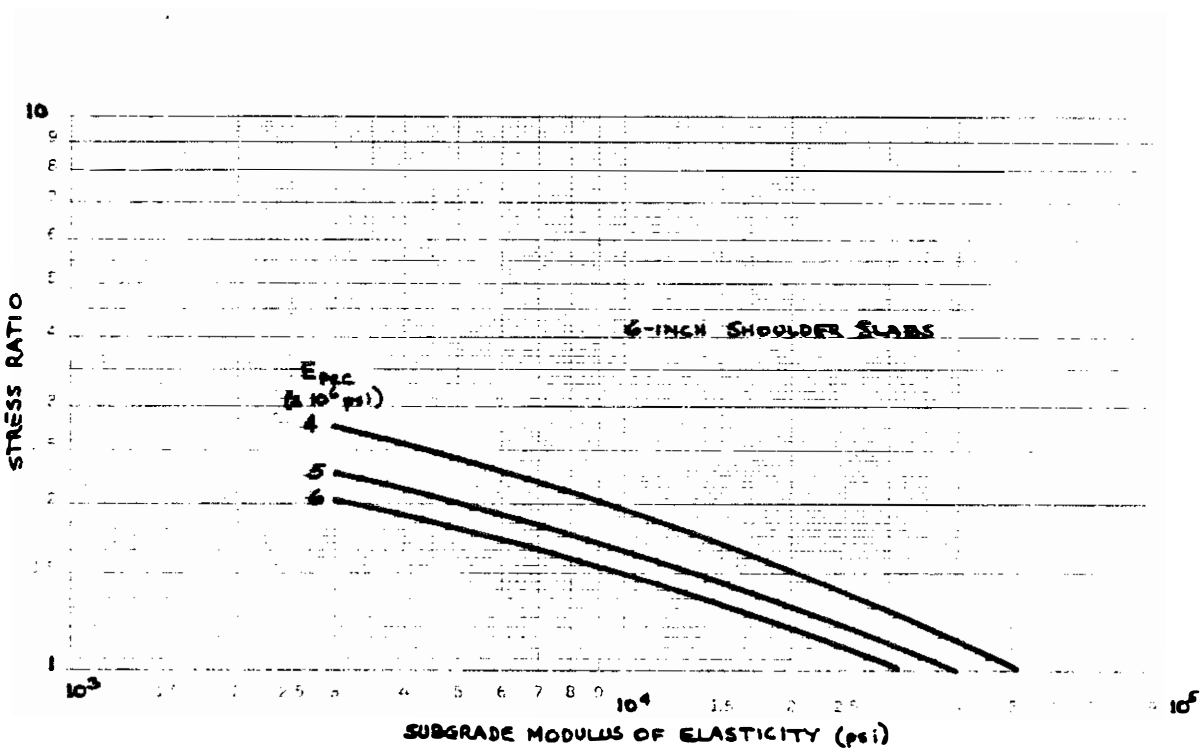


FIGURE 12. Variation between Stress Ratio and Moduli of Elasticity for Subgrade and Portland Cement Concrete.

LIST OF TABLES

TABLE 1. FIELD AND LABORATORY TESTING OF MATERIALS

CORE NO.	SITE	SLAB NO.	ROW	LENGTH OF CORE (inches)	CORE UNIT WEIGHT[a] (pcf)	COMPRESSIVE STRENGTH[a] (psi)	STATIC-CHORD MODULUS OF ELASTICITY[a] ($\times 10^6$ psi)	SONIC MODULUS OF ELASTICITY[b] ($\times 10^6$ psi)	SOIL MOISTURE CONTENT (%)	IN-PLACE CBR
1	A	72	A	6.1	148	10,304	1.91			
2	A	73	A	6.3	145	---	--		6.4	56
3	A	83	A	6.3	146	8,493	1.71			
4	A	83	B	6.5	146	7,313	1.66			
5	A	83	C	10.1	146	7,428	1.93	6.93	21.8	43
6	A	83	D	9.9	148	6,899	1.33			
7	A	83	E	10.0	150	8,615	2.04			
8	B	225	A	7.4	150	7,388	1.58			
9	B	225	B	7.5	147	7,219	1.77			
10	B	225	C	11.0	145	7,314	1.71			
11	B	225	D	11.0	145	7,370	1.97	5.89		
12	B	225	E	10.8	145	7,357	2.20		22.1	7
13	C	307	A	7.0	148	8,903	2.11			
14	C	307	B	6.8	145	6,816	1.90		18.7	
15	C	307	C	10.4	147	8,015	1.75	6.14		
16	C	307	D	10.5	149	6,883	1.98			
17	C	307	E	10.5	148	6,193	1.81			
Mean					147.1	7,657	1.84	6.32		
Standard Deviation					1.7	1,002	0.22	0.54		

[a] ASTM C 469-65

[b] ASTM C 215-60 (Determination by fundamental transverse vibration)

TABLE 2. SUMMARY OF DEFLECTION MEASUREMENTS AT CORE LOCATIONS

SITE	ROW	COMPRESSIVE STRENGTH OF CORE (psi)	THICKNESS OF CORE (inches)	ROAD RATER DEFLECTIONS (inches x 10^{-5}) SENSORS No.1	No.2	No.3	No.4
MAINLINE PAVEMENT AND RAMP PAVEMENT							
NOMINAL THICKNESS: 10 INCHES PCC; 6 INCHES DGA							
A	C	7,428	10.1	32	24	-	16
A	D	6,899	9.9	14	11	8	12
A	E	8,615	10.0	16	13	6	6
B	C	7,314	11.0	30	34	32	31
B	D	7,370	11.0	16	16	12	-
B	E	7,357	10.8	16	16	12	12
C	C	8,015	10.4	30	26	22	20
C	D	6,883	10.5	26	24	19	14
C	E	6,193	10.5	24	21	16	-
Mean		7,342	10.5	22.6	20.6	15.9	15.9
Standard Deviation		690	0.4	7.2	7.3	8.4	7.9
SHOULDER PAVEMENT							
NOMINAL THICKNESS: 6 INCHES PCC; 10 INCHES DGA							
A	A	10,304	6.1	42	42	35	26
A	A	8,493	6.3	37	36	30	24
A	B	7,313	6.5	30	24	14	11
B	A	7,388	7.4	53	51	40	36
B	B	7,219	7.5	54	53	-	40
C	A	8,903	7.0	61	52	46	31
C	B	6,816	6.8	85	76	60	50
Mean		8,062	6.8	51.7	47.7	37.5	31.1
Standard Deviation		1,238	0.54	18.2	16.3	15.5	12.5

TABLE 3. SUMMARY OF PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF A RANGE OF ELASTIC MODULI FOR PORTLAND CEMENT CONCRETE

		ROAD RATER DEFLECTIONS (inches x 10^{-5}) SENSORS				PREDICTED SUBGRADE MODULUS (psi) ASSUMED MODULUS FOR PCC (psi)		
SITE	ROW	No.1	No.2	No.3	No.4	$2.0x10^6$	$4.8x10^6$	$6.0x10^6$
MAINLINE PAVEMENT AND RAMP PAVEMENT								
NOMINAL THICKNESS: 10 INCHES PCC; 6 INCHES DGA								
A	C	32	24	-	16	17,692	13,743	12,832
A	D	14	11	8	12	46,341	36,684	34,406
A	E	16	13	6	6	55,352	46,294	44,009
B	C	30	34	32	31	12,007	9,013	8,344
B	D	16	16	12	-	35,696	27,019	25,086
B	E	16	16	12	12	33,398	26,156	24,482
C	C	30	26	22	20	16,271	12,537	11,339
C	D	26	24	19	14	20,312	15,933	14,907
C	E	24	21	16	-	23,225	17,528	16,244
Mean		22.6	20.6	15.9	15.9	28,922	22,767	21,294
Standard Deviation		7.2	7.3	8.4	7.9	14,776	12,380	11,817
SHOULDER PAVEMENT								
NOMINAL THICKNESS: 6 INCHES PCC; 10 INCHES DGA								
A	A	42	42	35	26	14,629	11,931	11,285
A	A	37	36	30	24	17,132	14,017	13,267
A	B	30	24	14	11	30,901	26,862	25,836
B	A	53	51	40	36	11,255	9,093	8,578
B	B	54	53	-	40	10,695	8,395	7,871
C	A	61	52	46	31	10,583	8,632	8,163
C	B	85	76	60	50	6,907	5,527	5,200
Mean		51.7	47.7	37.5	31.1	14,535	12,065	11,457
Standard Deviation		18.2	16.3	15.5	12.5	7,895	7,066	6,846

TABLE 4. PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF CORE STRENGTHS AND ELASTIC MODULI

SITE	ROW	ROAD RATER DEFLECTIONS (inches x 10^{-5}) SENSORS No.1	No.2	No.3	No.4	ELASTIC[1] MODULI FOR PCC CORES (psi)	PREDICTED[1A] SUBGRADE MODULI (psi)	STATIC-CHORD[2] ELASTIC MODULI FOR PCC CORES (psi)	PREDICTED[2A] SUBGRADE MODULI (psi)	ELASTIC MODULI[3] BY FUNDAMENTAL FREQUENCY FOR PCC CORES (psi)	PREDICTED[3A] SUBGRADE MODULI (psi)
MAINLINE PAVEMENT AND RAMP PAVEMENT NOMINAL THICKNESS: 10 INCHES PCC, 6 INCHES DGA											
A	C	32	24	-	16	5.0×10^6	13,500	1.9×10^6	18,000	6.9×10^6	12,500
A	D	14	11	8	12	4.9×10^6	36,000	1.3×10^6	43,000		
A	E	16	13	6	6	5.6×10^6	44,000	2.0×10^6	55,500		
B	C	30	34	32	31	4.9×10^6	9,000	1.7×10^6	12,750		
B	D	16	16	12	-	4.9×10^6	27,000	2.0×10^6	36,000	5.9×10^6	25,250
B	E	16	16	12	12	4.9×10^6	26,000	2.2×10^6	33,000		
C	C	30	26	22	20	5.3×10^6	12,000	1.8×10^6	17,000	6.1×10^6	11,400
C	D	26	24	19	14	5.0×10^6	10,500	2.0×10^6	24,000		
C	E	24	21	16	-	4.7×10^6	18,000	1.8×10^6	24,500		
Mean		22.6	20.6	15.9	15.9	5.0×10^6	21,778	1.9×10^6	29,306	6.3×10^6	16,383
Standard Deviation		7.2	7.3	8.4	7.9	0.3×10^6	12,299	0.3×10^6	13,860	0.5×10^6	7,698
SHOULDER PAVEMENT NOMINAL THICKNESS: 6 INCHES PCC, 10 INCHES DGA											
A	A	42	42	35	26	6.0×10^6	11,300	1.9×10^6	14,750		
A	A	37	36	30	24	5.4×10^6	13,500	1.7×10^6	18,000		
A	B	30	24	14	11	5.0×10^6	26,500	1.7×10^6	32,000		
B	A	53	51	40	36	5.2×10^6	9,000	1.6×10^6	12,000		
B	B	54	53	-	40	5.0×10^6	8,400	1.8×10^6	10,000		
C	A	61	52	46	31	5.6×10^6	8,200	2.1×10^6	10,500		
C	B	85	76	60	50	4.8×10^6	5,500	1.9×10^6	7,000		
Mean		51.7	47.7	37.5	31.1	5.3×10^6	11,771	1.8×10^6	14,893		
Standard Deviation		18.2	16.3	15.5	12.5	0.4×10^6	6,967	0.2×10^6	8,329		

1 $E_{pcc} = 33 \times (\text{unit weight})^{3/2} \times (fc)^{1/2}$

1A Predicted Subgrade Modulus Determined on Basis of Road Rater Deflections and E_{pcc}[1]

2 E_{pcc} in Accordance with ASTM C 469 -- Static Chord Method

2A Predicted Subgrade Modulus Determined on Basis of Road Rater Deflections and E_{pcc}[2]

3 E_{pcc} in Accordance with ASTM C 215 -- Fundamental Frequency Method

3A Predicted Subgrade Modulus Determined on Basis of Road Rater Deflections and E_{pcc}[3]

TABLE 5. SUMMARY OF PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF A RANGE OF ELASTIC MODULI FOR PORTLAND CEMENT CONCRETE (MAINLINE PAVEMENT AND RAMP PAVEMENT NOMINAL THICKNESS: 10 INCHES PCC; 6 INCHES DGA)

				PREDICTED SUBGRADE MODULUS (psi)		
				ASSUMED ELASTIC MODULUS FOR PORTLAND CEMENT CONCRETE		
SITE	LANE	ROW	LOCATION	$2.0x10^6$Psi	$4.8x10^6$Psi	$6.0x10^6$Psi
A	1	C	Midslab	33,867	26,665	24,984
A	1	D	Midslab	58,304	47,362	44,719
A	1	E	Midslab	58,722	48,354	45,815
Mean				50,298	40,794	38,506
Standard Deviation				14,231	12,246	11,723
A	Taper	F	Midslab	56,774	46,278	43,732
A	2	G	Midslab	44,505	35,764	33,697
A	2	H	Midslab	69,016	55,736	52,592
A	2	I	Midslab	63,388	51,450	48,607
Mean				58,970	47,650	44,965
Standard Deviation				12,840	10,514	9,960
A	3	J	Midslab	64,108	52,293	49,442
A	3	K	Midslab	58,838	48,195	45,605
A	3	L	Midslab	50,284	40,238	37,870
Mean				57,743	46,909	44,306
Standard Deviation				6,977	6,130	5,894
A	1	C	Third Point	28,607	22,411	20,967
A	1	D	Third Point	57,199	46,202	43,570
A	1	E	Third Point	53,585	43,483	41,062
Mean				46,464	37,365	35,200
Standard Deviation				15,570	13,022	12,389
OVERALL, SITE A						
Mean				53,631	43,418	40,974
Standard Deviation				11,728	9,850	9,385
BY LANE, SITE A						
A	1		Midslab	50,298	40,794	38,506
A	1		Third Point	46,464	37,365	35,200
A	Taper		Midslab	56,774	46,278	43,732
A	2		Midslab	58,970	47,650	44,965
A	3		Midslab	57,743	46,909	44,306
Mean				54,050	43,799	41,342
Standard Deviation				5,406	4,502	4,283

TABLE 6. SUMMARY OF PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF A RANGE OF ELASTIC MODULI FOR PORTLAND CEMENT CONCRETE (SHOULDER PAVEMENT NOMINAL THICKNESS: 6 INCHES PCC; 10 INCHES DGA)

				PREDICTED SUBGRADE MODULUS (psi)		
				ASSUMED ELASTIC MODULUS FOR PORTLAND CEMENT CONCRETE		
SITE	LANE	ROW	LOCATION	2.0×10^6 psi	4.8×10^6 psi	6.0×10^6 psi
A		A	Midslab	22,676	19,075	18,188
A		B	Midslab	27,738	23,798	22,811
Mean				25,207	21,437	20,500
Standard Deviation				3,579	3,340	3,269
A		A	Third Point	25,026	21,097	20,133
A		B	Third Point	26,513	22,507	21,507
Mean				25,770	21,802	20,820
Standard Deviation				1,051	997	972
OVERALL, SITE A Mean				25,488	21,619	20,660
Standard Deviation				2,178	2,023	1,978

TABLE 7. SUMMARY OF PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF A RANGE OF ELASTIC MODULI FOR PORTLAND CEMENT CONCRETE (MAINLINE PAVEMENT AND RAMP PAVEMENT NOMINAL THICKNESS: 10 INCHES PCC; 6 INCHES DGA)

				PREDICTED SUBGRADE MODULUS (psi)		
				ASSUMED ELASTIC MODULUS FOR PORTLAND CEMENT CONCRETE		
SITE	LANE	ROW	LOCATION	2.0×10^6 psi	4.8×10^6 psi	6.0×10^6 psi
B	1	C	Midslab	17,151	13,197	12,290
B	1	D	Midslab	30,706	24,046	22,503
B	1	E	Midslab	23,563	18,213	16,986
Mean				23,807	18,485	17,260
Standard Deviation				6,781	5,430	5,112
B	2	F	Midslab	14,490	11,125	10,350
B	2	G	Midslab	22,917	18,184	17,051
B	2	H	Midslab	16,852	13,046	12,164
Mean				18,086	14,118	13,188
Standard Deviation				4,347	3,650	3,466
B	3	I	Midslab	17,687	13,673	12,745
B	3	J	Midslab	21,747	17,052	15,951
B	3	K	Midslab	16,394	12,627	11,755
Mean				18,609	14,451	13,484
Standard Deviation				2,793	2,313	2,193
OVERALL, SITE B						
Mean				20,167	15,695	14,644
Standard Deviation				5,067	4,053	3,822
BY LANE, SITE B						
B	1			23,807	18,485	17,260
B	2			18,086	14,118	13,188
B	3			18,609	14,451	13,484
Mean				20,167	15,685	14,644
Standard Deviation				3,163	2,431	2,270

TABLE 8. SUMMARY OF PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF A RANGE OF ELASTIC MODULI FOR PORTLAND CEMENT CONCRETE (SHOULDER PAVEMENT NOMINAL THICKNESS: 6 INCHES PCC; 10 INCHES DGA)

				PREDICTED SUBGRADE MODULUS (psi)		
				ASSUMED ELASTIC MODULUS FOR PORTLAND CEMENT CONCRETE		
SITE	LANE	ROW	LOCATION	2.0×10^6 psi	4.8×10^6 psi	6.0×10^6 psi
B	Shoulder	A	Midslab	12,537	10,246	9,695
B	Shoulder	B	Midslab	13,714	11,232	10,636
Mean				13,126	10,739	10,166
Standard Deviation				832	697	665
C	Shoulder	A	Midslab	10,791	8,787	8,306
C	Shoulder	B	Midslab	8,448	6,810	6,420
Mean				9,620	7,799	7,363
Standard Deviation				1,657	1,398	1,334

TABLE 9. SUMMARY OF PREDICTED SUBGRADE MODULI DETERMINED ON THE BASIS OF A RANGE OF ELASTIC MODULI FOR PORTLAND CEMENT CONCRETE (MAINLINE PAVEMENT AND RAMP PAVEMENT NOMINAL THICKNESS: 10 INCHES PCC; 6 INCHES DGA)

				PREDICTED SUBGRADE MODULUS (psi)		
				ASSUMED ELASTIC MODULUS FOR PORTLAND CEMENT CONCRETE		
SITE	LANE	ROW	LOCATION	2.0×10^6psi	4.8×10^6psi	6.0×10^6psi
C	1	C	Midslab	16,264	12,487	11,620
C	1	D	Midslab	24,038	18,877	17,667
C	1	E	Midslab	22,024	17,121	15,981
Mean				20,775	16,162	15,089
Standard Deviation				4,035	3,301	3,121

TABLE 10. RESULTS OF INSPECTIONS OF CORES TAKEN AT TRANSVERSE JOINTS

SITE	CORE NO.	SLAB OR LANE NO.	JOINT NO.	DISTANCE FROM EDGE	DISTANCE FROM JOINT	LENGTH OF CRUSHED JOINT	DEPTH OF HOLE	VOIDS UNDER SLAB	CRACKING: HORIZONTAL	CRACKING: VERTICAL DISTANCE FROM JOINT	CRACKING: DIAGONAL DEPTH OF CRACK	CRUSHING AT BOTTOM OF SAWN CUT
JOINTS												
A	1A	1	74	54"	0"	17"	10 13/16"	No	At Dowel	7/16"	3 1/2"	Some
	2A	1	81	67 1/2"	0"	27"	10 3/4"	No	At Dowel	1 1/4"	6 3/8"	Some
	3A	1	75	66"	0"	5"	10 5/8"[a]	No	At Dowel	7/16"	1 1/2"	None
	1A-A	3	74	48"	0"	0"	10 7/8"	No	None	N/A	N/A	N/A
	3A-A	3	75	66"	0"	0"	11"	No	At Dowel	N/A	N/A	N/A
B	1A	1	220	102"	0"	32"	10 3/4"	No	At Dowel	1 11/16"	5 1/8"	A lot
	2A	1	222	78"	0"	6"	11 1/4"	No	At Dowel	1 1/8"	2 3/16"	Some
	3A	1	226	96"	0"	30"	11"	No	At Dowel	5/8"	2 7/16"	Some
	B1-A	3	220	30"	0"	0"	10 1/4"	No	At Dowel	N/A	N/A	N/A
	B2-2A	3	2	32"	0"	0"	10 7/16"	No	At Dowel	N/A	N/A	N/A
SLABS												
A	1	1	73	60"	6"	0"	10 11/16"	No	None	N/A	N/A	N/A
	2	1	80	72"	6"	0"	10 7/16"	No	None	N/A	N/A	N/A
	2B	1	80	78"	6"	0"	10 7/16"	No	None	N/A	N/A	N/A
	3	1	74	53 1/4"	6"	0"	10 1/2"[b]	No	None	N/A	N/A	N/A
B	1	1	219	90"	6"	0"	10 3/8"[b]	No	None	N/A	N/A	N/A
	3	1	225	90"	6"	0"	11"	No	At Dowel	N/A	N/A	N/A

[a] - Due to washing during coring
[b] - Core broke near bottom

TABLE 11. SUMMARY OF OBSERVED DISTRESSES OR DAMAGE

SECTION	LOCATION	LONGITUDINAL JOINTS SHATTERED (feet)	LONGITUDINAL CRACKS (number)	TRANSVERSE CRACKS (number)	TRANSVERSE JOINTS SHATTERED (feet)	CORNER/DIAGONAL CRACKS (number)
EXIT RAMP @ PRESTON HIGHWAY (NORTH) TO EASTBOUND JEFFERSON FREEWAY (UNPAVED PORTION) WAY (UNPAVED PORTION)						
A	Ramp	0	0	0	53	0
A	Shoulder	0	0	0	22	3
JEFFERSON FREEWAY (WEST) TO I-65 NORTH						
I	Mainline	15	0	1	366	31
I	Ramp	0	0	0	33	1
I	Shoulder	14	52	1	146	21
RAMP (EASTBOUND) I-65 NORTH TO OUTER LOOP ROAD						
B	Ramp	405	0	0	139	37
SUBSEQUENT OBSERVATIONS (11-18-85)						
I	Shoulder	0	4	0	0	0
Summary	Mainline/Ramp	420	0	1	591	69
Summary	Shoulder	14	56	1	168	24

TABLE 12. CALCULATION OF INITIAL CONSTRUCTION, SALVAGE VALUE, AND CURRENT REPLACEMENT COSTS -- 10-INCH PCC MAINLINE PAVEMENT

A. 1983 PAVEMENT WORTH: PROJECT I265-1(6) -- UNIT COSTS

1. Dense-Graded Limestone Aggregate $5.25/ton
2. 6-inch PCC Shoulder $8.57/square yard
3. 10-inch PCC Pavement $13.70/square yard
4. Dowels, Tiebars incidental to construction
5. 10-inch PCC Pavement Slab -- Initial Construction Cost
   a. 12 feet x 15 feet x = 180 square feet = 20 square yards
   b. $13.70/square yard x 20 square yards = $274.00/slab
   c. 6 inches DGA x 110 lbs/square yard-inch ÷ 2,000 lbs/ton x 5.25/ton = $1.73/square yard
   d. 12 feet x 15 feet = 180 square feet = 20 square yards
   e. $1.73/square yard x 20 square yards = $34.60/slab
   f. TOTAL INITIAL CONSTRUCTION COST = $308.60/SLAB

B. SALVAGE VALUE -- 16 INCHES DGA

1. 20 square yards
2. 16 inches DGA x 110 lbs/square yard-inch x $5.25/ton ÷ 2,000 lbs/ton x 20 square yards = $92.40/slab

C. CURRENT REMOVAL AND REPLACEMENT COSTS

1. 20 square yards x $50.48/square yard = $1,009.60/slab
2. 6 1-inch dia transverse tiebars @ $10.50 each = $ 63.00/slab
3. 12 1 1/4-inch dia load transfer dowel bars @ $11.30 each = $ 135.60/slab
4. TOTAL COST/SLAB = $1,208.20

TABLE 13. CALCULATION OF INITIAL CONSTRUCTION, SALVAGE VALUE AND CURRENT REPLACEMENT COSTS -- 6-INCH PCC SHOULDER PAVEMENT

A. 1983 PAVEMENT WORTH: PROJECT I265-1(6) -- UNIT COSTS

1. Dense-Graded Limestone Aggregate $5.25/ton
2. 6-inch PCC Shoulder $8.57/square yard
   a. 8 feet x 15 feet = 120 square feet = 13.33 square yards
   b. $8.57/square yard x 13.33 square yards = $114.24/slab
   c. 10 inches DGA x 110 lbs/square yard-inch ÷ 2000 lbs/ton x 5.25/ton x 13.33 square yard = $38.49
   d. TOTAL = $152.73

B. SALVAGE VALUE -- 16 INCHES DGA

1. 13.33 square yards
2. 16 inches DGA x 110 lbs/square yard-inch x $5.25/ton x 13.33 square yards ÷ 2,000 lbs/ton = $61.58/slab
3. TOTAL = $61.58/slab

C. CURRENT REMOVAL AND REPLACEMENT COSTS

1. 13.33 square yards x $43.41/square yard = $578.66/slab
2. 6 1-inch dia transverse tiebars @ $10.50 each = $63.00
3. TOTAL = $641.66