COMPARISON OF ROLLER COMPACTED CONCRETE PAVEMENT RESPONSES TO STATIC AND DYNAMIC LOADS

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ABSTRACT

The objective of the study was to compare Roller Compacted Concrete (RCC) pavements responses to static and dynamic loads. Two sets of non-destructive load testing were performed on RCC pavements. The first set used static loading and dynamic loading was utilized for the second. Static loading was provided by front wheels of a Letro-Porter (front loader) and a Heavy Weight Deflectometer (HWD) was used for dynamic loading. A back-calculation procedure for estimating concrete pavement parameters is also presented.

The Roller Compacted Concrete pavements were located at Moran Terminal, Boston, Massachusetts. The pavement consisted of a RCC layer of 38 cm (15 in.), a 23-cm (9-in.) gravel subbase and a compacted subgrade. The RCC layer was constructed in three lifts from bottom to top of 14, 14, and 10 cm (5.5, 5.5 and 4 in.). Four slabs were selected for testing. Strains and deflections were measured for static loading tests. A Benkelman beam was used for measuring the static load induced deflections. Deflection basins were determined during HWD tests. Load transfer efficiency at cracks and construction joints was also measured for both tests.

The back-calculation procedure has been demonstrated to be satisfactory in estimating in situ pavement material properties. Using estimated RCC elastic moduli and estimated moduli of subgrade reaction, the stresses in the slabs caused by the static loads were computed by the finite element computer program JSLAB and compared to field measured stresses. It was found that the computed stresses were always greater than the measured values. A factor of safety of about 4% can be expected when HWD deflection data are used in place of static vehicle test data in rigid pavement analysis and design.

INTRODUCTION

Nondestructive dynamic surface loading tests have been used for structural evaluation of pavements for many years. Basically, they involve delivering dynamic loads, either steady-state or impulsive, to a pavement surface and measuring the induced deflections using deflection sensors placed at different locations. The measured deflection basins are then used to ascertain the pavement's structural capacity. These tests have gained great popularity in the past few decades because of their ease of use and their ability to collect a great amount of data in a short period of time.

In the past twenty years, many procedures have been developed for back-calculating the in situ pavement properties by matching the measured and theoretically computed deflection basins. Although deflections were generated by dynamic loads, most of the methods utilized static analyses.[1-7] Therefore, concern arises as to how the dynamic nature of the load would affect the pavement deflections and the estimated pavement properties. Roesset and Shao[8] compared the deflections obtained from dynamic analyses with those provided by static analyses. They concluded that a static interpretation of the measured deflections in the dynaflect tests might be reasonable for a pavement with homogeneous foundation, and that dynamic effects were less important for the falling weight deflectometer tests. Based on their study using an elastodynamic model, Davies and Mamlouk[9] stated that static analyses of dynamic deflections on a pavement system might give misleading results if the operating frequency of the loading device is close to the resonant frequency of the pavement system. Nevertheless, static interpretation of the pavement deflections is still used extensively in the analysis methods because of its simplicity.

In this study, selected Roller Compacted Concrete

(RCC) pavement slabs were subjected to both dynamic and static loads. The RCC pavements were located at Moran Terminal, Boston, Massachusetts. Four slabs were selected for testing. Static loads were applied by a standing wheel load at various locations and strains were measured by strain gages. A Benkelman beam was used to measure the deflections at a few points, as well as load transfer efficiency across slab joints. A heavy weight deflectometer (HWD), which is similar to a conventional falling weight deflectomer (FWD), but has higher load capacity, was utilized to perform the dynamic load testing. Deflection basins were obtained using deflection sensors placed at different positions.

The objective of the study is to compare the deflections and stresses produced by the static loads to those caused by dynamic loads. A procedure for back-calculating the pavement material properties is also developed using the measured deflection basins and the finite element computer program JSLAB which was developed at the Construction Technology Laboratories, Inc.

BACKGROUND

Nondestructive load testing devices can be categorized as static and dynamic. The earliest device for measuring the static pavement deflection was the General Electric Travel Gage used by the California Division of Highways as early as 1938.(10) However, it was not until the development of the Benkelman beam at 1952 which made it practical to measure the surface deflections routinely and at low cost. Since then,this device has been used by many highway organization throughout the world.

To date, the Benkelman beam is still the most widely used device of its kind. A schematic of the device is shown in Figure 1. The equipment is versatile, simple, and inexpensive to operate. It requires a minimum amount of training for the operator to conduct the test. However, it is slow and labor intensive. The equipment measures only the maximum deflection which is not sufficient for use in mechanistic type of pavement models. Another major drawback of this device is that an immovable reference point, which is sometimes difficult to obtain, is required for making the deflection measurements. This, in turn, makes the absolute accuracy of this type of measurement questionable. The large amount of historical deflection data collected by various agencies with the Benkelman beam, however, make the static deflection procedures an important part of structural pavement evaluation. Many empirical rehabilitation and overlay design procedures for flexible pavements have been developed based on relationships between Benkelman beam deflections and pavement performance.(11-17)

All the methods were based on the following concept:

1. For adequately designed pavements, the deflection measured during the same season of the year remains about constant for their entire life.
2. A tolerable level of deflection can be established by the use of the fatigue characteristics of the pavement structure. The tolerable level of the deflection is a function of traffic type and volume, and the structural capacity of the pavements.
3. Overlay of a pavement can reduce its deflection. The thickness required to reduce the deflection to a tolerable level can be established.

Two types of dynamic load testing devices exist. One type, the Dynaflect, Road Rater and other vibratory devices produce a sinusoidal vibration on the pavement surface using some form of a dynamic force generator. The steady-state sinusoidal force is superimposed on a static load generated by the dynamic force generator. The surface deflections are measured by deflection sensors (geophones) placed at various locations. The measured deflections are generally referred as deflection basins.

The falling weight deflectometers (FWD) are the second type of the dynamic load testing devices. They deliver an impulse load onto a pavement surface by dropping a mass onto a buffered load plate. Figure 2 shows the schematic of a FWD. Variable peak force levels are achieved by varying the magnitude of the mass or the height of fall. Deflections are measured by geophones placed at different locations. Some researchers have suggested that the FWD is the best device among some selected NDT devices to simulate pavement responses under moving loads.(18)

There is general agreement among pavement engineers and researchers that deflection basins generated by the dynamic loading devices can provide valuable information for pavement evaluation and rehabilitation. Various parameters have been defined and used to describe the characteristics of the deflection basins. Many pavement evaluation procedures have been developed using measured deflection basins together with either

the elastic layered theory or the plate theory. Table 1 lists the various deflection basin parameters which have been used in the past. Basically, most of the methods estimate the in situ pavement properties by matching the measured deflections and the deflections computed by theoretical means. The deflection basin parameters listed in Table 1 have been used to facilitate this deflection matching process.

DEVELOPMENT OF A BACK-CALCULATION PROCEDURE

A procedure was developed at CTL for back-calculating the pavement material elastic modulus and the modulus of subgrade reaction of rigid pavement systems using FWD or HWD deflection data. The Heavy Weight Deflectometer (HWD) is a similar device as the FWD but has higher load capacity. The finite-element computer program JSLAB was used in developing this procedure.

Computer Program JSLAB

The computer program JSLAB was developed at the Construction Technology Laboratories, Inc. for analysis of jointed concrete pavements. The JSLAB model is based on the classical theory of a medium-thick plate on a Winkler foundation. A rigid pavement slab is modeled as assemblage of plate-bending elements with four nodes and twelve degrees of freedom on each element. The Winkler foundation is modeled as a uniform distributed subgrade through an equivalent mass foundation.

Various types of load transfer systems can be considered. The dowel bars are modeled as two-dimensional beam elements while the aggregate interlocks and keyways are modeled by shear springs connecting the nodes along the joints. The effect of a stiff base layer or an overlay, with or without bonding, can also be considered.

Development of Prediction Equations

Theoretical deflections were computed for 288 pavement sections using the JSLAB computer program. Three sets of slabs with dimensions of 3.7x3.7 m, 6.1x3.7 m, and 12.2x6.1 m (12x12 ft, 20x12 ft, and 40x20 ft) were used. For each set of slabs, the slab thickness varied from 13 to 51 cm (5 to 20 inches). The concrete elastic modulus varied from 13.8 to 41.4 GPa (2000 to 6000 ksi) and the modulus of subgrade reaction ranged from 27.2 to 217.4 MPa/m (100 to 800 pci). A 40-kN (9-kip) load was applied to the pavements with a slab thickness less than 30 cm (12 inches). For pavements with a slab thickness of 30 cm (12 inches) or greater, a 160 kN (36-kip) load was used. The load was applied to the slab center and deflections were recorded at the center of the load and at 0.3, 0.6, 0.9, 1.2 and 1.5 m (1, 2, 3, 4, and 5 ft) away from the loading center. For most of the sections, the load was uniformly distributed to an area of 710 cm^2 (110 $in.^2$) representing the area of a FWD loading plate. However, since the HWD had a loading area of 1587 cm^2 (246 $in.^2$), both these two values were used in twenty-four pavement sections which resulted in a total of 312 deflection basins. It was found that the loading area had no significant effect on the computed deflections for the considered pavement sections.

It is recognized that the radius of relative stiffness of a pavement system can be related to certain deflection basin parameters[7], and that measured deflection far away from the load can be used to predict the modulus of subgrade reaction with good accuracy.[3] From the computed deflection data, a regression equation was developed to predict the radius of relative stiffness of a pavement system from the deflection basin parameter Slope Factor (SF). The regression equation has the following form:

$$\left(\frac{1}{\ell}\right) = 0.00401148 + 0.102021 * (SF) - 0.00443311 * \log (SF) \qquad (1)$$

Where ℓ = radius of relative stiffness, in.

$$= \left(\frac{Eh^3}{12(1-\mu^2)K}\right)^{1/4}$$

E = concrete elastic modulus, psi

h = concrete slab thickness, in.

μ = concrete Poisson's ratio

K = modulus of subgrade reaction, pci

SF = deflection basin slope factor

$$= \frac{D_0 - D_2}{D_0}$$

D_0 = deflection under the load, in.

D_2 = deflection at 2 feet from the load

With a coefficient of determination (R^2) of 0.99343, Equation 1 should give a reliable estimate of the pavement radius of relative stiffness. Figure 3 shows the relationship between ℓ and SF for all pavement sections. The solid curve line in the figure represents the prediction equation.

For fixed slab thicknesses, multiple linear regression analysis was performed to derive the equations for estimating the modulus of subgrade reaction using the deflection measured at 0.9 m (3 feet) from the load (D_3). All the deflections were normalized with respect to a 40 kN (9-kip) standard load using the assumption of linear elastic behavior. The regression equations had the following forms:

$$\text{Log}(K) = a_0 + a_1 * \log(ND3) + a_2 * (\ell) + a_3 * (\frac{r}{\ell}) \quad (2)$$

$$\text{Log}(K) = a_0 + a_1 * \log(ND3) + a_2 * (\ell) + a_3 * (\frac{\ell}{L}) \quad (3)$$

Where K = modulus of subgrade reaction, pci

$ND3$ = normalized D3 deflection

$= D3 * \frac{9000}{P} * 1000$, in.

P = applied load, lb

ℓ = radius of relative stiffness as defined earlier, in.

r = radius of loading area, in.

L = concrete slab length, in.

a_0, a_1, a_2, and a_3 = constants to be determined by the regression analysis

Equation 2 was for pavement systems with a slab thickness of 15 cm (6 in.) or less while Equation 3 was for pavement systems with a slab thickness of 20 cm (8 in.) or greater. The constants a_0, a_1, a_2 and a_3 and the R^2 values for these equations are listed in Table 2. All the R^2 values for these equations are greater than 0.990.

Once the radius of the relative stiffness of the pavement system and the modulus of subgrade reaction were determined, the concrete elastic modulus of a pavement with known slab thickness could be calculated. The Poisson's ratio of the concrete was assumed to be 0.15. A computer program ECOPP (Estimation of COncrete Pavement Parameters) was coded using these regression equations.

Adjustment for Subgrade Nonlinearity

In developing above regression equations, the subbase/subgrade was assumed to be linearly elastic. Adjustments might be necessary to account for the nonlinear behavior of the subbase/subgrade materials. Following the concept of resilient modulus of granular materials, a nonlinear modulus of subgrade reaction was assumed to be dependent on the stress state. The following material model was used to describe the nonlinear behavior:

$$K_i = C_1 (T_i)^{C_2} \quad (4)$$

Where K_i = nonlinear modulus of subgrade reaction at point i

T_i = vertical stress at top of subgrade at point i

$= KD_i$

K = modulus of subgrade reaction determined from the regression equations

D_i = deflection measured at point i

C_1 and C_2 = constants

It has also been demonstrated that the material nonlinearity was quite localized under FWD loads.[23] Thus, it was assumed that the modulus of subgrade reaction varied only within 1.5 m (5 feet) of the load and a constant K was used outside 1.5 m (5 feet). Within 1.5 m (5 feet) of the load, the area was further divided into two parts. The first part covered the area from the loading center to 0.9 m (3 feet) from the load, the second part from 0.9 m (3 feet) to 1.5 m (5 feet). The modulus of subgrade reaction was assumed to be constant within each part under any particular load.

In the program ECOPP, the modulus of subgrade reaction (K) was first estimated from deflection measured at 0.9 m (3 feet) from the load (D_3). The K was then adjusted to account for the material nonlinearity using Equation 4 and measured deflections at the center of the load, at 0.9 m (3 feet) away from the load and at 1.5 m (5 feet) away from the load (D_0, D_3 and D_5, respectively). From Equation 4, the material nonlinear behavior can be expressed as follows:

$$K_0 = C_1 (T_0)^{C_2} = C_1 (KD_0)^{C_2}$$

$$K_3 = C_1 (T_3)^{C_2} = C_1 (KD_3)^{C_2}$$

$$K_5 = C_1 (T_5)^{C_2} = C_1 (KD_5)^{C_2}$$

Since $K_3 = K$, K_0 and K_5 can be calculated as follows:

$$K_0 = (\frac{D_0}{D_3})^{C_2} * K \quad (5)$$

$$K_5 = (\frac{D_0}{D_5})^{C_2} * K \quad (6)$$

The value of K_0 was used to represent the modulus of subgrade reaction for the area from the load center to 0.9 m (3 feet) from the load. The area from 0.9 m (3 feet) to 1.5 m (5 feet) from the load used the value of K_3 and the value of K_5 was assumed for area outside 1.5 m (5 feet) from the load. The power term C_2 is

an input item in the program ECOPP and should be determined based on the measured deflection basins.

FIELD AND LABORATORY TESTING

Two sets of nondestructive testing (NDT) were performed on Roller Compacted Concrete (RCC) pavements. the first set used static loading and dynamic loading was utilized for the second. The static loading test was conducted on May 2 and May 3, 1990, while the dynamic testing was performed on October 30, 1991.

The RCC pavements were located at Moran Terminal at Boston, Massachusetts. The pavements consisted of a RCC layer of 38 cm (15 in)., a 23 cm (9-in). gravel subbase and a compacted subgrade. The RCC layer was constructed in three lifts of 14, 14, and 10 cm (5.5, 5.5 and 4 inches). Four slabs were selected for both the static and dynamic load testing. They were designated as Slabs 1 through 4. Information derived from Slabs 1 through 3 was used for this paper. Slab dimensions were about 30.5x4.9 m, 30.5x8.2 m, and 39.0x5.2 m (100x16 ft, 100x27 ft, and 128x17 ft) for Slabs 1, 2, and 3, respectively. Figure 4 shows the locations of the tested slabs and the general layout of the Moran RCC pavements.

Static Load Testing

The static loads were provided by a Marathon LeTourneau Letro-Porter used for transporting the containers in the terminal. When it was empty,the front wheel of the Letro-Porter had a tire load of 160 kN (36,000 lb). With a tire pressure of 517 kPa (75 psi), contact area was 3097 cm^2 (480 $in.^2$). The wheel dimensions of the Letro-Porter are shown in Figure 5. Strains caused by the wheel load placed at different locations were measured by strain gages bonded to the slab surface. Figures 6, 7 and 8 show the load positions and strain gage layout on Slabs 1, 2, and 3, respectively. Also shown in these figures are the locations of load transfer measurements for each slab. When the load was applied at one side of a construction joint or a crack, both deflections in the loaded and the unloaded slabs were measured. The load transfer efficiency (LTE) is defined as the deflection in the unloaded slab divided by the deflection in the loaded slab, expressed as percentage. Deflections were measured by a Benkelman beam. Attempts were also made to measure the load-induced deflections along the slab centerlines. However, since the pavement systems were very stiff and the precision of the dial gage used in the Benkelman beam was limited, no meaningful data were obtained.

Dynamic Load Testing

A heavy weight deflectometer (HWD) was used in the test. The HWD delivered an impulse load to the pavement surface through a loading plate. The HWD can provide loads of over 267 kN (60 kips). The diameter of the loading plate is 45 cm (17.7 inches). Deflections were measured by velocity transducers placed at different locations. The loading and sensor layouts are displayed in Figures 9, 10, and 11 for Slabs 1, 2, and 3, respectively. HWD loads were applied at 15 positions on each slab. Four different load levels were used for each load position. The use of several load levels was necessary in order to determine the linearity of the HWD load-deflection characteristics. Two drops were performed for each load level.

Although load testing was conducted at fifteen positions on each slab, only the center load (S11.1 in the figures) deflection data and the joint load (S10.1 in the figures) deflection data were used in this study, and are displayed in Table 3. The center load deflections were used to estimate the RCC elastic modulus and the modulus of subgrade reaction while the joint load deflections were used to compute the LTE across the joints. Analyses of the complete data of pavement response will be reported later in other studies.

Testing of Core Samples

Three 10-cm (4-in.) diameter cores were taken from each slab. The three lifts were found to bond together for all specimens. Laboratory tests were conducted to determine the elastic modulus, compressive strength, split tensile strength and unit weight of the RCC specimens. Shear test was also used to determine the shear strength of the specimens. The RCC elastic moduli estimated from the HWD deflection basins were compared with the laboratory measured values to evaluate the suitability of the back-calculation procedure. The other lab data are not reported in this paper.

ANALYSIS OF DATA

General

Data were analyzed in three steps. The RCC elastic modulus and the modulus of subgrade reaction were first estimated from HWD deflection data using the ECOPP procedure. Estimated RCC elastic moduli were compared to those measured from core

samples to ascertain the suitability of the back-calculation procedure. The second step involved adjusting the estimated moduli of subgrade reaction for material nonlinearity by matching HWD measured deflection basins and those computed by JSLAB. The modification procedure has been described in an earlier section of this report. Lastly, using the estimated parameters, deflections and stresses caused by the static loads were computed and compared to field measured values.

Determination of E and K

HWD deflections should be used with caution when attempting to back-calculate pavement material properties. It is a well-known fact that the temperature differential in a concrete slab will cause it to curl. When the top of the slab is warmer than the bottom, it will curl up at the slab interior and lose partial contact with the subbase/subgrade. The results might be misleading if deflections were collected under this situation.

Four load levels were used for each load position on the slabs. The load-deflection characteristic for center load position (S11.1) is shown in Figures 12, 13, and 14 for Slabs 1, 2, and 3, respectively. It can be observed that the load-deflection relationship is approximately linear for all three slabs. The deflections were used for predicting pavement parameters. However, some variation can be expected since they are not perfectly linear.

Actual RCC slab thicknesses were measured from cores and were 40 cm, 38 cm and 37 cm (15.8 in., 14.8 in., and 14.5 in.). Table 4 lists estimated RCC elastic moduli and moduli of subgrade reaction for the slabs at different loads. In general, the estimated values at different load levels were close to one another except the ones obtained at 89-kN (20-kip) load level of Slab 1, which were excluded from the analysis. Coefficients of variation of the RCC moduli estimated from responses to different loads were 4.1%, 6.6%, and 4.3% for Slabs 1, 2, and 3, respectively. Estimated moduli of subgrade reaction for the slabs had coefficients of variation of 3.6%, 7.4%, and 6.9%. Average estimated RCC elastic moduli also compared favorably to laboratory measured values determined from cores. As shown in Table 4, the estimated values are 11.0% higher, 0.3% lower, and 14.9% higher than the measured values for Slabs 1, 2, and 3, respectively. Considering the variations involved in field testing, ECOPP back-calculation procedure was considered to predict pavement material properties with reasonable accuracy.

Adjustment of K for Material Nonlinearity

Using predicted RCC moduli and moduli of subgrade reaction, deflection basins for the four different loads were computed and compared to measured values. It was observed that the computed and measured deflection basins matched well at distance three or more feet away from the loads. They did not match well close to the loads. The discrepancy between the measured and computed deflections was as high as 22% for some cases. Computed deflections were always greater than the measured ones. The discrepancy might be attributed to the nonlinear behavior of subgrade material. It was hypothesized that the subgrade was stiffer when closer to the load center line than away from it.

Equations 5, and 6 presented in earlier section were used for adjusting the moduli of subgrade reaction. After a trial and error process, the constant C_2 in these equations was determined to be 1.5 for all three slabs. Variable moduli of subgrade reaction for the RCC pavements were determined accordingly. The adjusted moduli of subgrade reaction are displayed in Table 5. Deflections were computed by JSLAB using adjusted K values and were compared to the measured deflections as shown in Figures 15, 16, and 17. As can be observed from these figures, deflection basins matched well with each other up to about 0.9 m (3 feet) from the load. Thus, the estimated values were considered satisfactory in representing in situ pavement material properties.

Comparison of Static and Dynamic Load Testing Data

Load transfer efficiency (LTE) across pavement joints was measured for both static and HWD tests. LTE was measured at about the center of transverse joints and at longitudinal joints 2.4 m (8 feet) from transverse joints. The measured LTE is shown in Table 6. It can be noted that the LTE measured at the static load testing was always higher than that measured by HWD. This might be because that (1) HWD tests were conducted over one year after performing the static load tests, and the pavement joints have deteriorated during this period, and (2) air temperature at the time of static load testing were about 8.3°C (15°F) higher than that during HWD tests. The joint load transfer efficiencies obtained from static load testing were used in the analysis.

Using E and K estimated from HWD deflection data and LTE from field measurements, deflections and stresses in the slabs caused by static loading were computed using the

computer program JSLAB. The computed stresses were then compared to the measured stresses under the same static loads. The static loading was provided by a Letro-Porter front wheel. When empty, it has a tire load of 160 kN (36,000 lb). The contact area of the tire is about 3097 cm^2 (480 $in.^2$) with a tire pressure of 517 kPa (75 psi). Six gages were attached on each slab and load was applied to five positions as shown in Figures 6, 7, and 8. The five load positions were designated as L1 through L5, and the six gages were designated as G1 to G6. Only the readings taken from gages under the load and two feet from each side of the load were used in the analysis, which resulted in 42 data points.

Table 7 displays the computed and measured stresses in the slabs. A paired t-test was conducted on the 42 pairs of measured and computed stresses. The null hypothesis was rejected at 99% confidence level. Therefore, it can be inferred that, with 99% confidence, there is a difference between the computed stresses based on HWD results and the static load induced stresses in the RCC slabs. Figure 18 displays the comparison between the computed stresses based on parameters estimated from HWD data and measured stresses. The computed stresses are always greater than measured stresses. The average difference between them are 233 kPa (33.8 psi) with a standard deviation of 134 kPa (19.4) psi. It can also be observed that, expressed as a percentage, the computed and measured stresses are closer to each other at a higher stress level than those at a lower stress level. At the highest measured stress level in this study of 903 kPa (131 psi), the difference between the computed and measured stresses is about 15%. When extrapolated to a common design allowable working stress of 2.41 MPa (350 psi), the difference between them is about 4%. Therefore, a factor of safety of 4% can be expected when HWD deflection data are used in rigid pavement analysis and rehabilitation.

CONCLUSIONS

Based on the study, the following conclusions can be drawn:

1. A back-calculation procedure ECOPP was developed. This procedure has been demonstrated to predict rigid pavement material elastic modulus and modulus of subgrade reaction of rigid pavements with reasonable accuracy.
2. The stresses in RCC slabs computed using HWD deflection based pavement parameters were greater than the field measured, static load induced stresses. The mean difference between them was 233 kPa (33.8 psi).
3. HWD deflections can be used in pavement analysis and rehabilitation in place of static test data since they provide more conservative results compared to the measured static stresses. At a common design working stress of 2.41 MPa (350 psi), a factor of safety of 4% can be expected when using HWD deflections in pavement analysis and rehabilitation.

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REFERENCES

1. R.C. Coole, "Overlay Design Based on Falling Weight Deflectometer Measurements," Transportation Research Record 700, TRB, Washington, D.C., 1978, pp 59-72.
2. M.C. Wang and B.A. Anani, "Evaluation of In Situ Elastic Moduli from Road-Rater Deflection Basin," Transportation Research Record 810, TRB, Washington, D.C., 1981, pp 54-57.
3. B.F. McCullough and A. Taute, "Use of Deflection Measurements for Determining Pavement Material Properties," Transportation Research Record 852, TRB, Washington, D.C., 1982, pp 8-14.
4. A.F. Stock and J. Yu, "Use of Surface Deflection for Pavement Design and Evaluation," Transportation Research Record 954, TRB, Washington, D.C., 1984, pp 64-69.
5. P. Ullidtz and R.N. Stubstad, "Analytical-Empirical Pavement Evaluation Using the Falling Weight Deflectometer," Transportation Research Record 1022, TRB, Washington, D.C. 1985, pp 36-44.
6. N.A. Ali and N.P. Khosla, "Determination of Layer Moduli Using a Falling Weighter Deflectometer,"

Transportation Research Record 1117, TRB, Washington, D.C., 1987, pp 1-10.

7. A.M. Ioannides, "Dimensional Analysis in NDT Rigid Pavement Evaluation," ASCE, Journal of Transportation Engineering, Vol. 116, No. 1, 1990, pp 23-36.

8. J.M. Roesset and K.Y. Shao, "Dynamic Interpretation of Dynaflect and Falling Weight Deflectometer Tests," Transportation Research Record 1022, TRB, Washington, D.C., 1985, pp 7-16.

9. T.G. Davies and M.S. Mamlouk, "Theoretical Response of Multilayer Pavement Systems to Dynamic Nondestructive Testing," Transportation Research Record 1022, TRB, Washington, D.C., 1985, pp 1-7.

10. W.M. Moore, D.I. Hanson, and J.W. Hall, Jr., "An Introduction to Nondestructive Structural Evaluation of Pavements," TRB, Transportation Research Circular, No. 189, 1978, pp 1-33.

11. N.A. Huculak, "Evaluation of Pavements to Determine Maintenance Requirement," Highway Research Record No. 129, HRB, Washington, D.C., 1966, pp 12-27.

12. E. Zube and R. Forsyth, "Flexible Pavement Maintenance Requirements as Determined by Deflection Measurement," Highway Research Record No. 129, HRB, Washington, D.C., 1966, pp 60-75.

13. H.R. Smith, "A Deflection Survey Technique for Pavement Evaluation in Developing Countries," TRRL Report LR 525, Transport and Road Research Laboratory, United Kingdom, 1973.

14. P.J. Norman, R.A. Snowdon, and J.C. Jacobs, "Pavement Deflection Measurements and Their Application to Structural Maintenance and Overlay Design," TRRL Report LR 571, Transport and Road Research Laboratory, United Kingdom, 1973.

15. W.A. Fingalson and T.D. Robinson, "Deflection Study of Flexible Pavement Overlays," Minnesota Department of Highways, Investigation No. 630.

16. "Department of Highways Test Method Calif. 356-D," California Department of Transportation, October 1973.

17. R.I. Kingham, "Development of the Asphalt Institute's Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements," The Asphalt Institute, Research Report No. 69-3, June 1969.

18. M.S. Hoffman and M.R. Thompson, "Comparative Study of Selected Nondestructive Testing Devices," Transportation Research Record 852, TRB, Washington, D.C., 1982, pp 32-41.

19. M.C. Wang, T.D. Larson, A.C. Bhajandas, and G. Cumberledge, "Use of Road Rater Deflections in Pavement Evaluation," Transportation Research Record 666, TRB, Washington, D.C. 1978, pp 32-38.

20. M.R. Thompson and M.S. Hoffman, "Concepts for Developing an NDT-Based Design Procedure for Determining Asphalt Concrete Overlay Thickness," Transportation Research Record 930, TRB, Washington, D.C., 1983, pp 12-18.

21. M.S. Hoffman and M.R. Thompson, "Backcalculating Nonlinear Resilient Moduli from Deflection Data," Transportation Research Record 852, TRB, Washington, D.C. 1982, pp 42-51.

22. G.W. Sharpe, H.F. Southgate, and R.C. Deen, "Pavement Evaluation by Using Dynamic Deflections," Transportation Research Record 700, TRB, Washington, D.C., 1979, pp 34-46.

23. D.W. Chang, J.M. Roesset, and K.H. Stokoe, "Nonlinear Effects in Falling Weight Deflectometer Tests," paper presented at 71st Annual Meeting, TRB, Washington, D.C., 1992.

Table 1 Deflection Basin Parameters

Parameter	Formula	Source
Surface Curvature Index Base Curvature Index	SCI = D0-D1 BCI = D2-D3	ref. 19 ref. 19
Basin AREA Basin Shape Factor	AREA = 6{[D0+2D1+2D2+D3]/D0} F1 = (D0-D2)/D1 F2 = (D1-D3)/D2	ref. 7, 20, 21 ref. 21 ref. 21
Spreadability Basin Slope	Spreadability = (D0+D1+D2+D3+D4)/5D0 Basin Slope = D0-D4	ref. 3 ref. 3
D0, Projection	D0, Projection = 10 ^ [2log(D1)-log(D2)]	ref.22
Tangent Slope	TS = (D0-Dx)/x	ref.4

Note: Di = Deflection measured at i+1th sensor
Dx = Deflection at the tangent point
x = Distance of the tangent point from the point of maximum deflection

Table 2 Regression Equations for Estimating Modulus of Subgrade Reaction

Pavement Slab Thickness in.	Constants				Coefficient of Determination R^2
	a0	a1	a2	a3	
5	4.67014	-1.00366	-0.04159	-2.93908	0.99764
6	4.08076	-0.99399	-0.02821	-1.88585	0.99744
8	3.19287	-0.99076	-0.01298	0.40057	0.99836
10	3.21337	-1.00799	-0.01342	0.42543	0.99890
12	3.20290	-0.99595	-0.01353	0.48629	0.99817
15	3.14526	-0.98294	-0.01243	0.56773	0.99618
18	3.07694	-0.97320	-0.01160	0.65605	0.99324
20	3.03577	-0.96488	-0.01079	0.71466	0.99292

Table 3 Measured Deflections for HWD Test

Slab No.	Load Position	Load, kN	Deflection, cm						
			Distance from Loading Center, m						
			-0.3	0.0	0.3	0.6	0.9	1.2	1.5
1	S10.1	89	0.0050	0.0194	0.0158	0.0129	0.0109	0.0091	0.0074
		156	0.0088	0.0341	0.0279	0.0229	0.0195	0.0162	0.0133
		191	0.0115	0.0445	0.0363	0.0298	0.0252	0.0209	0.0171
		254	0.0140	0.0558	0.0459	0.0377	0.0317	0.0263	0.0215
	S11.1	89	0.0096	0.0111	0.0099	0.0089	0.0081	0.0072	0.0062
		156	0.0164	0.0180	0.0169	0.0150	0.0140	0.0125	0.0109
		200	0.0212	0.0237	0.0217	0.0195	0.0180	0.0159	0.0139
		267	0.0267	0.0298	0.0274	0.0246	0.0226	0.0201	0.0175
2	S10.1	89	0.0119	0.0149	0.0124	0.0101	0.0088	0.0076	0.0065
		156	0.0208	0.0270	0.0216	0.0178	0.0156	0.0133	0.0113
		196	0.0261	0.0345	0.0269	0.0223	0.0193	0.0165	0.0140
		267	0.0332	0.0422	0.0334	0.0276	0.0237	0.0202	0.0171
	S11.1	89	0.0086	0.0096	0.0086	0.0077	0.0070	0.0062	0.0054
		165	0.0151	0.0174	0.0154	0.0138	0.0127	0.0114	0.0100
		205	0.0191	0.0216	0.0193	0.0174	0.0161	0.0143	0.0126
		280	0.0241	0.0279	0.0243	0.0218	0.0199	0.0177	0.0155
3	S10.1	89	0.0080	0.0220	0.0178	0.0144	0.0120	0.0100	0.0082
		156	0.0144	0.0387	0.0313	0.0256	0.0218	0.0178	0.0146
		196	0.0181	0.0532	0.0397	0.0323	0.0273	0.0225	0.0185
		249	0.0212	0.0669	0.0495	0.0402	0.0336	0.0277	0.0236
	S11.1	89	0.0112	0.0124	0.0114	0.0103	0.0094	0.0083	0.0073
		156	0.0195	0.0219	0.0198	0.0181	0.0170	0.0150	0.0133
		200	0.0246	0.0277	0.0249	0.0228	0.0213	0.0189	0.0169
		267	0.0302	0.0351	0.0312	0.0283	0.0258	0.0232	0.0207

1 cm = 0.394 in., 1 m = 3.3 ft, 1 kN = 0.225 kip

Table 4 Measured and Estimated Pavement Parameters

Slab No.	Load kN	Estimated l, cm	Estimated Subgrade Modulus MPa/m	Estimated RCC Modulus MPa	Average Estimated RCC Modulus MPa	Measured RCC Modulus MPa
1	89 156 200 267	90.93* 102.93 101.80 102.21	104* 97 97 103	12,937* 19,854 18,843 20,429	19,708	17,754
	C.V.	0.6 %	3.6 %	4.1 %		
2	89 165 205 280	89.16 87.20 92.32 86.98	128 132 123 145	18,958 17,900 20,975 19,561	19,349	19,413
	C.V.	2.8 %	7.4 %	6.6 %		
3	89 160 200 267	97.64 100.92 100.96 96.04	87 84 83 96	17,518 19,168 19,127 18,142	18,489	16,095
	C.V.	2.5 %	6.9 %	4.3 %		

Note: * excluded from further analyses
10 cm = 3.94 in., 100 kN = 22.5 kips
1000 MPa = 145 ksi, 100 MPa/m = 368 pci

Table 5 Adjusted Modulus of Subgrade Reaction

Slab No.	Load kN	Adjusted Modulus of Subgrade Reaction, MPa/m		
		Distance up to 0.9 m from load	Distance of 0.9 to 1.5 m from load	Distance above 1.5 m from load
1	156 200 267	143 145 156	97 97 103	67 66 70
	Average	148	99	68
2	89 165 205 280	202 213 193 244	128 132 123 145	83 91 87 101
	Average	213	132	90
3	89 160 200 267	133 122 123 152	87 84 83 96	61 57 58 68
	Average	132	88	61

100 kN = 22.5 kips, 1 m = 3.3 ft
100 MPa/m = 368 pci

Table 6 Load Transfer Efficiency Across the Joints

Slab No.	Joint Type	LTE (%) from	
		Static Test	HWD Test
1	Transverse	100 (= 38/38)*	26
	Longitudinal	75 (= 30/40)*	N.A.
2	Transverse	83 (= 25/30)*	78
	Longitudinal	92 (= 58/63)*	N.A.
3	Transverse	57 (=10/18)*	35
	Longitudinal	91 (= 25/28)*	N.A.

Note: * measured deflections in parenthesis, 0.001 cm
N.A. = not available
1 cm = 0.394 in.

Table 7 Comparison of Measured and Computed Stresses

Load Position	Gage Position	Slab 1		Slab2		Slab 3	
		Computed Stress, kPa	Measured Stress, kPa	Computed Stress, kPa	Measured Stress, kPa	Computed Stress, kPa	Measured Stress, kPa
L5	G6	745	296	517	331	731	352
	G5	1034	903	1055	738	1262	683
	G4	676	614	593	310	779	496
L4	G5	579	572	476	248	655	331
	G4	931	869	883	579	1048	572
	G3	483	138	365	269	524	221
L3	G4	490	414	386	310	545	331
	G3	924	869	876	503	1034	683
	G2	490	234	372	255	531	296
L2	G3	490	317	379	193	538	331
	G2	931	710	876	855	1034	669
	G1	490	317	372	269	538	241
L1	G2	490	338	372	193	531	427
	G1	917	648	869	524	1034	607

100 kPa = 14.5 psi

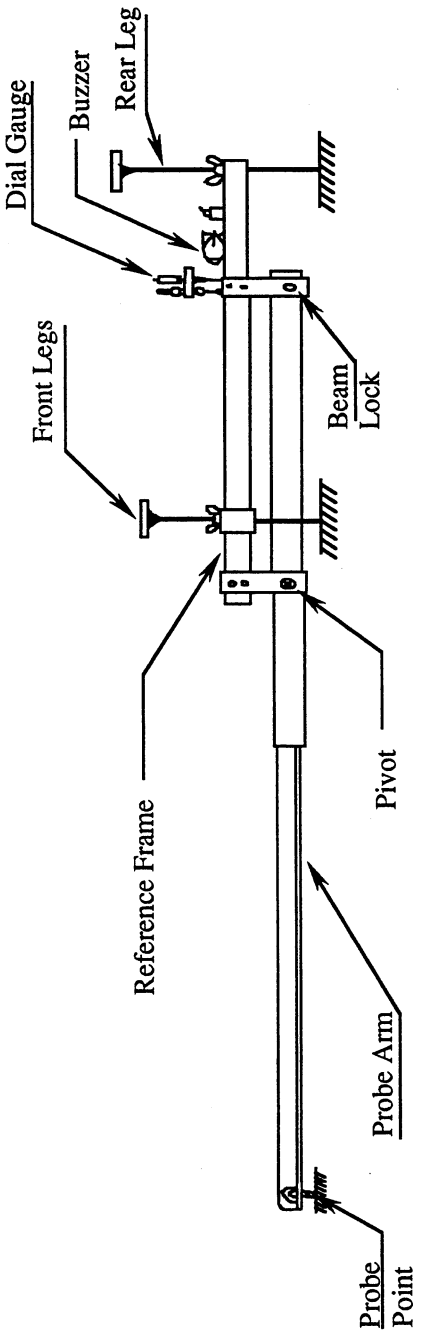
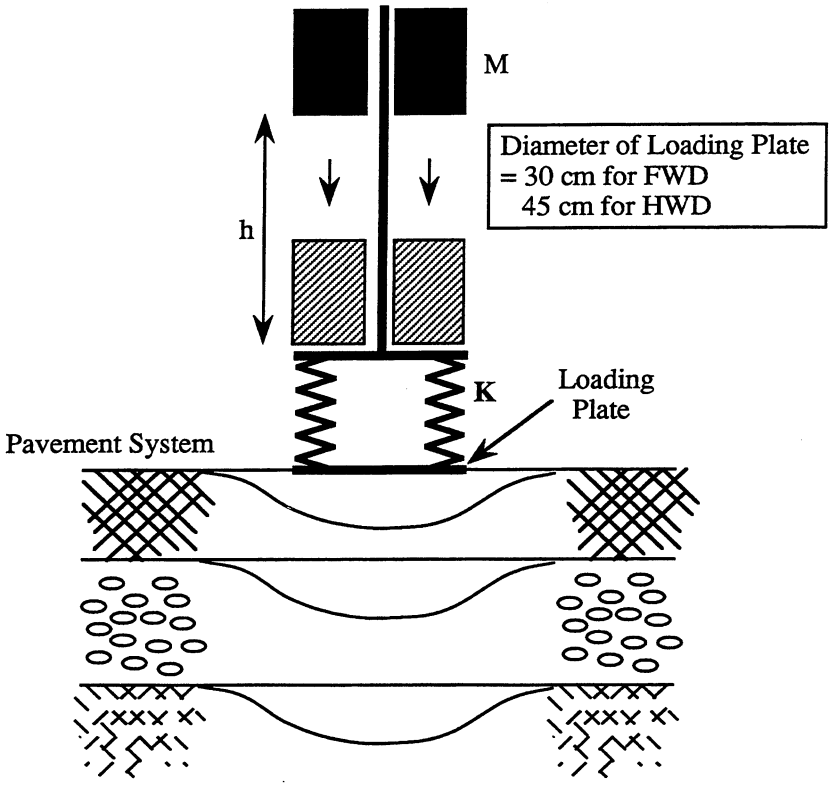


Figure 1 Benkelman Beam



10 cm = 3.94 in.

Figure 2 Schematic Presentation of a FWD or HWD

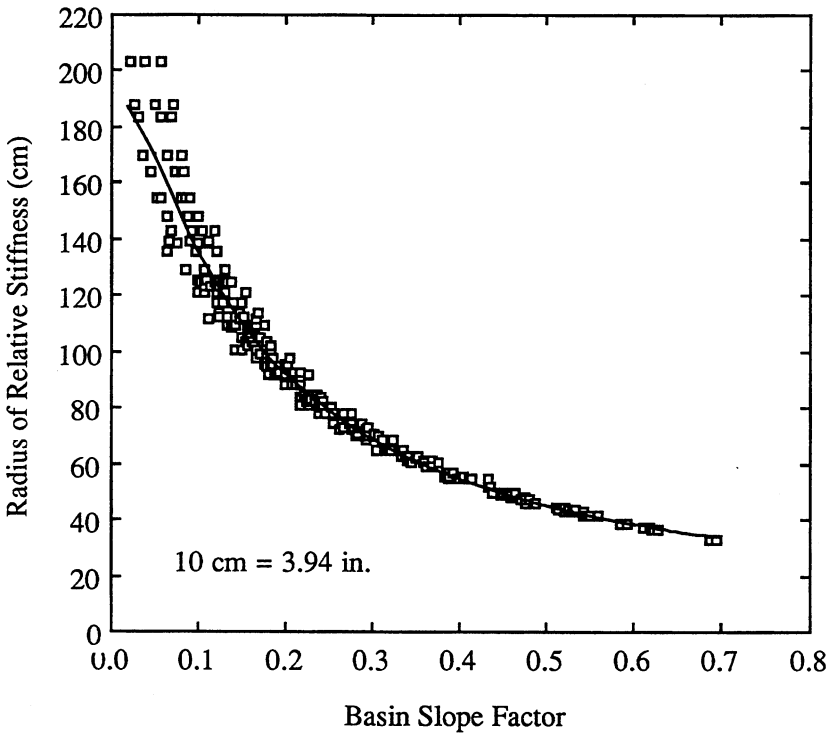


Figure 3 Radius of Relative Stiffness vs. Slope Factor

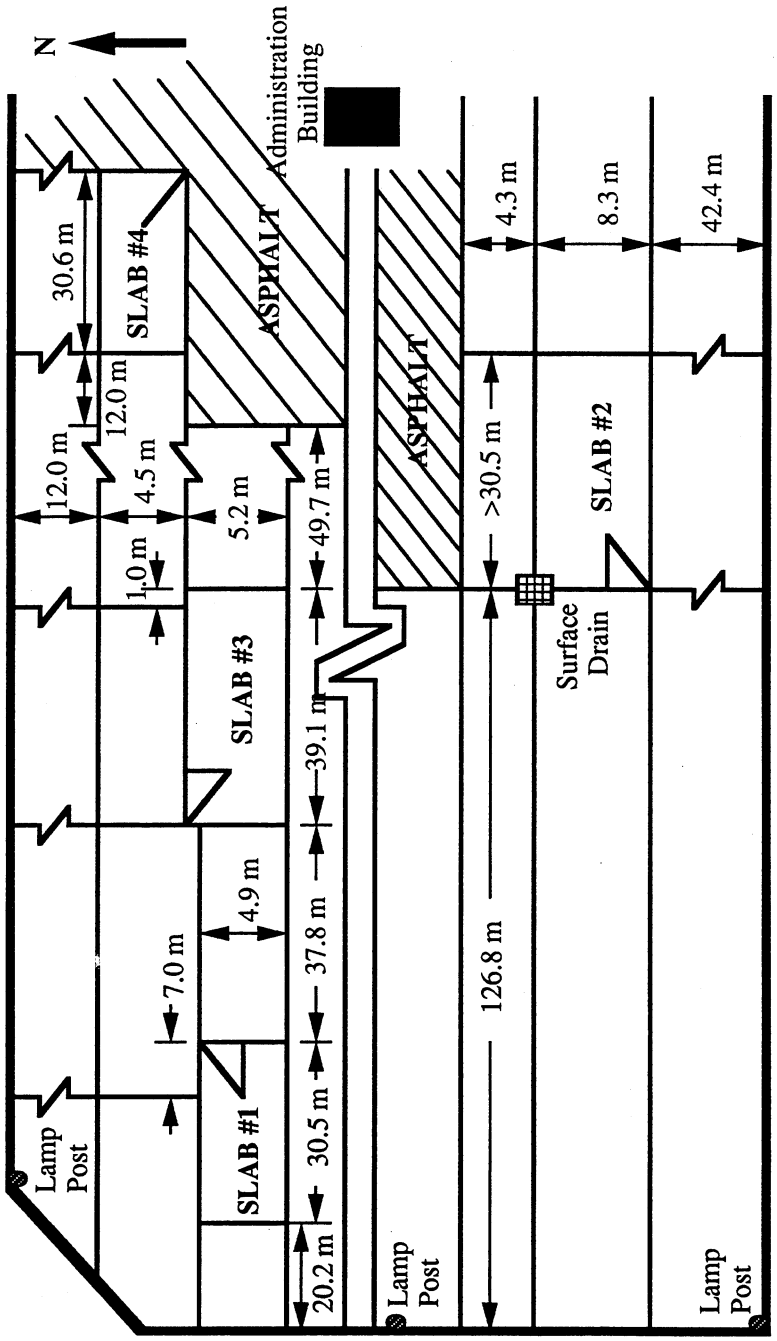
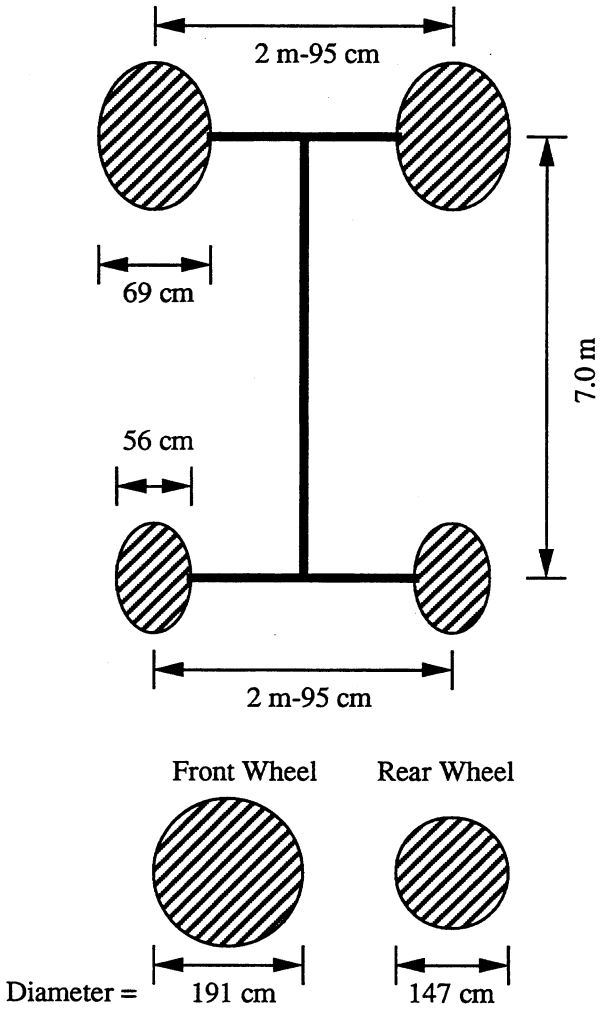
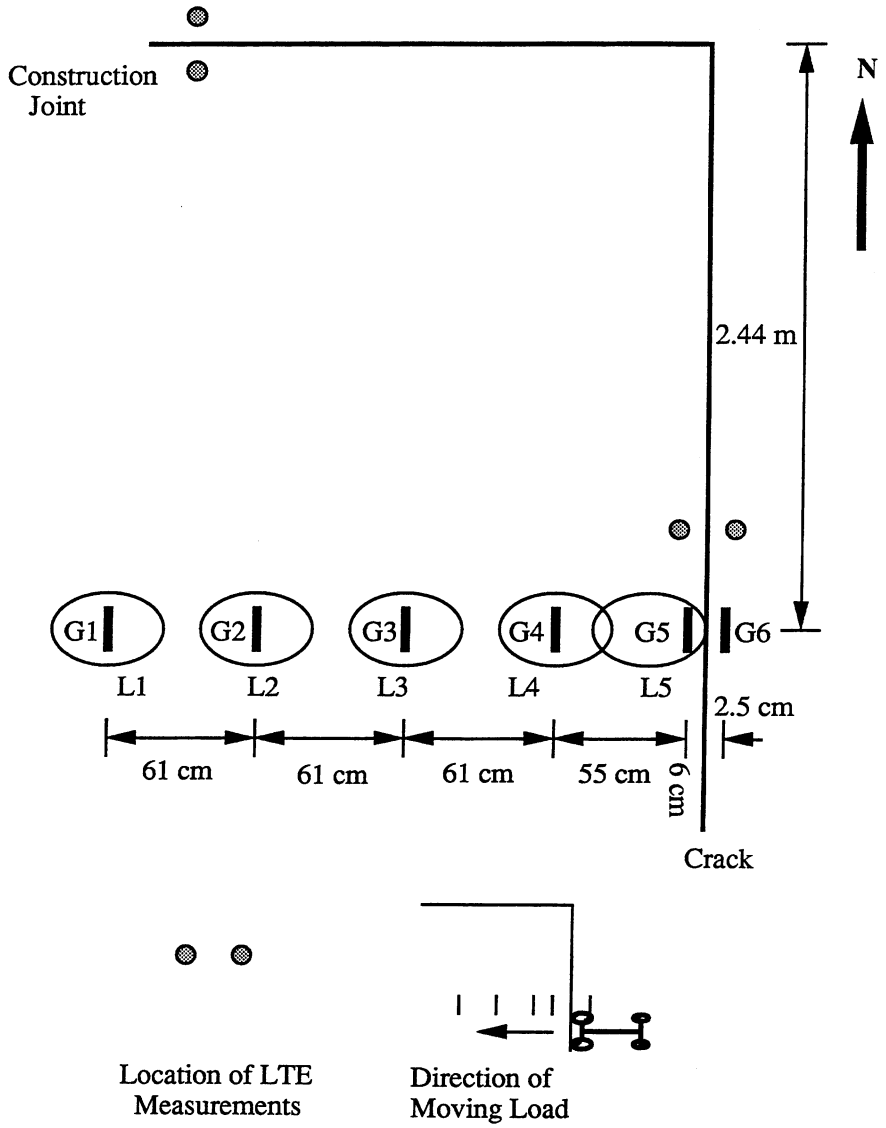


Figure 4 Layout of Moran Terminal RCC Slabs



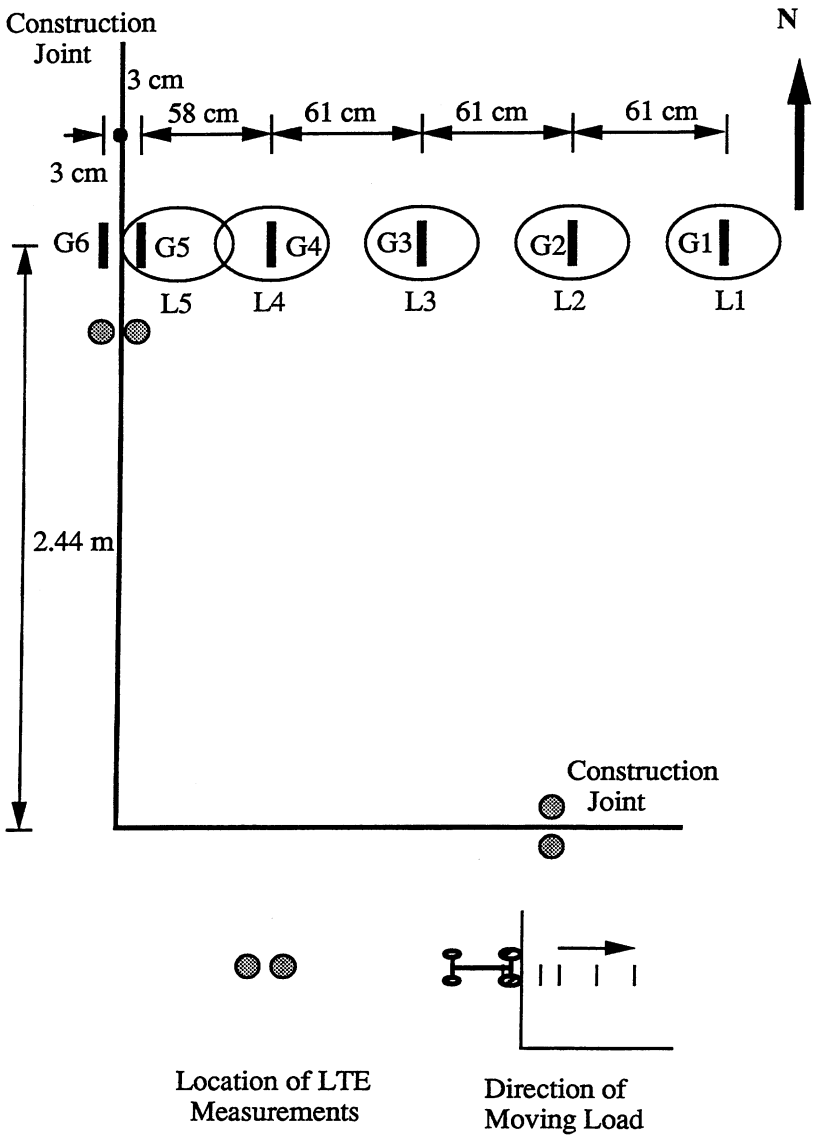
1 m = 3.3 ft, 10 cm = 3.94 in.

Figure 5 Dimensions of the Letro-Porter Wheel Loads



1 m = 3.3 ft, 10 cm = 3.94 in.

Figure 6 Load and Strain Gage Layout on Slab 1 for Static Load Testing



1 m = 3.3 ft, 10 cm = 3.94 in.

Figure 7 Load and Strain Gage Layout on Slab 2 for Static Load Testing

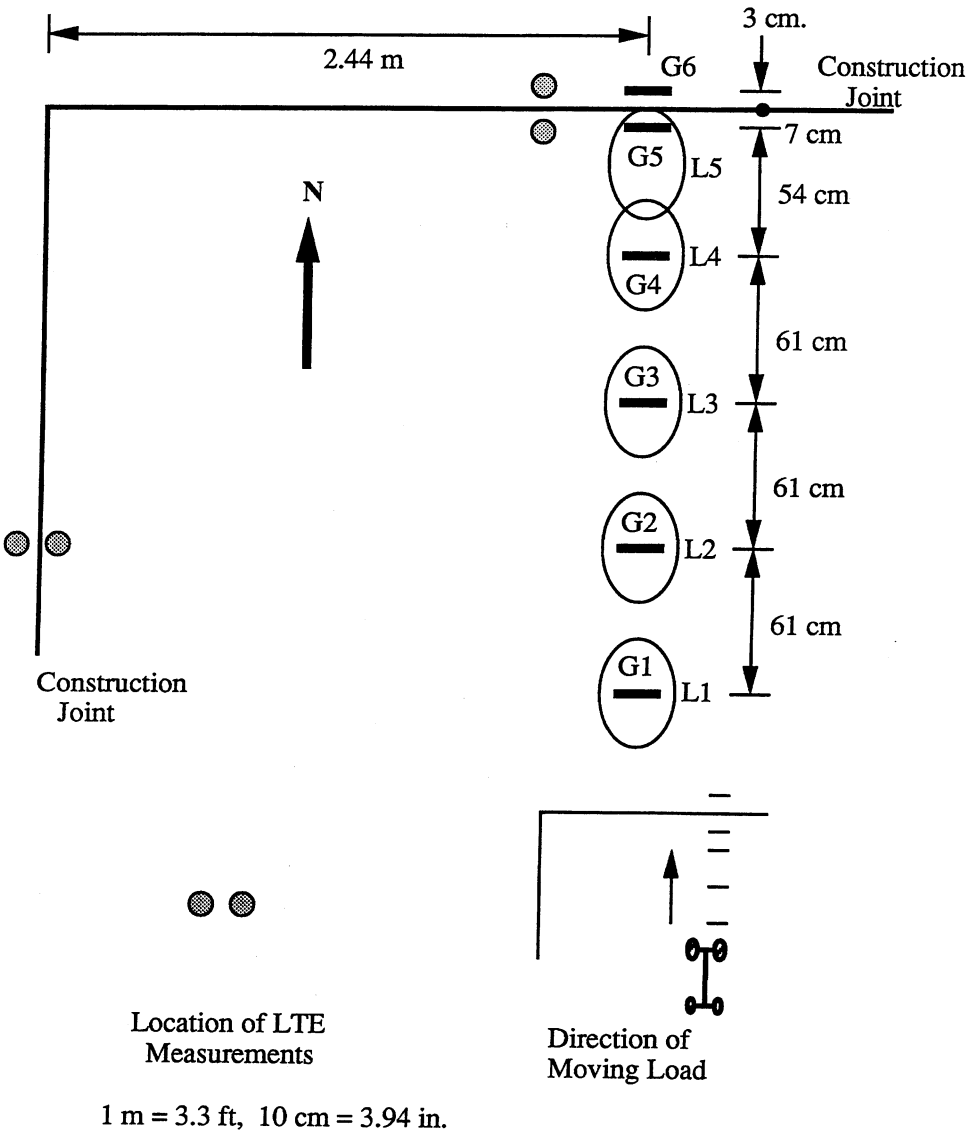


Figure 8 Load and Strain Gage Layout on Slab 3 for Static Load Testing

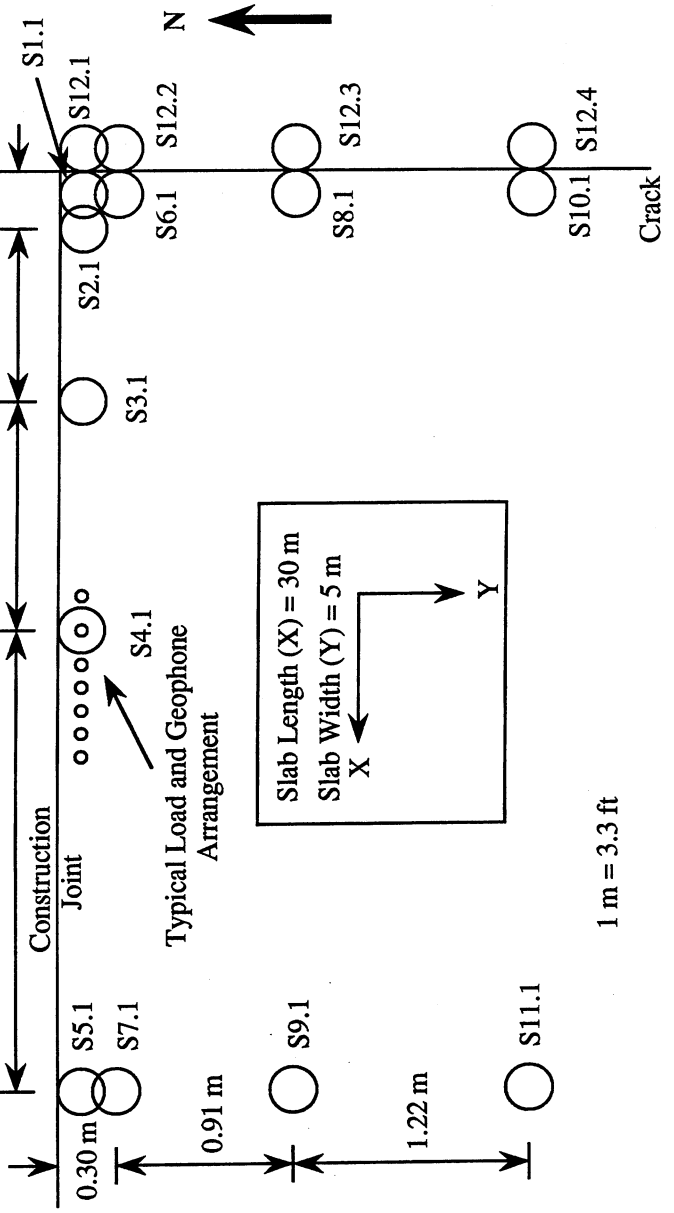


Figure 9 HWD Testing Plan for Slab 1

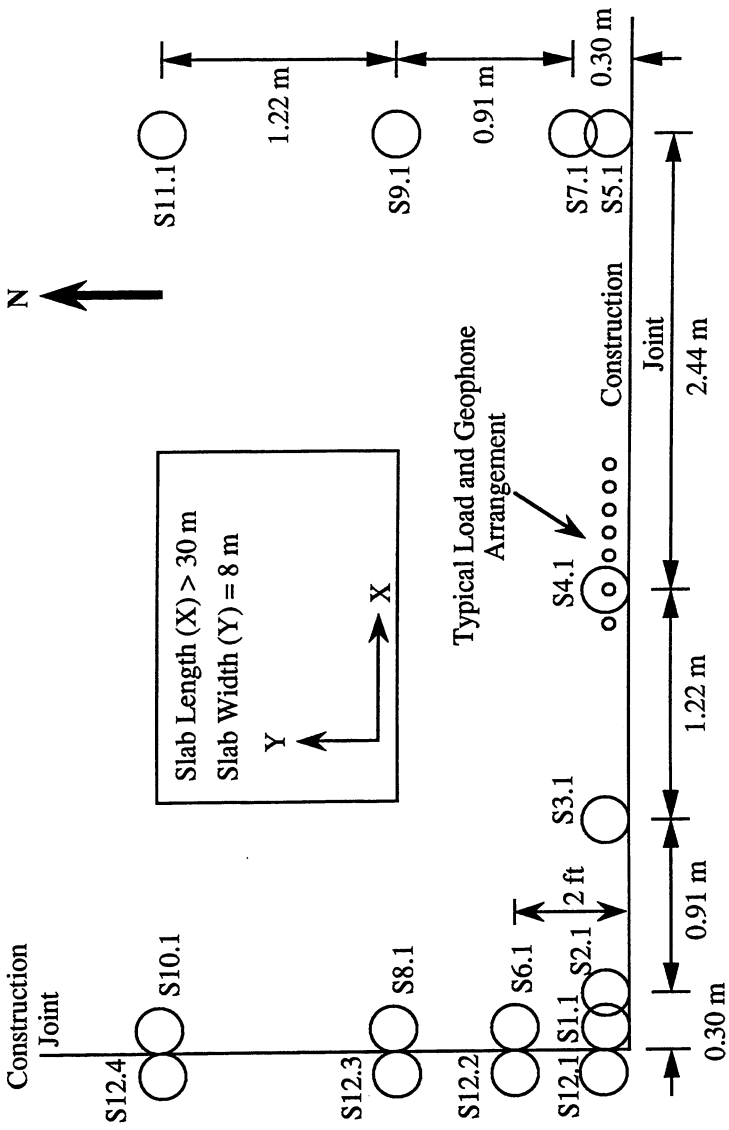


Figure 10 HWD Testing Plan for Slab 2

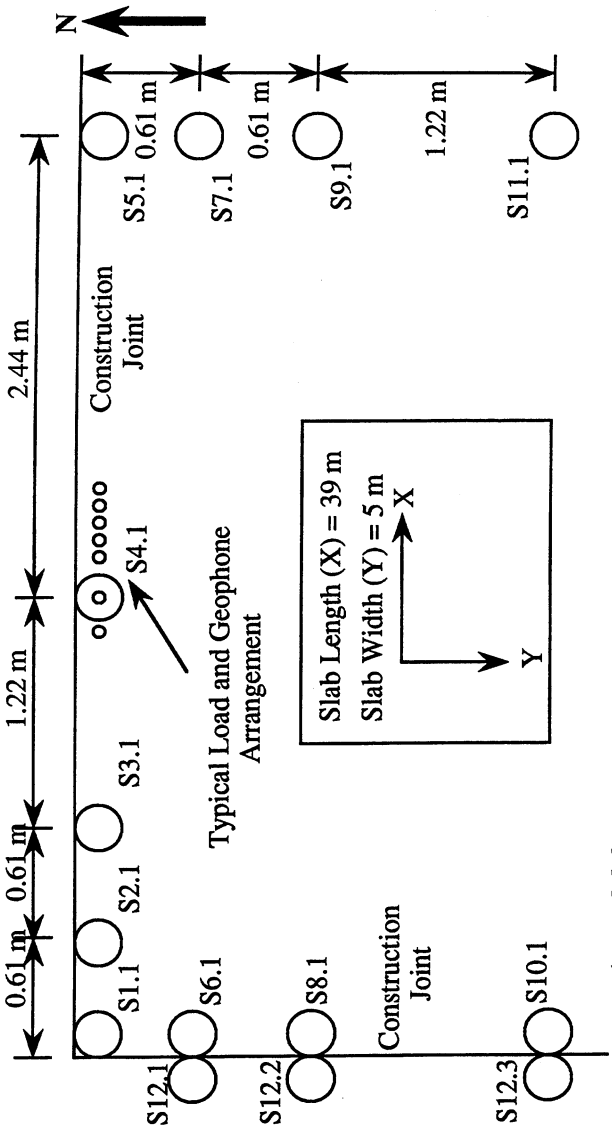


Figure 11 HWD Testing Plan for Slab 3

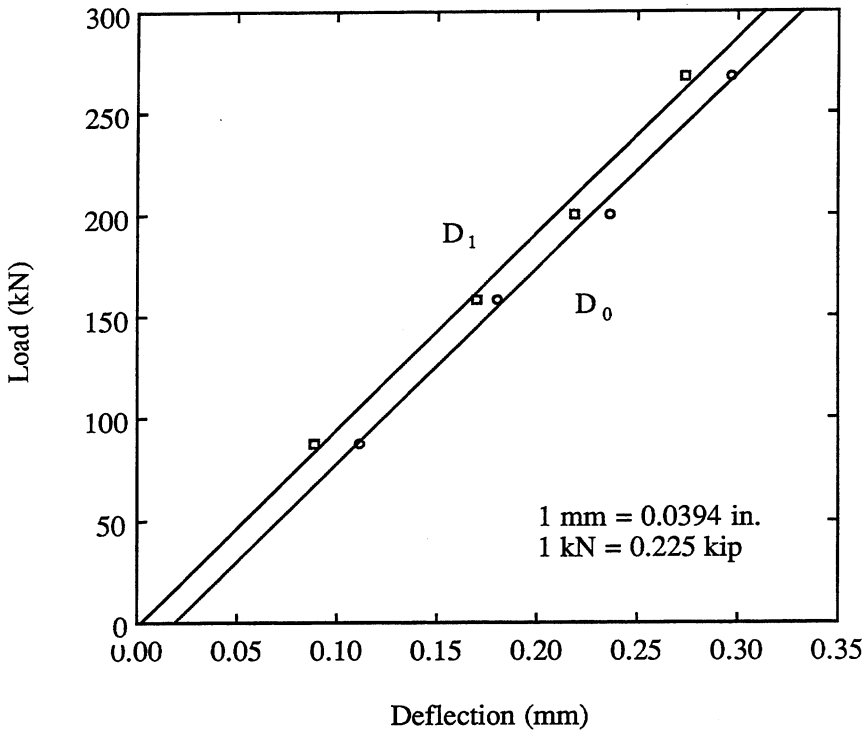


Figure 12 Load vs. Deflection for Slab 1

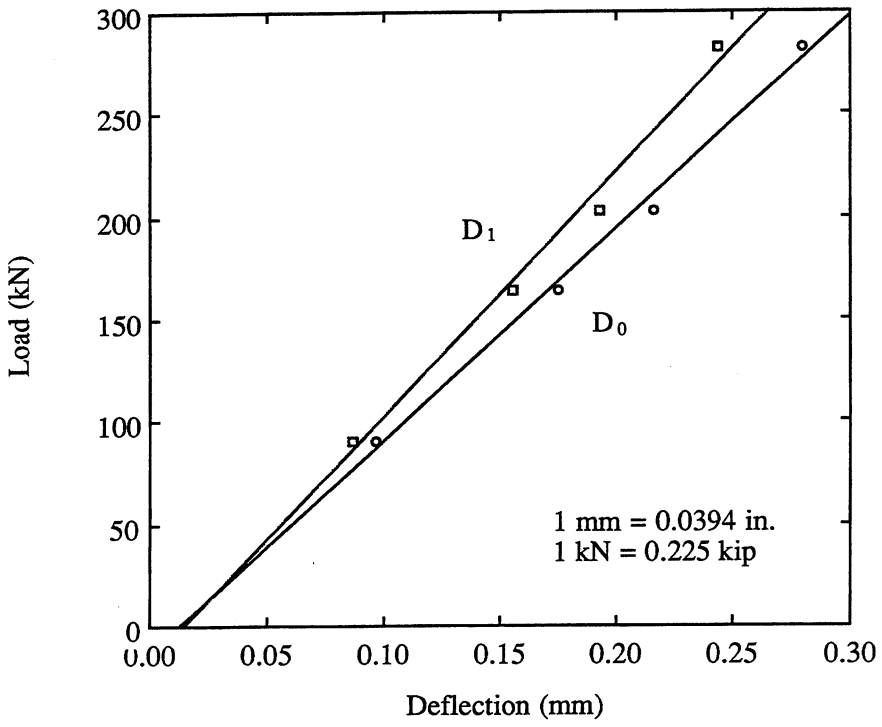


Figure 13 Load vs. Deflection for Slab 2

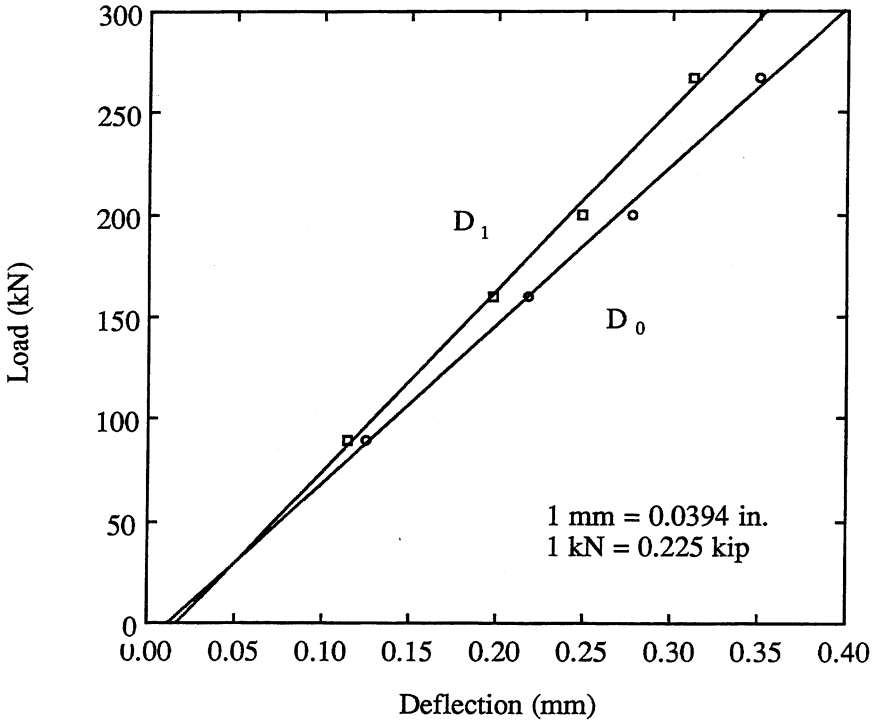


Figure 14 Load vs. Deflection for Slab 3

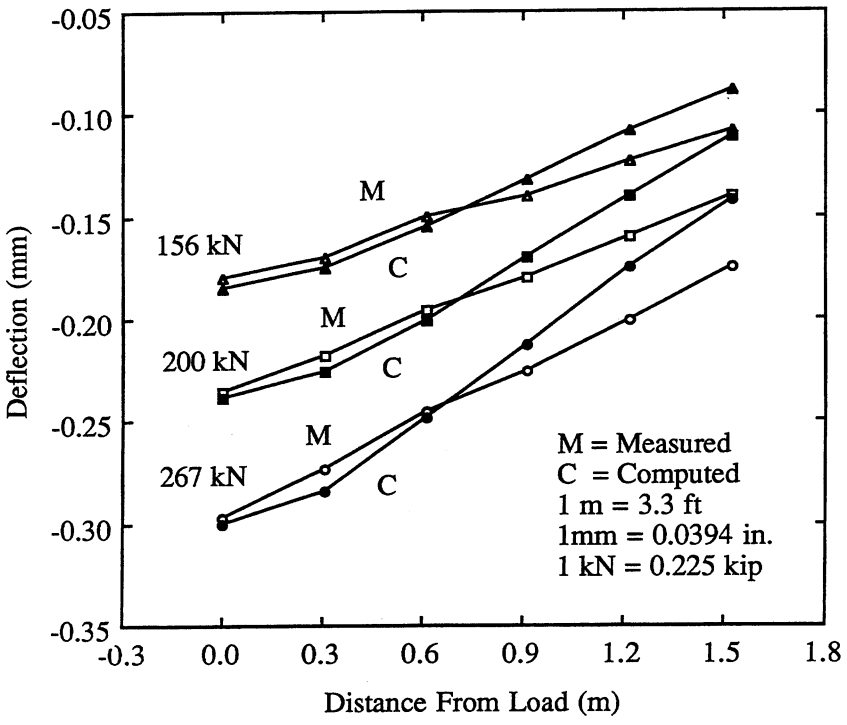


Figure 15 Comparison Between Measured and Computed Deflections at Various Loading for Slab 1

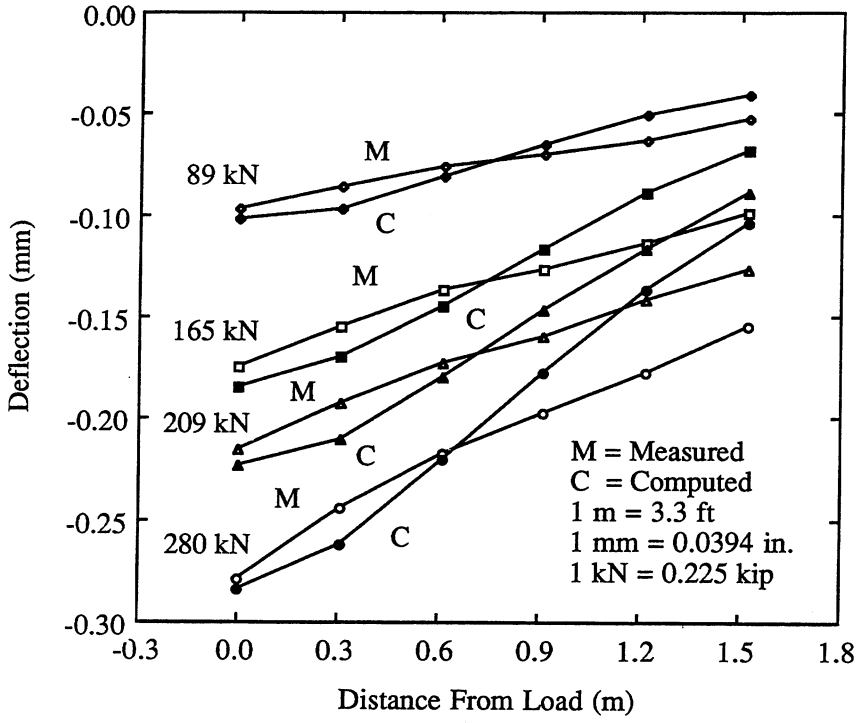


Figure 16 Comparison Between Measured and Computed Deflections at Various Loading for Slab 2

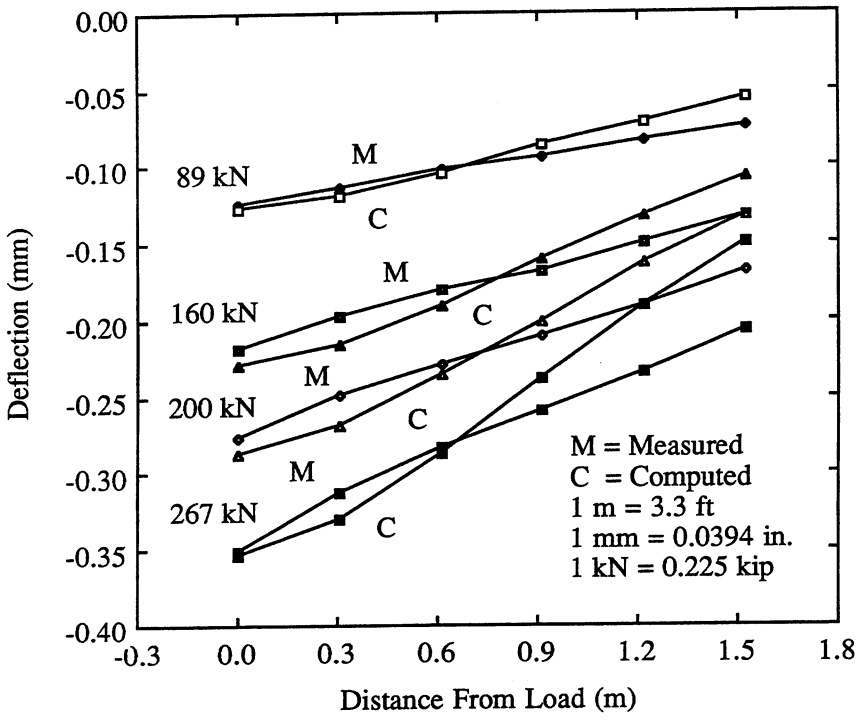


Figure 17 Comparison Between Measured and Computed Deflections at Various Loading for Slab 3

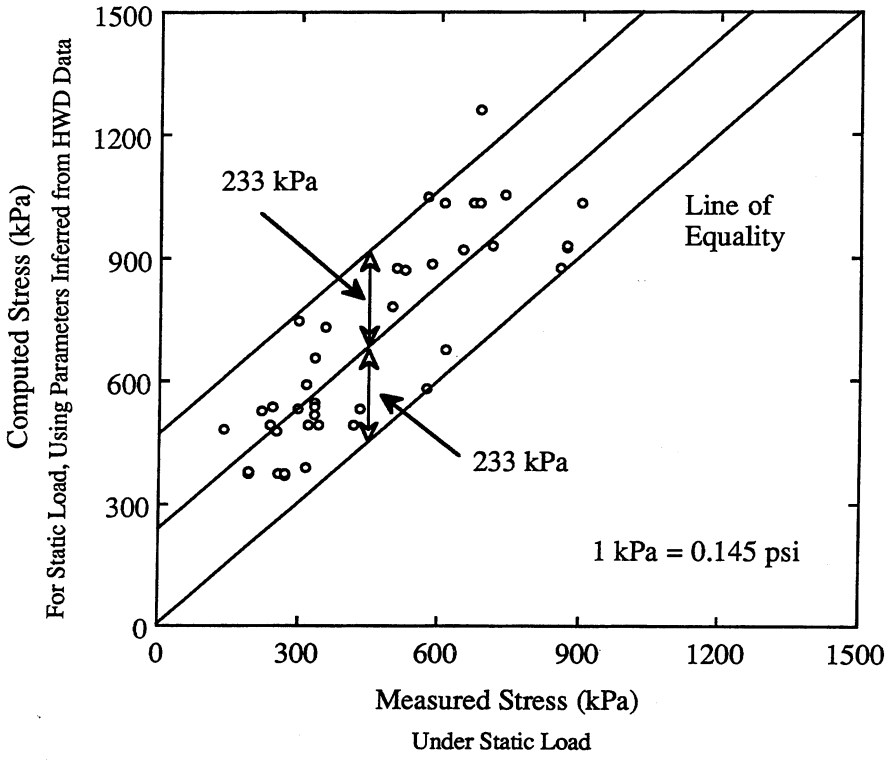


Figure 18 Comparison Between Measured and Computed Stresses