

Verification Of Curling In PCC Slabs At FAA National Airport Pavement Test Facility

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ABSTRACT

During the testing period from the summer to winter of 1999, heavy weight deflectometer (HWD) tests were routinely conducted on the three Portland Cement Concrete (PCC) test items in the FAA's National Airport Pavement Testing Facility (NAPTF). The analysis of the HWD data indicates that the measured deflections at the center of slabs remain almost the same but at the joints and corners, the measurements vary significantly. In wintertime, the deflections at the joints and corners are significantly larger than those measured in the summer. In addition, the joint load transfer capability, defined by the ratio between unloaded and loaded side deflections (LTD), was lower in winter. Also, the dummy joints received lower values than the doweled joints. A stronger nonlinear relationship between corner deflections and loads was also observed in the wintertime. The analysis indicates that the slab shapes varied all the time but were always curled up and more significant for slabs resting on a stronger sub-grade in wintertime. It has been found from the HWD data that the calculated LTDs could be very different when the HWD loading is applied on both sides of the joints, indicating that LTD may be sensitive to traffic direction. However, the sum of the above two deflections (SDs) still remain almost the same for both traffic directions. The SDs varied significantly from the summer to the winter, or it is sensitive to curling state of the slab. Therefore, the parameter SD that has not been used often in engineering practice may be a good indicator for investigating the slab curling.

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Introduction

The National Airport Pavement Test Facility (NAPTF), built for the FAA in a partnership with the Boeing Company, was completed in May 1999. The test facility is a fully enclosed instrumented test track, 900 feet long by 60 feet wide. The rail-based test vehicle has two loading carriages and can be configured for up to six wheels per carriage with loads up to 75,000 pounds per wheel. The test vehicle is programmed for a controlled aircraft wander simulation. The nine pavements tested in the first year consist of three sub-grade classifications: low (CBR 3-4), medium (CBR 7-9) and high (CBR 30-40). CBR stands for California Bearing Ratio of which the value gives an indication of the relative strength of the sub-grade. Each of the three-subgrade classifications is divided into one rigid (PCC), and two flexible (asphalt) surfaces. The three rigid sections are designated LRS, MRS, & HRS and outlined in figure 1.

The three PCC pavements were extensively tested with a heavy weight deflectometer. On June 14 and 15, 1999, Engineering & Research International, Inc. (ERI) did the first series of heavy weight deflectometer tests by contract to the FAA. ERI utilized a KUAB Model 150 falling weight deflectometer with a 12 inch (30.5 cm) load plate and a pulse width of approximately 27 msec. The following points (location of load plate center) in figure 2 were tested: West side of the middle transverse joints (points T01, T03, ..., T23), North side of the middle longitudinal joints (L01, L03, ..., L19) and one of the four corners (North-West, N01, N05, ..., N29). All points in figure 2 were tested between ends of October, 1999 to January, 2000. The temperature range for each test is listed in Table 1.

Locations of the HWD deflection sensors are shown in figure 3. D0 defines the position of deflection at the center of the loading plate. D-1 is the deflection sensor 12 inch (30.5 cm) in front of the loading plate and D1 to D5 are deflection sensors behind the loading plate with 12 inch (30.5cm) spacing. The arrangement of one sensor D-1 located in front of the loading plate makes it easy to obtain the load transfer in both directions at a joint. For example, when D0 locates at T03 after the vehicle drives from West to East (figure 2), the ratio of D-1/D0 indicates the load transfer coefficient from West to East. When the vehicle moves 12 inch (30.5 cm) ahead and D0 locates at T04, ratio D1/D0 indicates the load transfer capability of the same joint from East to the West.

Description Of The HWD Equipment

The FAA's Airport Technology R&D Branch is responsible for implementing a research initiative into Non-Destructive Pavement Testing, which ultimately will provide the FAA with a fast, economical and accurate method of evaluating the strength of existing airport pavements and subgrades. The specific outputs of this research initiative will be reports and guidance for assisting the FAA to develop the appropriate standards for use at our Nation's airports. The standards

developed should impact airport pavement maintenance and management systems, as well as airport pavement overlay designs.

Primarily with this goal in mind, the Airport Technology R & D Branch, set out to purchase a heavy weight deflectometer device which would allow the variability for this research initiative, and would also satisfy specific testing needs of the National Airport Pavement Test Facility. The NAPTF's requirements are for a heavy weight deflectometer capable of determining the uniformity of the test pavement structures and measurements of pavement response during traffic testing. The major objectives of conducting heavy weight deflectometer tests include:

- (1) To verify the uniformity of the sub-grade strength,
- (2) To determine the joint load transfer capability,
- (3) To investigate the variation of the pavement structure versus time and temperature,
- (4) To further observe pavement distresses during the NAPTF vehicle's trafficking tests.

The particular device acquired was a KUAB 2m Heavy Weight Deflectometer (HWD). The KUAB 2m HWD operates on the principal of dropping weights on a series of hard, rubber buffers separated by a second series of weights and buffers which are connected to a loading plate resting on the pavement surface. This two mass system results in a consistent and uniform, half-wave sine curve for the loading pulse. The loading plate is segmented into quarters to ensure the loading force is evenly distributed. Weights and buffers can be added, or removed as necessary, to adjust peak load and loading time. The loading pulse shape is also influenced by the combination of weights and buffers utilized. The drop heights can be adjusted to control the peak load. The KUAB 2m HWD has seven seismometers each utilizing a spring for reference and linear voltage differential transformer (LVDT) for the sensor.

The standard configuration for the FAA's heavy weight deflectometer has been established with the following parameters: the segmented 12 inch (30.5 cm) loading plate, a pulse width of 27-30msec, and four drop heights consisting of a 36,000 pound "seating drop" followed by impact loads of 12,000, 24,000, and 36,000 pounds loading force. The first drop of 36,000 pounds of force is considered a "seating" drop, and is not used in the analysis. The peak load and deflections are recorded for all four drops along with air and pavement surface temperatures. One seismometer is placed at the center of the load plate (D0), another is placed 12 inch (30.5 cm) in front of the load plate (D-1), and the remaining sensors are located behind the load plate at 12 inch (30.5 cm) intervals, (D1, D2, D3, D4, D5, in figure 3).

The FAA's HWD machine was originally calibrated by the manufacturer, and routinely checked by the operator. After thousands of test drops, the HWD was sent to Roadway Inventory & Testing Section, DOT of Pennsylvania for

calibration on May 11, 2000. All of the seven deflection sensors received excellent precision records. The Final Calibration Factors for the sensors from number -1 to 5 (figure 3) are 1.006, 0.998, 0.998, 0.999, 1.006, 1.003 and 0.994. The results indicate that the maximum error of the sensor reading is about 0.6%, significantly lower than the 2% required by ASTM standard D-4694 (ASTM 1987). The loading system also received satisfactory results in the first test with loading calibration factor 1.001. Therefore, no reason has been found to challenge the reliability of HWD results received by using the FAA's KUAB machine.

The FAA's HWD operator is responsible for completing a pre-test checklist on a daily basis during the traffic tests. A visual inspection of the device is accomplished to check all fluid levels, to ensure no worn, or cracked buffers are utilized, to check for loose wiring or cables, and to check the drop guides and electro-magnet surfaces for the presence of dirt, dust or excessive grime buildup. A series of five drops is conducted to verify that the relative load force variation is less than 3%. This also helps to minimize the error of operation of the device and associated computer software during operational testing. The seismometers and distance measuring sensors are checked and adjusted on a monthly basis. These adjustments assist with negating temperature variations on the measurement devices.

Data Location

The Heavy Weight Deflectometer data collected and utilized for this report, as well as other HWD tests conducted at the National Airport Pavement Test Facility, have been included as part of the National Airport Pavement Test Facility database. The database is available for download or direct access at the FAA Airport Technology Research and Development Branch web site: www.airporttech.tc.faa.gov. The sensor designations differ in this report from the database. This was accomplished for uniformity with accepted past practices during analyses

HWD Results At Center Of Slabs

The mean of measured D0's and D5's at the center of the slabs are presented in Figure 4. The Coefficients of Variation of data in each test item are between 3.7% to 10.5%, with most lower or close to 7%. The comprehensive pavement structural stiffness indicated by D0 and sub-grade stiffness by D5 are relatively uniform within each item. D0 bars indicate that the pavement structural stiffness of item HRS is much higher than that of item LRS and MRS because of the much stronger sub-grade in HRS (CBR = 30 to 40 for HRS against CBR = 7 to 9 for MRS and CBR = 3 to 4 for LRS).

Figure 4 also indicates that no significant change of D0s occurred from June to December of 1999. Comparison of D5's also indicates that the sub-grade

strengths are different (as expected) for the three test items and they have no significant change within each item from June through December of 1999, either.

Heavy weight deflectometer data collected by using the FAA's KUAB device was used to back-calculate pavement relative stiffness l , Winkler Foundation stiffness k (pci) and concrete layer elastic modulus E . The results are listed in Table 2. The back-calculation method was proposed by Hall etc, 1991 and the formula may also be found in Reference FHWA-HI-94-021.

The back-calculated pavement properties are strongly related to the model used for back-calculation. It can be found that the k value for HRS is higher than the value accepted for pavement design by the current FAA specification. FAA Advisory Circular "Airport Pavement Design and Rehabilitation" (AC 150/5320-6D) does not allow a value of k higher than 500 pci). E values of the PCC layer are also much higher than the concrete E value used to develop the FAA specification of 27.6 MPa (4,000,000 psi). It should be noted that the objective of the design specification is essentially different from that of the pavement response analysis. The design specification model intends to estimate pavement performance under variable traffic and environmental effects within its service life (usually 20 years for most airport pavements). The response model is utilized to predict response of a specific pavement under a well-defined load and/or environmental excitation(s) within a much shorter time span, from a few seconds to a few days. Therefore, for pavement life performance and short time load responses, different values of E and k are reasonable.

Based on the laboratory tests for the core samples from the PCC slabs in Feb. 2000, the average of the E values was close to 41.3 Mpa (6,000,000 psi). This is also much lower than the back-calculated E values for the different test items. It should be noted that the elastic modulus (E) from the laboratory tests is a parameter solely related to the material property. However, the back-calculated E value is no more a unique property for material. It becomes a parameter depends not only on material property but also on the structural model plus the boundary conditions employed in the back-calculation. Therefore, if material properties are used in pavement response analysis, the employed structural model should be compatible to the model used in back-calculating these material properties. Otherwise, the predicted responses would be difficult to match the measured ones.

The back-calculations were conducted based on the mean value of sensor readings in fifteen slabs of each PCC pavement item. In addition, the following values are used for response analysis in this paper.

$K = 54.2, 108.4$ and 178.9 MPa/m (200, 400 and 600 pci) for LRS, MRS and HRS respectively. And $E = 55.2$ MPa/m (8,000,000 psi) is used in response analysis for all three items

HWD Results At Joints And Corners

The mean values of D_0 at joints and corners are given in Figure 5. Three groups in figure 5 show the mean D_0 s received from PCC items LRS, MRS and HRS

respectively. Within each group, the first, third and fifth from the left present average D0s at middle of transverse and longitudinal joints, and at corners respectively received in June 1999. The second, fourth and sixth in each group are calculated from the data received from October 1999 to Jan. 2000. The joint tests were done only on one side of the joints in June 1999. The data collected from both sides of all joints is available for the period from Oct.1999 to Jan. 2000.

Comparison of the first and third bars in each items indicates that in June 1999 (3 to 4 months after the construction), the mean of D0 measured at transverse and longitudinal joints are similar though all transverse joints are dummy and all longitudinal joints are doweled. This is probably due to that the joints had not been completely cracked at that time. All D0s received at joints in October and later are much larger than the values received in June. The deflections in the middle of transverse joints are increased more than those at longitudinal joints because the later (doweled joints) provides relatively better load transfer especially in winter after the joints being cracked.

Comparison of the first, third and fifth bars shows that the measured D0 at corners were higher than those at joints in June 1999. However, since October 1999, the difference among deflections at transverse, longitudinal joints and at corners became much more significant. This is true for the comparison of data for all three PCC items, however, the higher strength of sub-grade, the larger difference was observed. For example, the ratios between mean D0s at corners on October or later and on June 1999 for LRS, MRS and HRS are 1.62, 1.87 and 1.98 respectively. We believe that these differences are partially contributed by the different slab curling up situations.

All HWD tests done between Oct. 1999 to Jan. 2000 were conducted by four drops: 36000, 12000, 24000 and 36000 lbs except the tests at slab center in item LRS. To investigate the deflection behavior, we calculate difference of the mean of center deflections under 24000 and 12000, and define it as the deflection due to the load increment between 12 to 24 kips. Similarly we calculated the deflection due to the load increment between 24 to 36 kips. The results are shown in figure 6 and indicate that the relationship between load and deflection are almost linear at center. However, the results shown in figure 7 indicate the load-deflection relationship is nonlinear at the corners.

The significant increase of deflections at joints and corners measured in October or later and the strong nonlinear feature between deflection and load magnitude mostly caused by upward curling of the concrete slabs.

Load Transfer Capabilities Of Joints

Figure 8 shows variation of average load transfer capability (LTD) defined by the ratio between deflections on two sides of the transverse and longitudinal joints in June, 1999 and October 1999 or later. First, comparison between the column 1

and 3 of each group indicates that the LTDs at transverse joints were slightly higher or equal to those of the longitudinal joints in June 1999. However, comparison between the columns 2 and 4 shows that the LTDs at the transverse joints were significantly lower than those at longitudinal joints in Oct. 1999 or later. Second, comparison between the columns 1 and 2, 3 and 4 in figure 8 indicates that all joints lost more or less load transfer capabilities from June to Oct. 1999. However, the transverse joints lost much more than the longitudinal joints. The average differences at longitudinal joints are 0.06, 0.05 and 0.07 for LRS, MRS and HRS respectively while those at transverse joints are 0.195, 0.27 and 0.23. The transverse joints are dummy and the longitudinal joints are doweled. The lower temperature in October and later led all joints to be cracked which significantly reduced the load transfer capability for the transverse joints (dummy) while the load transfer capability of the longitudinal joints were still relatively high through the dowels.

Joint load transfer capabilities also depend on the moving direction of the load. For example, the LTD from point T01 to T02 (figure 2) is defined as D_1/D_0 (figure 3) when the center of HWD loading plate locates at T01 while the LTD from T02 to T01 is defined as D_1/D_0 when the load is applied at T02. Therefore, we divide all LTDs of transverse joints into two groups, one involves the higher and the other involves the lower LTDs of the two values at each joint. figure 9 presents the means of higher and lower LTDs for transverse and the longitudinal joints. It can be seen that the difference between the higher and lower is large for the transverse joints but small for the longitudinal joints. The dowel bars provided similar load transfer capabilities on both directions especially after the joints are totally cracked. Since the joint cracks are not always vertical, the dummy joints show significant different load transfer capabilities in two directions.

Sum Of Deflections (SD) On Two Sides Of Joints

Hammon et al, 1997 suggests to use following formula for a two-slab system which fully contacts sub-grade and is linked by an elastic joint defined by a shear model:

$$d_L + d_U = d_E \quad (1)$$

where

δ_E is the load induced deflection at a joint edge when the joint load transfer capability is zero. It is often called “free edge deflection”.

δ_L is the deflection at the point on the loaded side of slab

δ_U is the deflection at the point on the unloaded side of slab

Before the load is applied, vertical locations of the two “points” on loaded and unloaded side of the joint are identical. If a joint has infinitely large load transfer capability, the deflections on two sides of the joint would be the same even if a load is applied on one side of the joint. None of any real joint has infinitely large

load transfer capability, so that the deflection on unloaded side is always more or less smaller than that on the loaded side. The ratio of δ_U and δ_L is defined as Load Transfer capability by Deflection (LTD) of a joint. However, Very few people have been interested in the information provided by the Sum of Deflections (SD).

Guo 1999 presents numerical calculations for a six-slab system and verifies that equation (1) is true when the slabs fully contact the foundation.. After substituting calculated δ_L and δ_U by finite element program into equation (1), the author find that the difference between the results received from Eq (1) and the free edge deflection directly calculated by the finite element program is smaller than 1%.

Equation (1) indicates a very important feature: the SDs are independent to the load transfer capability of a joint. For all joints with zero to full load transfer capability, the SD is a constant that equals to the free edge deflection of the system under the same load. However, equation (1) is true only if the slabs are always fully contacted to the foundation. Unfortunately, all slabs are always curled, more or less, up or down. Therefore, we have to know how SDs vary under warping or curling condition.

In FAA PCC test pavements, deflections on both sides of all joints were measured. Figure 10 shows the variation of the sum of deflections on two sides of transverse joints. In June of 1999, the SDs in LRS, MRS and HRS were 25.0, 25.1 and 26.3 mils (0.635, 0.638 & 0.668 mm) respectively. About four to five months later, the corresponding SDs increased into 28.8, 32.9 and 43.4 mils (0.732, 0.836 & 1.102 mm) respectively. The previous analysis verifies that curling up had been increased from summer to later fall and winter, and the stronger supporting system led to larger curling up. Therefore, figure 10 indicates that the SD increases when the curling up of slab corners increases.

Figure 11 summaries the different characteristics of LTD and SD. As shown in figure 9, the load transfer capability of a joint depends on the traffic direction. There exist significant difference between the LTD values in two directions for transverse joints. The left columns in figure 11 are the average ratios of LTDs between the high and low groups. The average load transfer capability on the high side is about 38 to 42 % higher than that on the low side for the transverse joints in the three testing PCC items. The second columns of figure 11 present the ratio of SDs on the same two groups. It is interesting to see that the SDs almost do not change from the high LTD side to the low LTD side, or, the SDs seems independent to the LTDs of the joints. This is true for all three PCC testing items.

Summary

The HWD data collected from the FAA' NAPTF test pavements indicates that significant increase of curling up of the slabs was observed from the summer to the winter 1999. The slabs on the higher strength of sub-grade were curled more than those on the lower strength of sub-grade. Increase of deflections at joints

and corners, plus the nonlinear relation between the deflection and HWD load intervals verified the increase of curling up. The test data also shows that the sum of deflections on two sides of a joint (SD) is almost independent to the load transfer capability defined by the ratio between deflections on the unloaded and loaded sides. Since the SD is only sensitive to the curling of slabs, it may be used to find the true curling state of a PCC pavement.

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P501, H=28cm(11")	P501, H=24.8cm(9 3/4")	P501, H=22.9cm(9")
P306, H=15.6cm(6 1/8")	P306, H=14.9cm(5 7/8")	P306, H=15.2cm(6 ")
P154, H=21.3cm(8 3/8")	P154, H=21.9cm(8 5/8")	P154, H=16.8cm(6 5/8")
Sub-grade CBR=3-4	Sub-grade CBR=7-8	Sub-grade CBR=30-40
LRS	MRS	HRS

Figure 1 Cross Section of Three PCC Items

NORTH

Slab 1	Slab 2	Slab 3	Slab 4	Slab 5				
Lane 1								
C01	T01 T02	C02	T03 T04	C03	T05 T06	C04	T07 T08	C05
L01	N01 N02	L03	N05 N06	L05	N09 N10	L07	N13 N14	L09
L02	N03 N04	L04	N07 N08	L06	N11 N12	L08	N15 N16	L10
Lane 2								
C06	T09 T10	C07	T11 T12	C08	T13 T14	C09	T15 T16	C10
L11	N17 N18	L13	N21 N22	L15	N25 N26	L17	N29 N30	L19
L12	N19 N20	L14	N23 N24	L16	N27 N28	L18	N31 N32	L20
Lane 3								
C11	T17 T18	C12	T19 T20	C13	T21 T22	C14	T23 T24	C15

Figure 2 Definition of HWD testing location for PCC items LRS, MRS and HRS.

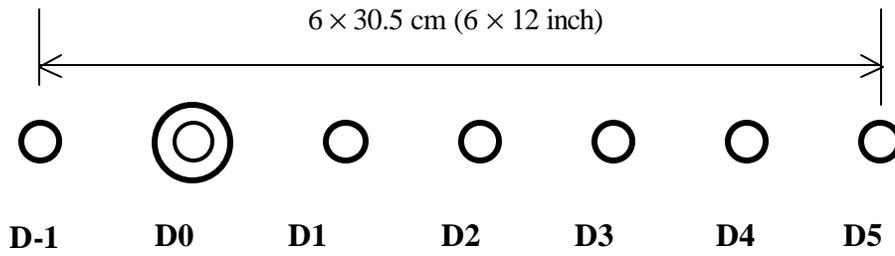


Figure 3 Locations of Deflection Sensors. D0 is at The Center of Loading Plate

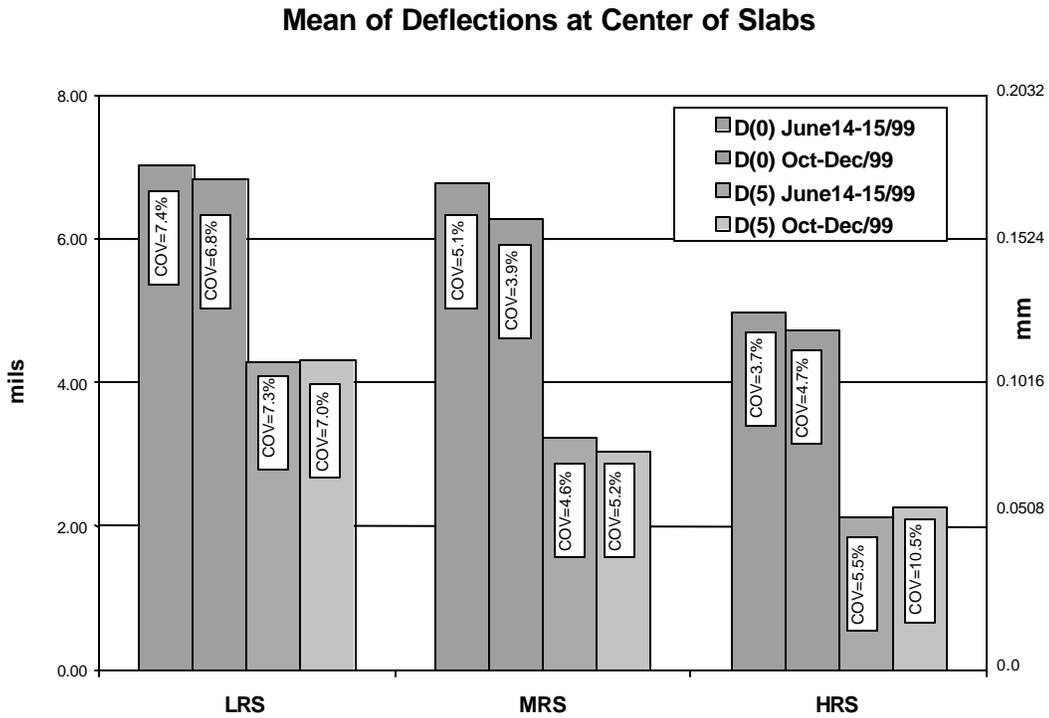


Figure 4 Deflections D0 and D5 at the Center of Slabs with Coefficient of Variation in Percentage

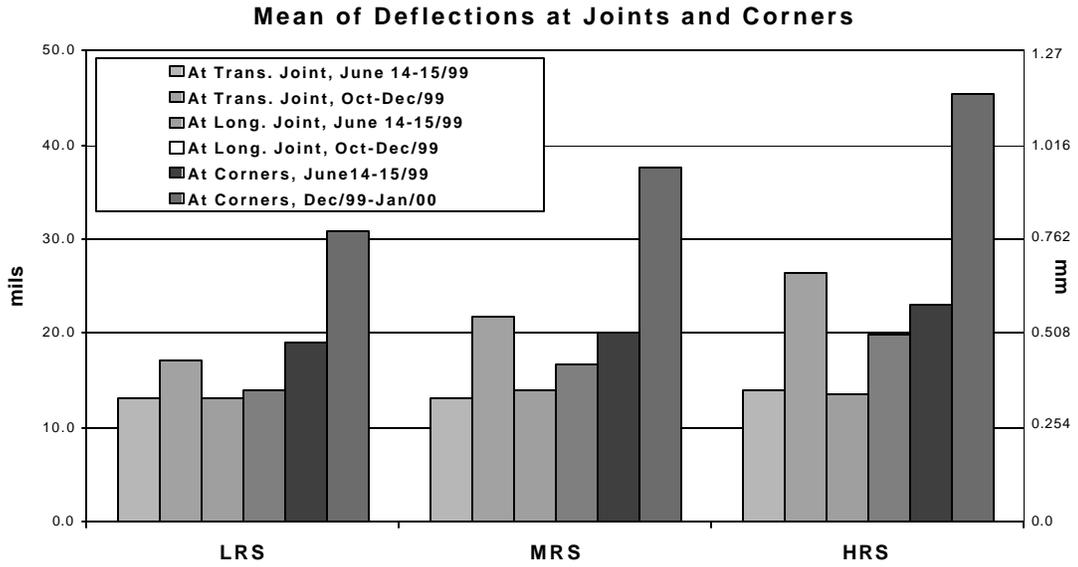


Figure 5. Deflection D0 at the Joints and Corners (Load = 24,000 lbs)

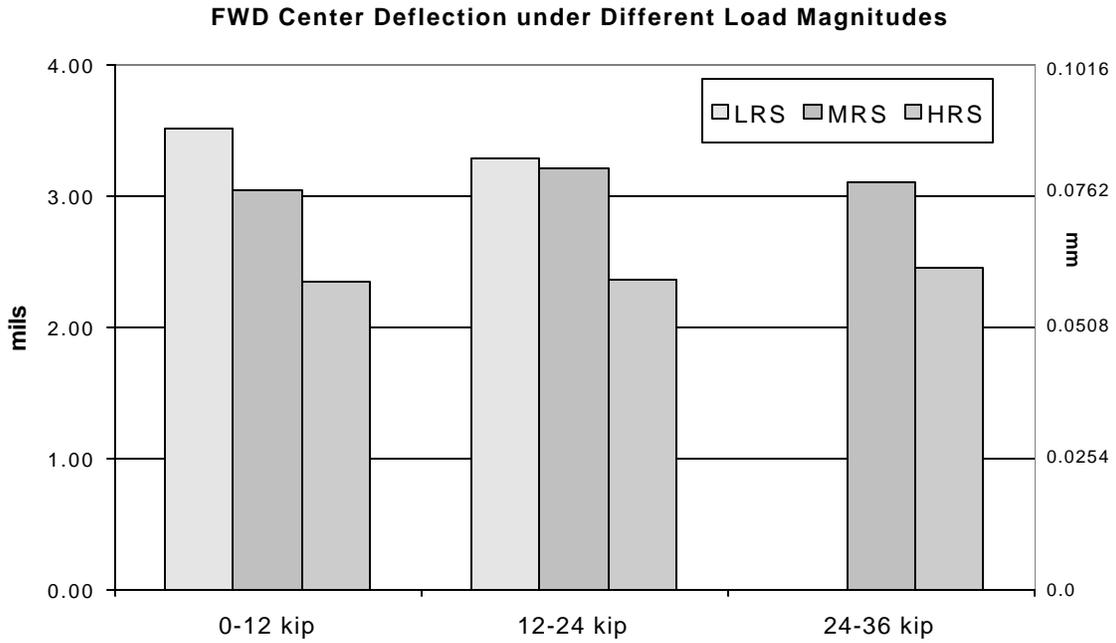


Figure 6. Linear Relationship of Deflections at Slab Center under Different Loads

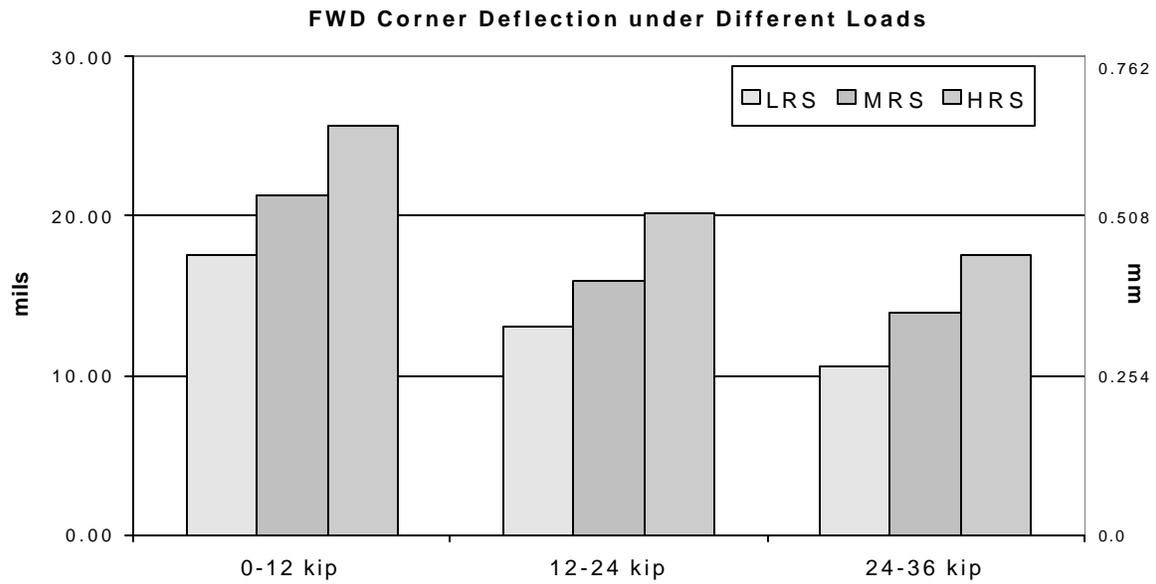


Figure 7. Nonlinear Relationship of Deflections at Slab Corner under Different Loads

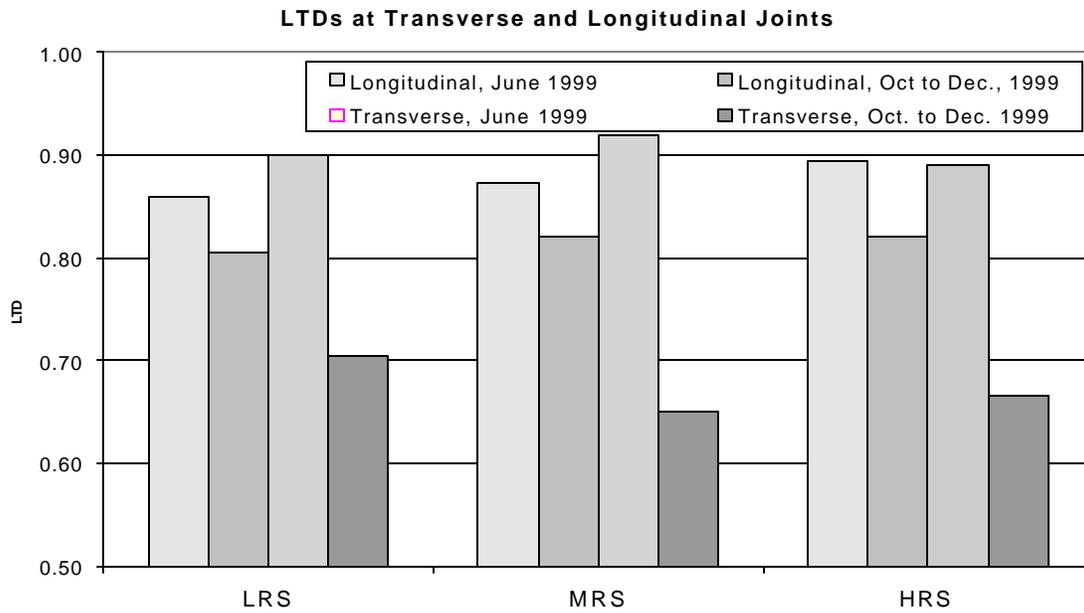


Figure 8. Load Transfer Coefficients Defined by Deflection Ratio

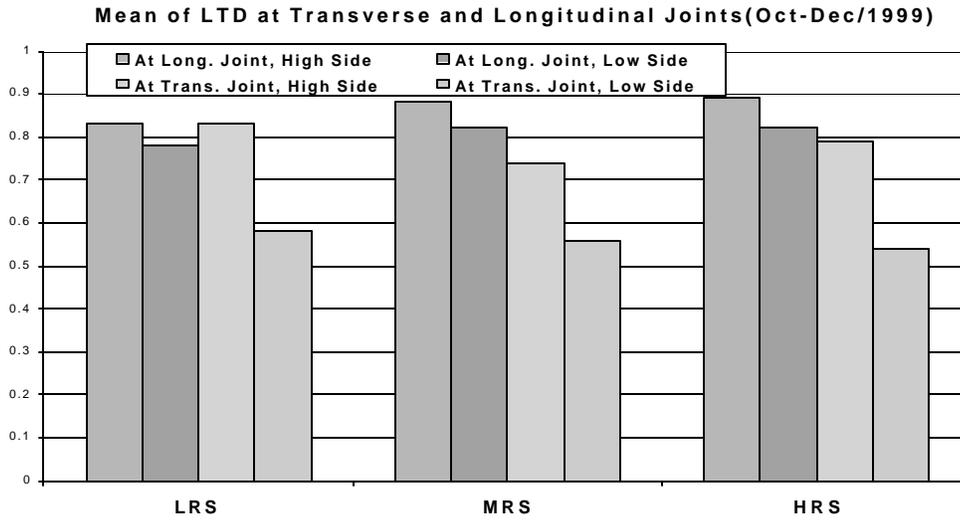


Figure 9. Load Transfer Capability Varies by the Transfer Direction

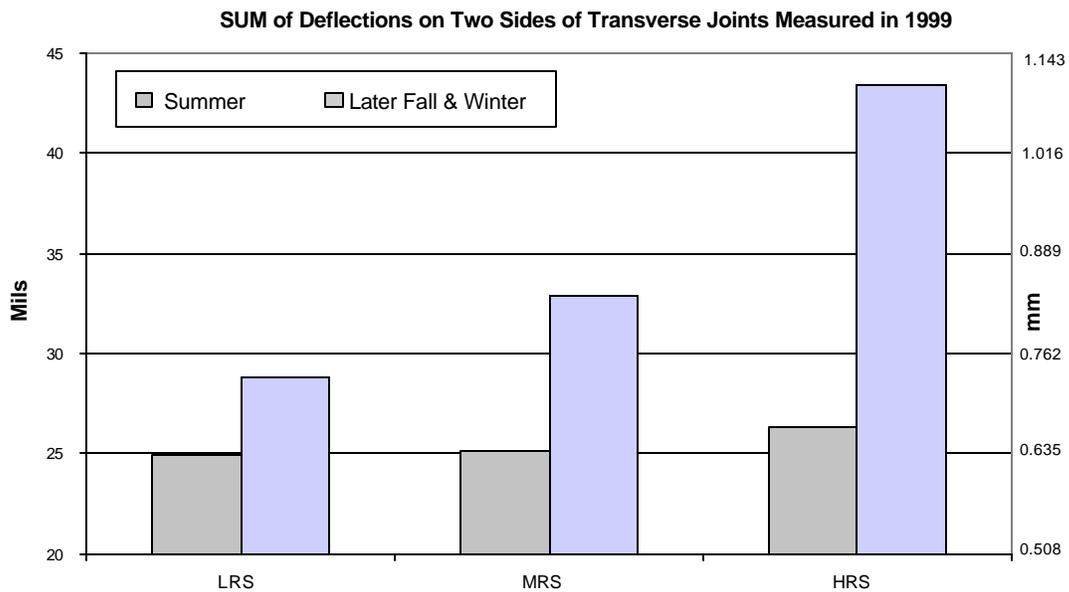


Figure 10. Variation of SUM of Deflections (SDs) on Two Sides of the Transverse Joints

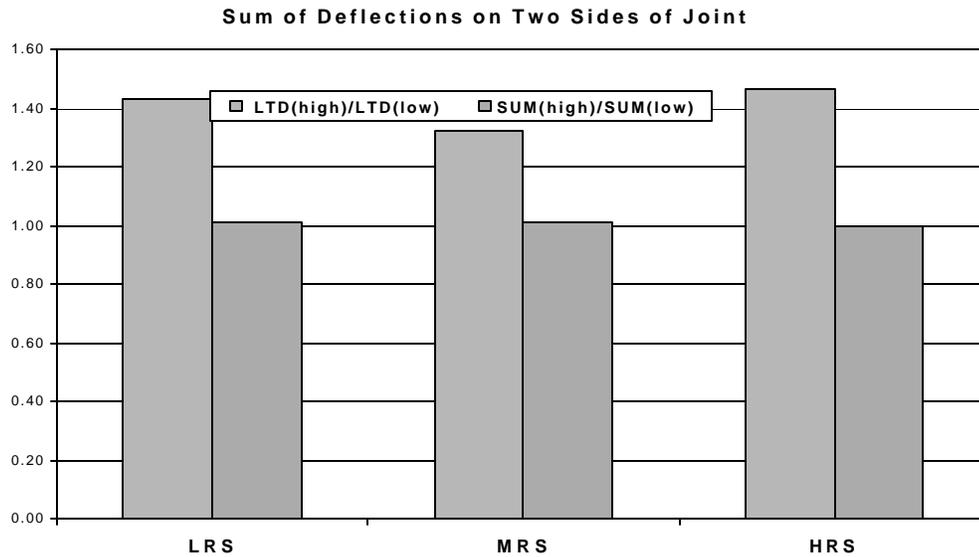


Figure 11 Comparison of the Properties of Sum of Deflections and Load Transfer Coefficient

TABLE 1. TEST DATE & TEMPERATURES FOR HWD TESTS ON PCC PAVEMENTS

Test Items	Date	Measured Temp. Range, °F
LRS, by ERI	6/14/99	67 – 74
MRS & HRS by ERI	6/15/99	67 – 72
LRS, Center & Joints only	10/22/99	51 – 66
MRS, all	12/21/99	50 – 52
HRS, all	12/22/99	43 – 48
LRS, Corner only	12/23/99	46 – 47
LRS, Corner only	1/28/00	33 – 36

TABLE 2. BACK-CALCULATED PAVEMENT PARAMETERS

	l , cm (inch)	k , MPa/m (pci)	E , MPa $\times 10^3$ (psi $\times 10^6$)
LRS	118 (46.6)	54.5 (201)	57.5 (8.34)
MRS	87.9 (34.6)	107 (396)	50.7 (7.36)
HRS	79.2 (31.2)	178.9 (660)	68.3 (9.90)