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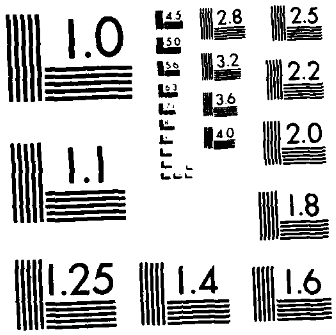

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Longitudinal Joint Systems in Slip-Formed Rigid Pavements

Volume V - Summary of Field Test Results from Chicago O'Hare International Airport

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Urbana, Illinois

February 1983

Final Report

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16. Abstract

This fifth and final report dealing with the design and construction of longitudinal joints in slip-formed concrete pavements presents a summary of findings from tests conducted at Chicago O'Hare International Airport. Tests performed include nondestructive tests performed with several types of vibrator and impulse loading devices, and loading with commercial aircraft. Pavement responses under the various loading conditions were measured using the geophones normally used with the NDT type equipment, and with deflection gages and strain meters installed in the pavement slabs. In these tests particular emphasis was placed on measuring the load transfer efficiency across transverse and longitudinal joints and the effect of load transfer efficiency on the behavior of the pavement system. Results from these tests clearly show the importance of load transfer on certain types of distress in concrete pavements, as loss of load transfer efficiency can lead directly to corner breaks and pavement faulting.

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METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

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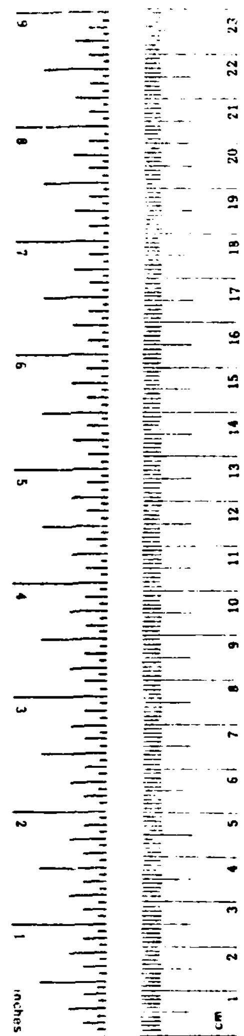
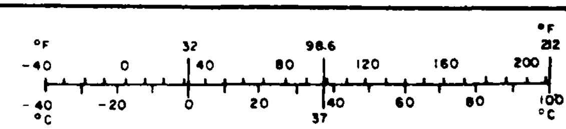


Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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EVALUATION OF PAVEMENTS
AT CHICAGO O'HARE
INTERNATIONAL AIRPORT

Effective management of pavement systems and scheduling of funds for pavement rehabilitation require the projection of maintenance and rehabilitation needs over a period of years. To accomplish this with any degree of reliability and assurance requires documentation of the present pavement condition and the responses of the pavement to loads. These data are necessary to predict the probable condition of the pavement a number of years into the future.

A number of test methods have been proposed to evaluate the present condition of a pavement. These include nondestructive (NDT) as well as destructive test methods. The best test methods and best equipment for NDT tests have been the subject of many heated discussions by engineers at technical meetings, seminars and workshops around the country. Currently there is a Transportation Research Board (TRB) Task Force trying to unravel the problems associated with NDT equipment and test procedures and make recommendations for future use of these devices.

In 1975 the engineers for the City of Chicago and more specifically the chief airport design engineer, Mr. Don Arntzen, undertook a systematic program to evaluate some of the proposed NDT equipment and test methods, and to compare the results with the actual performance of the pavements tested over a number of years. Test procedures and approaches used in this study included an annual evaluation of the pavements using the U.S. Army Corps of Engineers 16 kip vibrator plus other NDT equipment and procedures on a selective basis; instrumenting a number of pavement sites and measuring the pavement responses under both vibratory (NDT) equipment and aircraft type loading; and comparing of the responses obtained using various NDT devices and aircraft loading with theoretical results and with the actual performance of the pavements over time. Results from these studies have provided valuable insight into the behavior and performance of portland cement concrete (PCC) pavements, and have provided valuable guidance on the test procedures and data needed to evaluate the probable future performance of these pavements.

While most of the cost of this test program have been borne by the City of Chicago, other agencies have provided some monies and support. Specifically, the USDOT provided funds for some of the instrumentation at selected sites and analysis of some of the data through research contract DOT-FH-11-7484 with the University of Illinois. The State of Illinois provided support by providing the City of Chicago with a NDT device (Road Rater Model 2008) and an instrumentation van with an operator to record data from tests on instrumented pavement sites.

The volume of data collected on this project is too great to present in detail in this report. Most of the data have been presented to the City of Chicago in the form of reports and recommendations to the airport engineers. In this report, only a summary of the findings will be given along with the conclusions from the study and recommendations for future evaluation procedures. A detailed description of the instrumentation installed at several sites, the tests performed at these sites, and some of the more pertinent findings are given later in this report. Other details are given only as needed to support the conclusions reached and the recommendations contained herein.

Test and Evaluation Programs

The initial test program which is considered to be a part of this program was started in 1975. The initial program was basically an annual evaluation of the pavements using the U.S. Army Engineers WES Vibrator using the standard procedure developed by engineers at WES to evaluate PCC pavements [1]. This procedure consists essentially of testing the pavements under a range of loads (usually from 0 to 30 kips) at fixed frequency (normally 15 Hz) and determining a dynamic stiffness modulus (DSM) for the pavements [1]. Under this mode of testing, the test loads are placed at an interior point on a PCC slab at some distance from all edges and joints.

Starting with the 1977 NDT Program, additional tests were added to evaluate the relative deflection across the joints and cracks at a number of locations.

In 1978 instrumentation was designed and installed in the pavements at 5 locations shown in Figure 1. A description of the instrumentation installed and the locations on the pavements is given later in this report. The basic thrust of the instrumentation was to provide data to validate the results from theoretical analyses of the PCC pavements, and especially to evaluate the effect of joints and load transfer across the joints on the behavior and performance of the jointed PCC pavements and across the cracks for continuously reinforced pavements.

Most of the instrumentation packages had been installed and verified by the fall of 1978 and tests were conducted on the instrumented sites. During this first fall, only aircraft loads in a normal operation mode were applied to the instrumented sites. An attempt was made during these tests

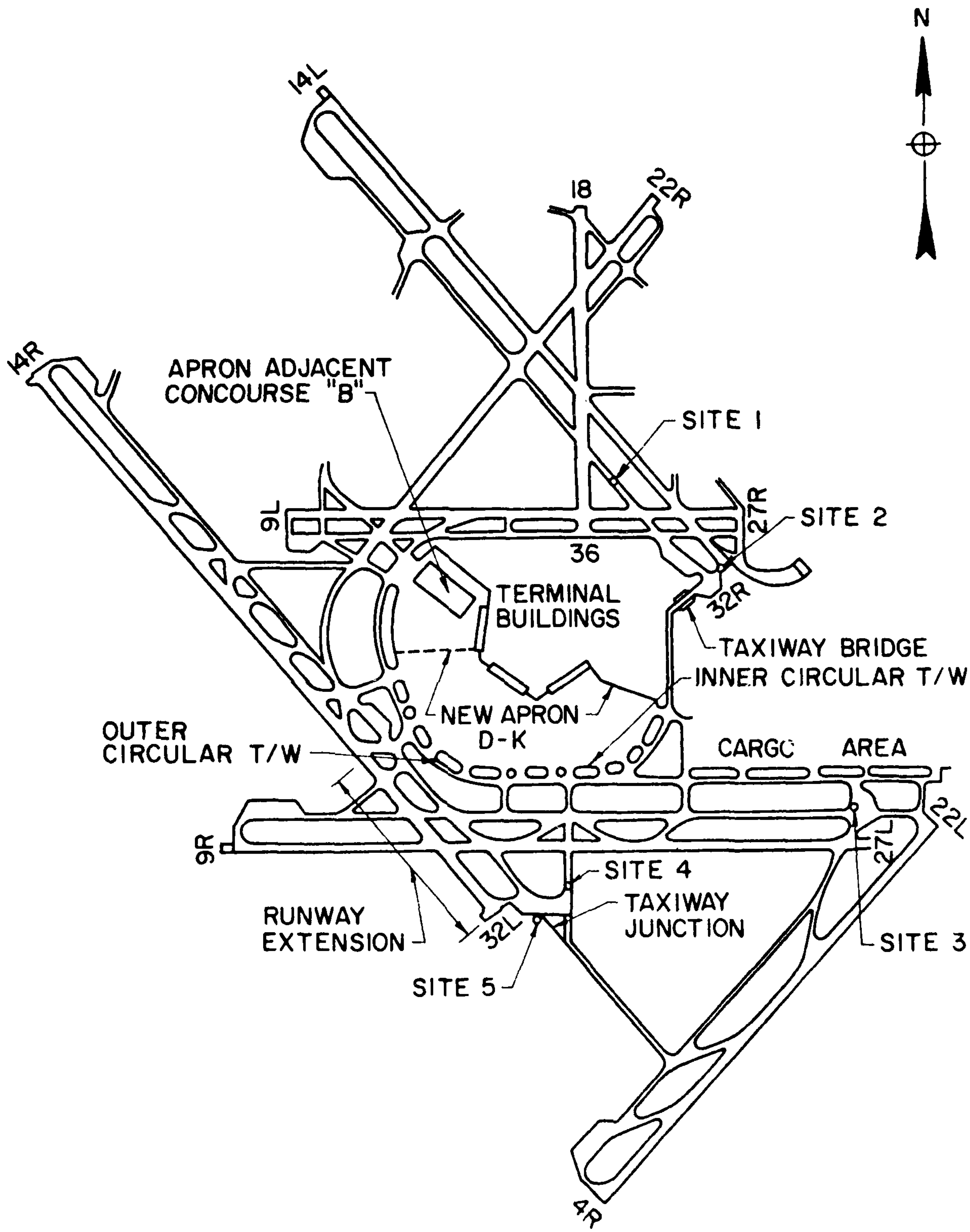


Figure 1. Layout of O'Hare International Airport Pavements with the Instrumented Sites Designated

to monitor the instruments from an off-pavement site using an instrumentation trailer supplied by the Illinois Department of Transportation (IDOT). It was the intent that as the aircraft, under normal operating conditions, ran over the instrumented pavements the relative location of the aircraft gear could be determined while the response of the pavement to the aircraft could be recorded on magnetic tapes. This concept worked fine in theory but logistic problems and difficulties encountered in determining the exact location of the aircraft gear relative to the instruments caused this approach to be abandoned for subsequent tests.

During May 1979, at the time scheduled for the normal NDT evaluation of the pavements, the instruments were again installed in the pavements and the pavements tested under loads consisting of three NDT devices and commercial aircraft. The NDT devices used were the U.S. Army Engineers Waterways Experiment Station (WES) 16^k vibrator, a Road Rater model 2008 with an 8^k maximum capacity for dynamic loading, and a dynaflect. The Road Rater model 2008 was supplied by the Illinois DOT and the Dynaflect by Region 15 of Federal Highway Administration (FHWA). Aircraft loads applied during the 1980 testing program were applied using a 727 aircraft supplied by United Airlines with a mechanic to guide the aircraft over the instrument installed in the pavements.

In the fall of 1980 a new NDT testing device known as a Falling Weight Deflectometer (FWD) became available. In October of 1980 the instruments were again installed at some locations and comparative tests made using the FWD and WES vibrators.

Also during 1980, instruments were installed in the prestressed pavement overlay on the east end of Runway 9R-27L. These pavements were tested under the gear loads of a 727 aircraft guided over the site just prior to opening the pavement to traffic in August 1980.

The final series of tests on the pavements at O'Hare were completed in the spring of 1982. These later tests were limited to a NDT evaluation of Runway 14R-32L using the FWD device recently acquired by the City of Chicago. In this test program most of the tests were conducted near the edges and joints in the pavements with emphasis on measuring the load transfer effectiveness of the various types of joint load transfer devices, and with detection of voids under the slabs near the joints. In this case some NDT tests were run both before and after pressure grouting under the slabs.

Instrumentation of Test Sites

The instrumentation packages installed consisted of elastic wire strain meters manufactured by Carlson Instruments of California (Carlson Gages) to measure the strains in the Portland Cement Concrete (PCC) slabs, and whip type devices installed in permanent boxes to measure deflections. Details of both the Carlson Gages and the whip devices and installation procedures are given in Appendix A. Packages of these gages were installed at several sites at Chicago's O'Hare International Airport at the locations shown in Figure 1. The instrumentation package layout shown in Figure 2 was placed at Sites numbered 1, 2, 4 and 5 and the layout shown in Figure 3 placed at Site 3.

The 5 sites chosen for instrumentation were selected to represent a range of pavement systems. These included the following:

Site 1: A plain, jointed, PCC slab tapered from 24 to 27 inches in thickness with doweled transverse and tied longitudinal joints on an

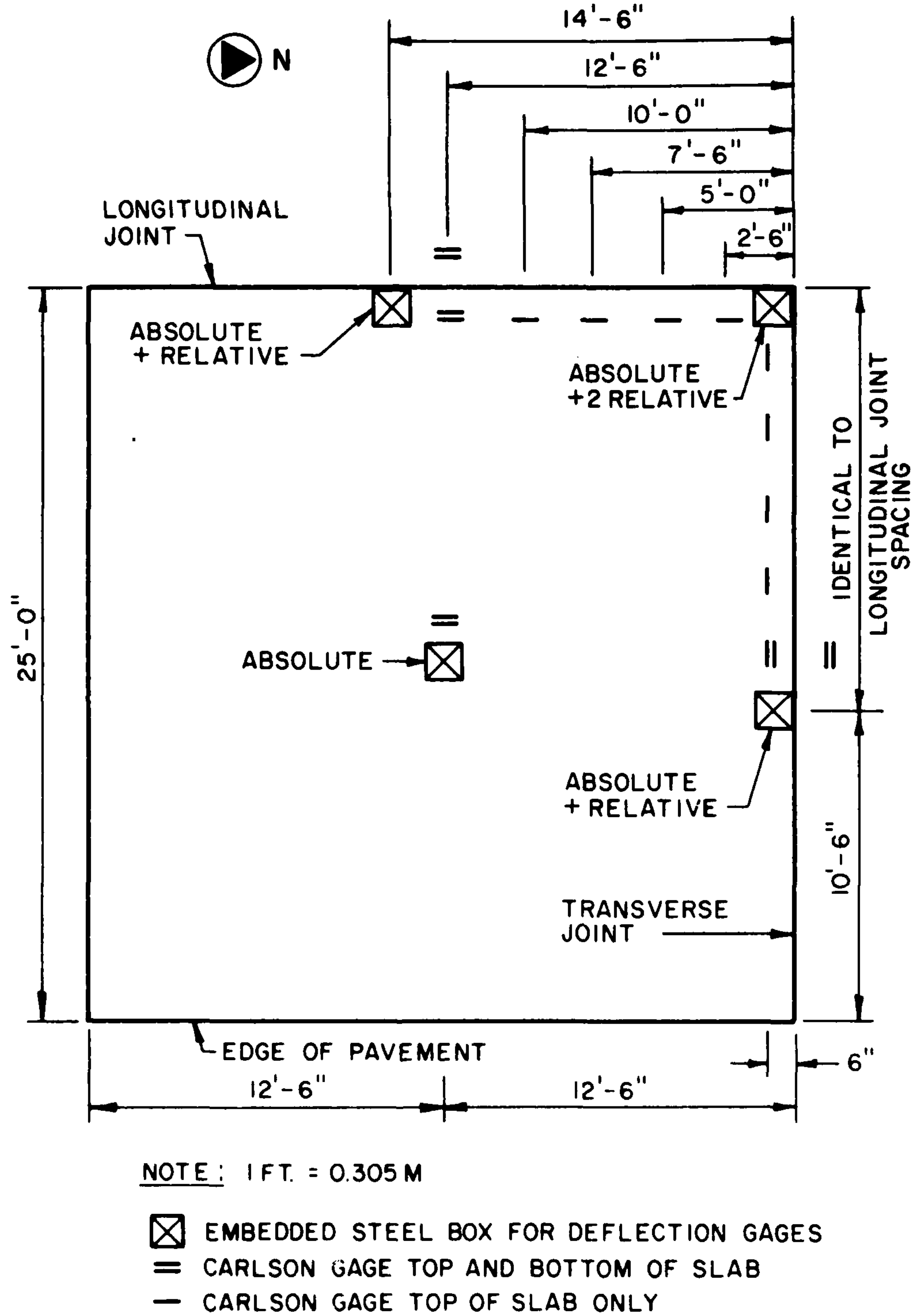
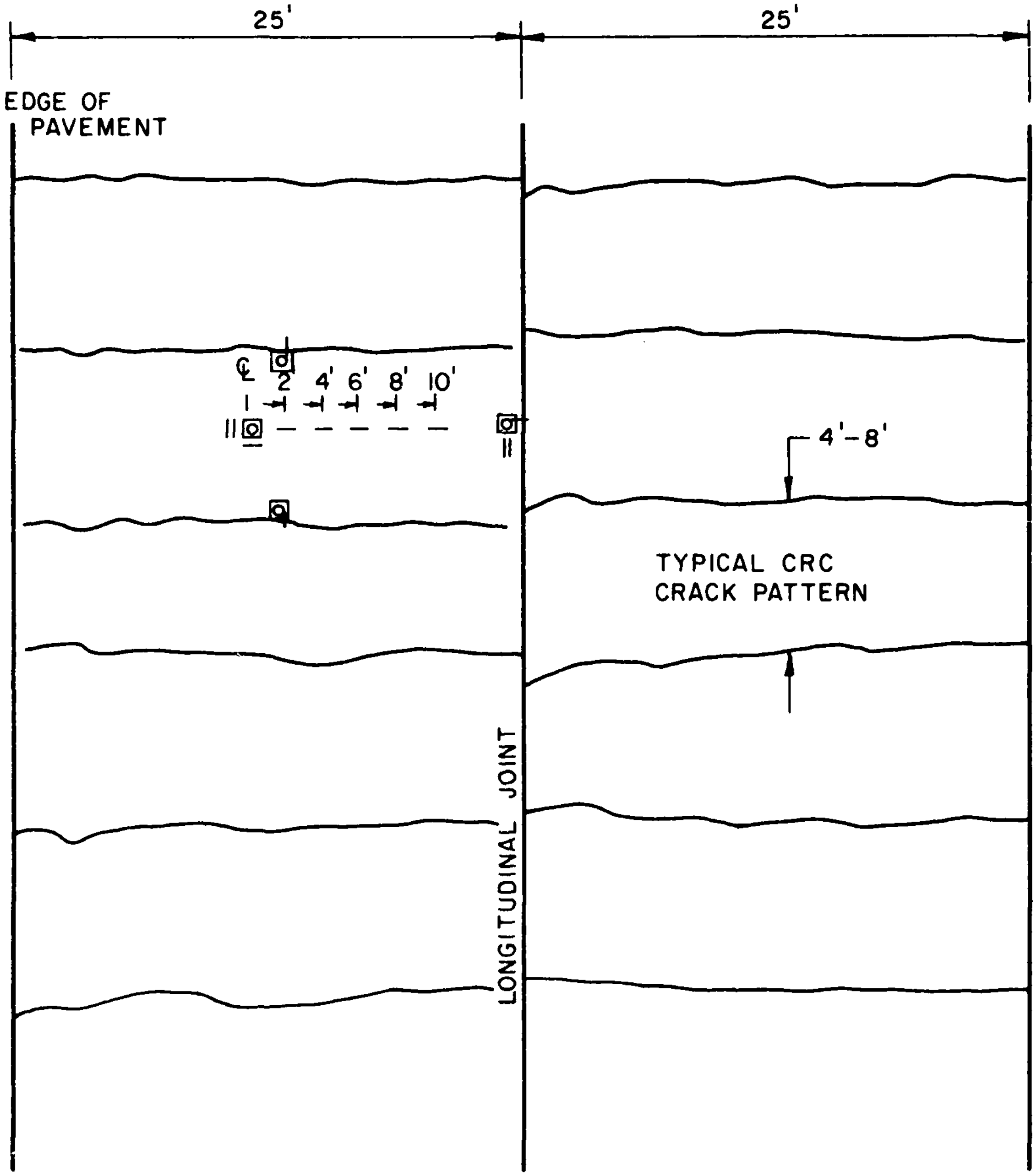


Figure 2. Typical Instrumentation Layout at One Site at O'Hare Airport



ABSOLUTE DEFLECTION
RELATIVE DEFLECTION
= CARLSON GAGES TOP AND BOTTOM OF SLAB
— CARLSON STRAIN GAGES TOP OF SLAB ONLY

Figure 3. Typical Instrumentation Plan for CRC Pavements, Site #3

asphalt concrete (AC) stabilized subbase 18 inches thick (new construction). This pavement had carried very little traffic at the time of testing.

Site 2: A jointed, plain, PCC slab, 18 inches in thickness, with doweled transverse and tied longitudinal joints on an AC stabilized subbase 18 inches thick (new construction; no results are available from this site).

Site 3: Continuously reinforced concrete (CRC) pavement, 12 inches in thickness on 12 inches of unstabilized granular subbase. This pavement had been in service for approximately 10 years at the time of testing, and has carried significant aircraft traffic.

Site 4: Reinforced, jointed, PCC slabs 15 inches in thickness with 50 foot joint spacing with tied longitudinal and doweled transverse joints, placed on 12 inches of unstabilized granular subbase. This pavement was constructed in the 1960's. This test site is directly in the wheel path of one of the most frequently used taxiways at O'Hare.

Site 5: Reinforced, jointed, PCC slab 18 inches in thickness with 50 foot joint spacing, tied longitudinal and doweled transverse joints, placed on 6 inches of AC subbase. This pavement was constructed in the 1960's. This pavement was at the edge of an apron used as a holding area for aircraft using runway 14R-32L and had very little traffic at the time of testing.

While the facilities for testing were installed at all 5 sites, only Sites 3, 4 and 5 were tested extensively as part of this study.

Presentation of Findings

The NDT data collected by the Corps of Engineers are presented in reports listed in References 2, 3, 4 and 5. Details of the test procedures used are given in the above references. The data from the tests have been reduced to a dynamic stiffness modulus (DSM) which represents the slope of the line obtained when the magnitude of dynamic force is plotted against the transient pavement deflection. Such a slope represents the resistance of the pavement to deflection under load and was generally measured at an interior point on a slab; that is, at a point on the slab away from all cracks or joints.

For the primary runways at O'Hare International Airport (OIA) the NDT was run at a number of locations (approximately 20) per runway. Each time the tests were run, an attempt was made to conduct the test as close as possible to the test location from the earlier tests. The data from each test location was reduced to a DSM value and reported as such. All tests were conducted at about the same time of the year, usually May or June except for the 1980 tests which were conducted in October and November. Figure 4 shows a plot of the DSM along Runway 4R-22L taken in three successive years.

Figure 5 shows a plot of the average DSM values for the six runways at OIA for the period from 1975 through 1980. The curves as plotted indicate no particular trend, but show a significant variability with time. Specifically, Runway 4R-22L has the most uniform response with the average DSM, varying from approximately 4700 in 1976 to about 5300 in 1980. Runway 4R-22L was constructed as a continuously reinforced slab 14 inches thick with two levels of longitudinal reinforcing steel and is a relatively new pavement in excellent condition. These data would indicate there has been little change in the pavement condition over the duration of the tests (5 years) which is probably a valid conclusion, especially since all DSM results were obtained near the center of a slab.

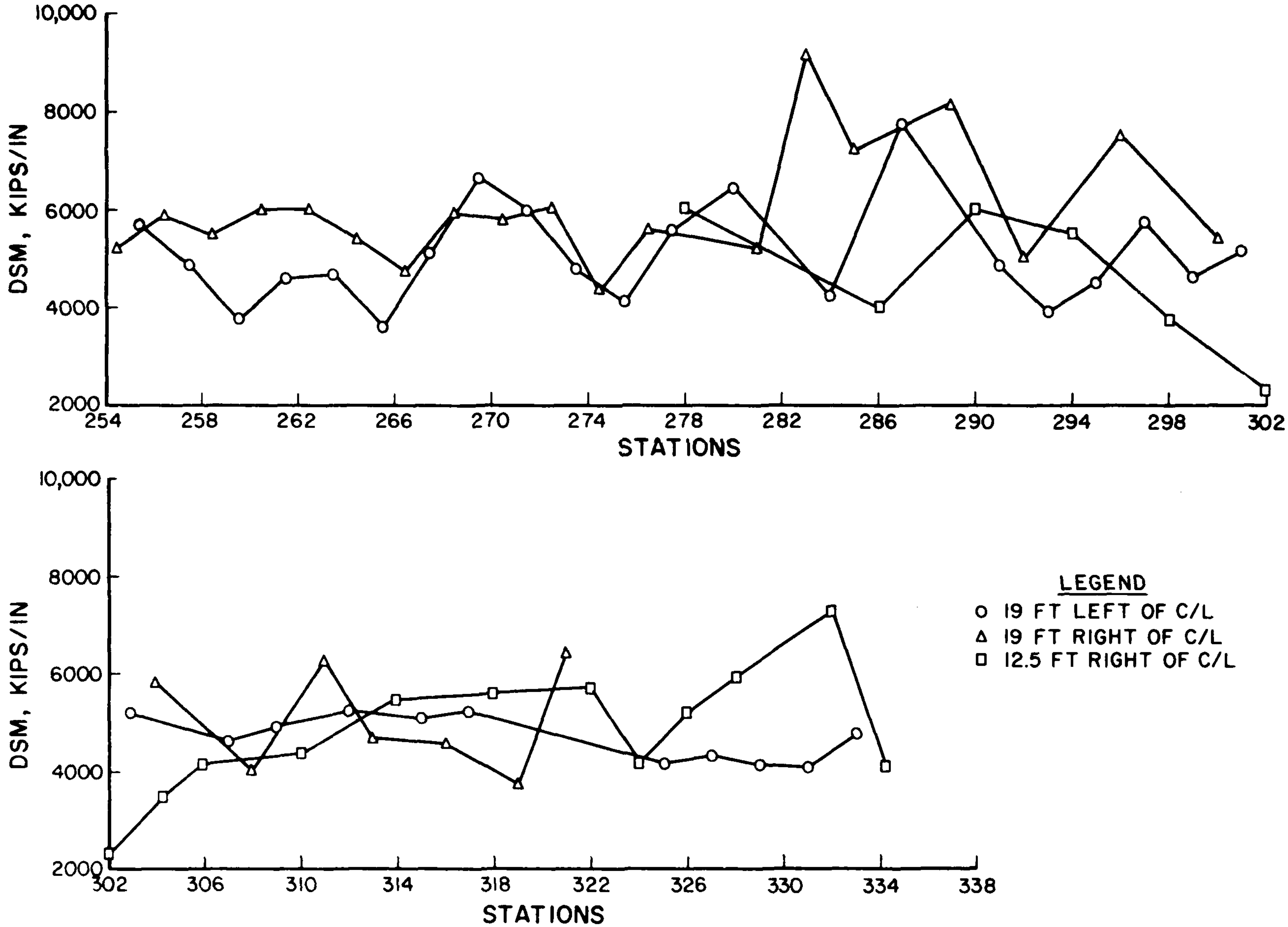


Figure 4. DSM Profiles for Runway 4R-22L

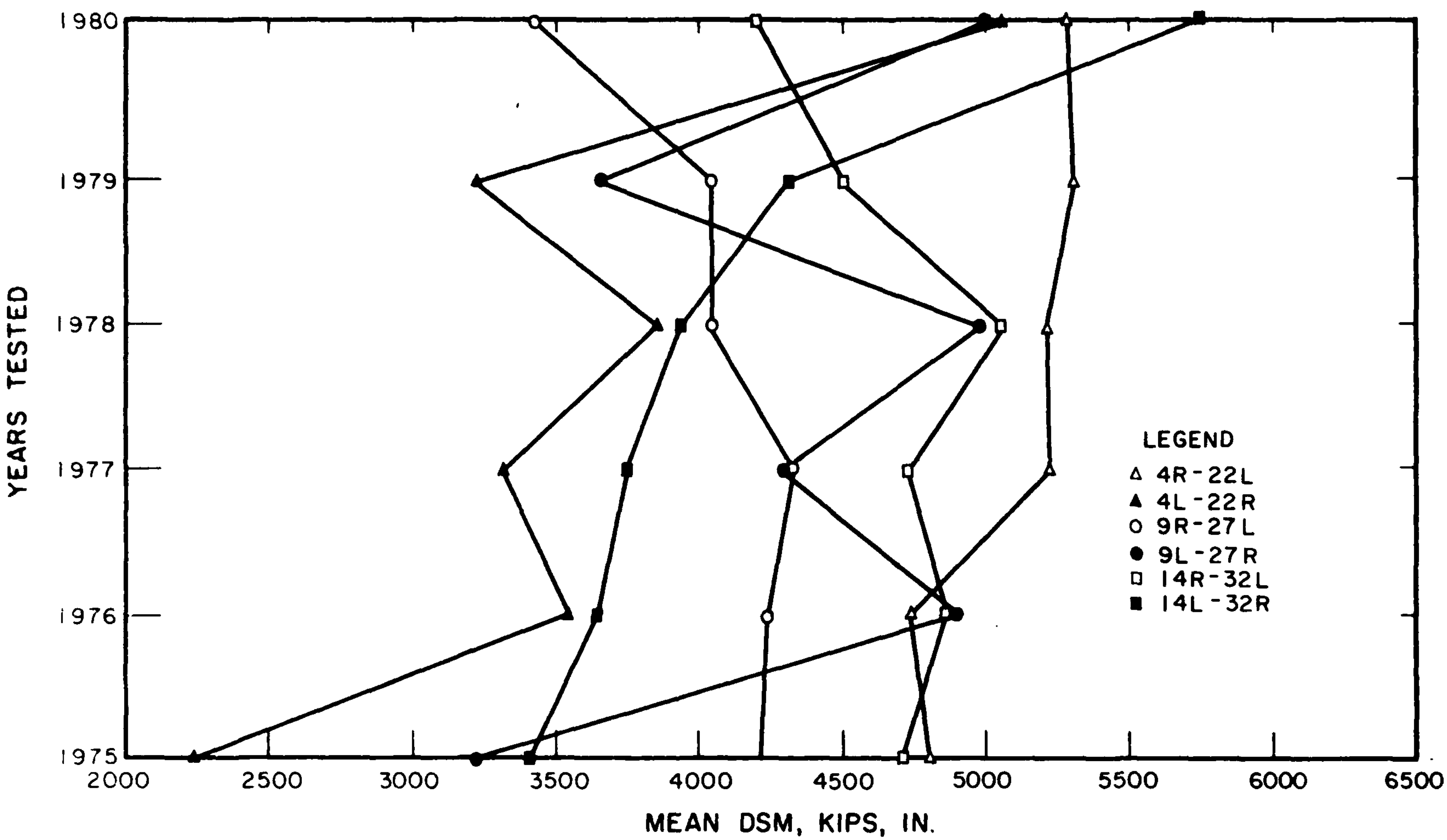


Figure 5. Mean DSM for Each Runway versus Year Tested

In contrast, Runway 9L-27R and 4L-22L show large year-to-year changes in the average DSM. Both pavements are jointed PCC slabs with an asphalt concrete (AC) overlay. Also, the data from both of these pavements as well as the data from Runway 14L-32R show a trend of increasing DSM between 1975 and 1980. This trend would indicate a general increase in the stiffness of the pavement over this period of time, when in actual fact the overall condition of the pavement as determined from visual inspection was worse in 1980 than it was in 1975.

Analysis of the DSM data from NDT conducted annually indicates this approach to evaluation of pavements with PCC slabs, either bare or overlaid with AC, will not provide the necessary information on which one can base any meaningful performance trends. Indeed the data indicate that some slabs showed increasing stiffness with time while the remaining life of the pavements was actually decreasing. Thus new approaches to the evaluation of these pavements were required.

Visual evidence of distress in the PCC slabs at OIA indicates nearly 100 percent of all distress in the PCC slabs is joint related. Distress such as spalling and faulting at the joints are obviously joint related, but even some distress less directly related to the joints can in fact be traced to the performance of the joints.

In areas where PCC slabs show corner or diagonal crack patterns, removal of slabs in these areas revealed the cracks were related to the joints and more specifically to the load transfer across the joints.

As a part of the NDT program conducted by WES the load transfer efficiency (LTE) across the joints was measured using the 16^k vibrator. Figure 6 shows a typical testing and data collection plan for this type

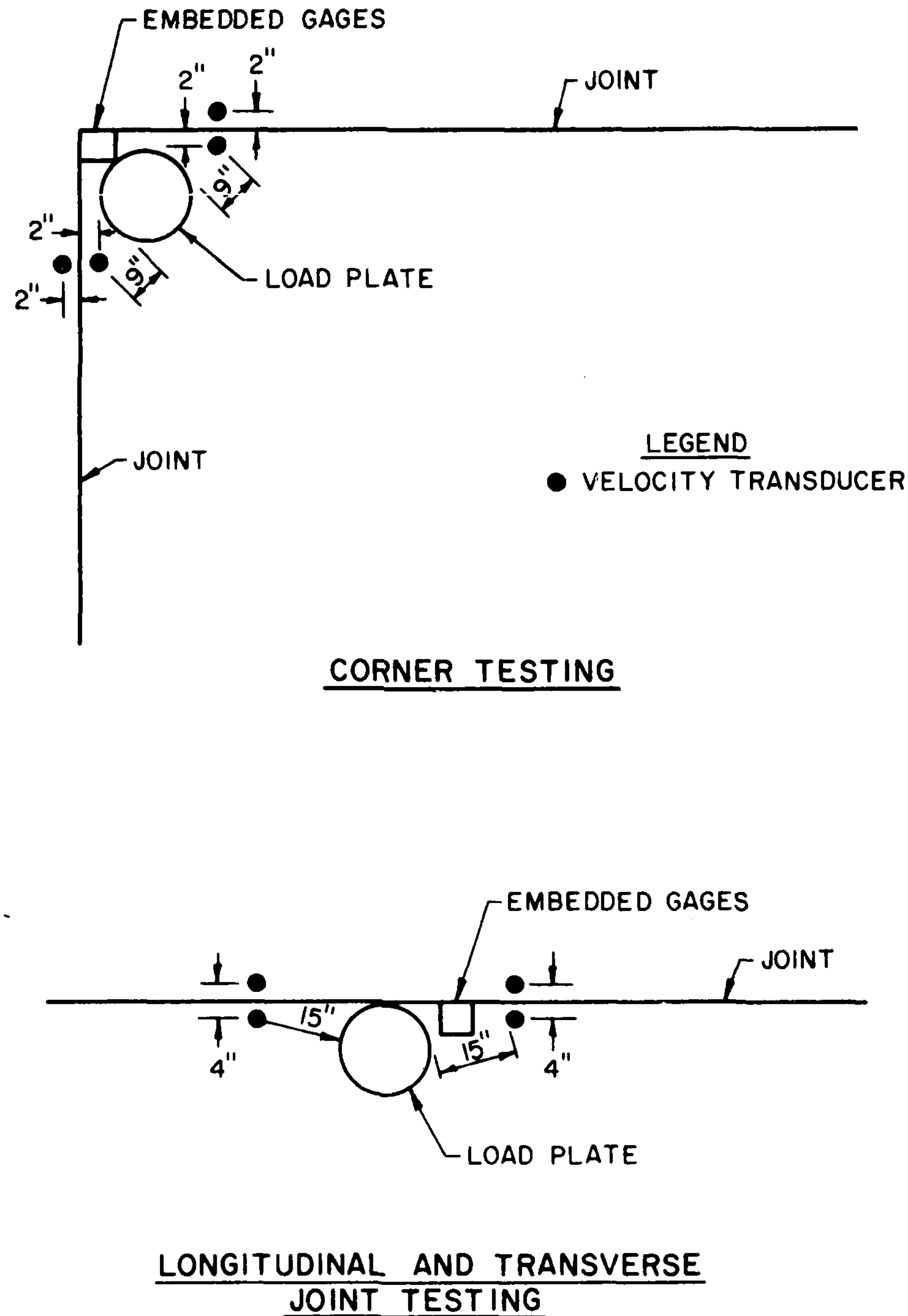


Figure 6. Location of Load Plate and Velocity Transducers during Joint Deflection Tests

of an evaluation. With this arrangement the LTE across a joint is determined by the equation

$$LTE = \frac{\delta u}{\delta L} \times 100$$

where

δu is the deflection of the unloaded slab

δL is the deflection of the loaded slab

Using the plan similar to that shown in Figure 6, the LTE was determined for a number of joints at OIA. Table 1 gives the measured LTE for various facilities. When evaluating the data in Table 1, it must be kept in mind that these are average values. In some instances the actual measured LTE was as low as 10 percent while others approached 100 percent. Also, when measuring LTE with a vibratory type device there is a sympathetic vibration across a joint even if no load transfer is present. From other studies it is estimated that the LTE measured with the WES Vibrator at a frequency of 15 Hz is between 10 and 20 percent greater than the actual LTE measured by aircraft loads or by an impulse type NDT device such as the FWD.

Comparison of the LTE determined using the WES Vibrator, the FWD device, and a B-727 airplane was made at Sites 4 and 5 where the instrument packages were installed. Table 2 gives a summary of the findings from this comparison. In evaluating these data certain facts must be kept in mind. Probably the most important is that the aircraft load is applied through dual tires spaced nearly three feet apart whereas the NDT loadings are applied through plates at a single location. Also, with the NDT equipment it is possible to read the data from both the geophone pickups and the deflection instruments, whereas with the aircraft loading only the data from the deflection instruments are available.

Table 1. Load Transfer Efficiencies Measured Using WES 16^k Vibrator

Test Location, Joint, Time	Type Pavement	No. of Tests	LTE
Inner circle TW (1978)	15" JCP	4	19
Transverse joint			
Inner circle TW (1978)	15" JCP		
Transverse joint		7	37
Corner			
Transverse joint		7	24
Longitudinal joint		7	29
Outer circle TW (1980)	12" AC/15" JCP	12	85
N-S taxiway (1978)	15" JCP		
Transverse joint			

Table 2. Comparison of Load Transfer Efficiencies determined by Using Various Types of Loading at the Instrumented Sites

Location and Joint	LTE (%) WES	FWD	B-727
Site #4			
Transverse joint	95	57	62
Longitudinal joint	81	45	50
Corner			
Transverse joint	38	33	30
Longitudinal joint	22	18	-
Site #5			
Transverse joint	94	56	60
Longitudinal joint	91	73	-
Corner			
Transverse joint	64	48	55
Longitudinal joint	77	-	-

A study of the data in Table 2 shows two factors. First, it appears that the responses of the pavements to loading using the FWD and a typical aircraft (B-727) are in somewhat closer agreement than are the responses from loading with the WES Vibrator and the same aircraft. This confirms findings by others [6] which show that the pavement response to FWD loading more nearly matches that due to vehicles than the response of pavements to steady-state vibratory equipment. A second observation apparent in the data given in both Table 1 and Table 2 is that the LTE is lowest near the corners of the slab and improves as the load is applied nearer the midpanel along the joint. This is clearly shown in data presented in Figure 7 which were collected using the FWD device on Runway 14R-32L.

There is some question as to just what causes the lower load transfer efficiency near the slab corners. If all dowels are in place in the transverse joint and either a sound keyway or some other load transfer mechanism present in the longitudinal joint, then one would expect that the LTE should be as high near the corners as it is away from the corners. There are, however, data which indicate that much of the cause for the low LTE at the corners is due to loss in efficiency of the load transfer mechanisms under repeated traffic.

Evidence of the breakdown in LTE near the corners of jointed PCC slabs and the significance of this breakdown is seen in the data from FWD testing at the south end of Runway 14R-32L. Table 3 shows a summary of the deflection and LTE across transverse joints measured both near the corner and at a point nearly midway between the longitudinal joints. The key for the location of tests is given in Figure 8. The data show that on the average the LTE across the transverse joint near the corner is about one-half the LTE across the

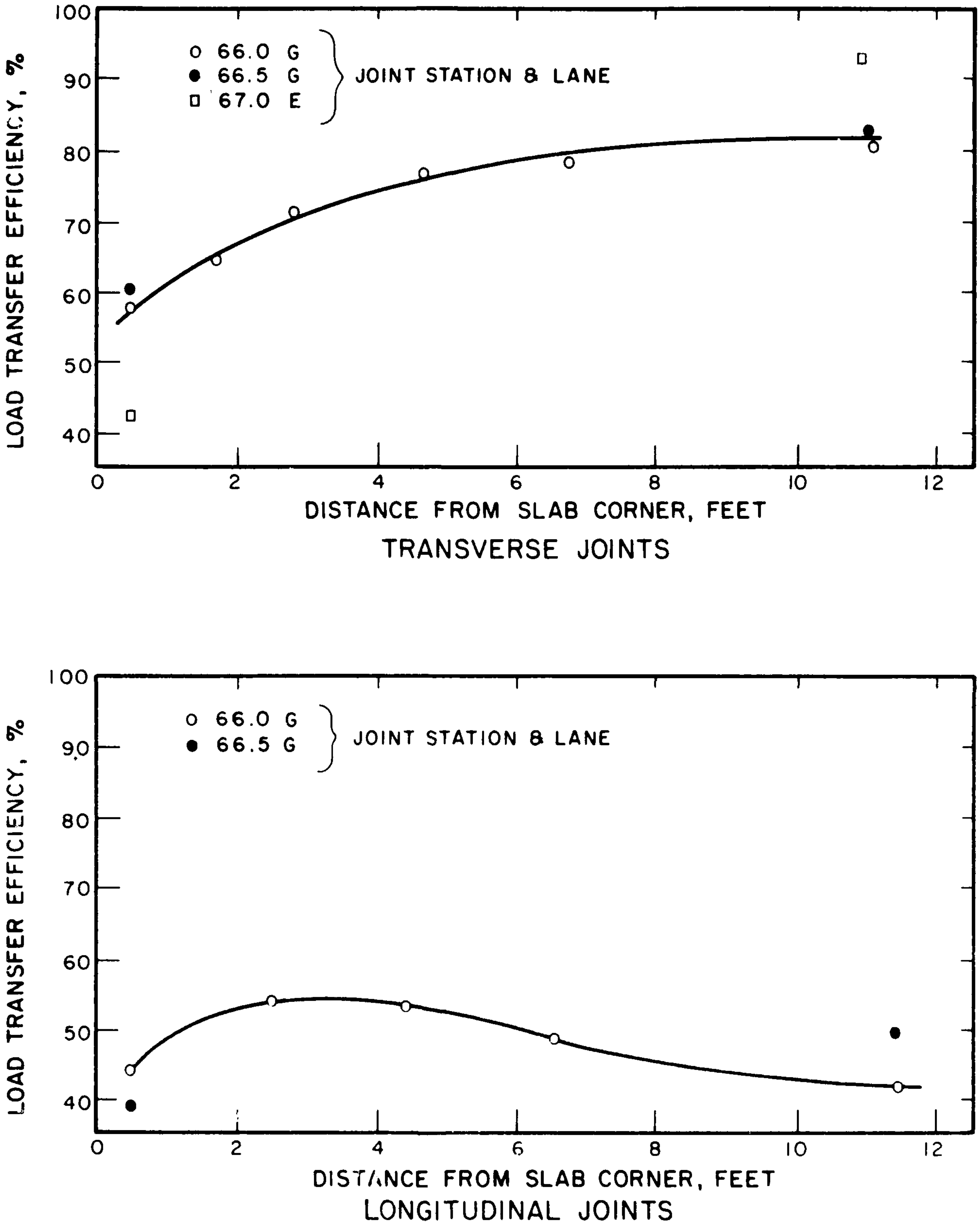


Figure 7. Load Transfer as a Function of Distance from the Corner

Table 3. Summary of Results from FWD Tests on JCP on Runway 14R-32L OIA

Location Sta., Lane		Corner		Transverse Joint	
		Deflection (Mils)	LTE across Transverse Joint (&)	Deflection (Mils)	LTE across Joint (%)
64.5	D	29.8	40	8.0	81
65	A	29.5	35	7.6	76
65	B	29.5	37	6.8	90
65	E	24.9	48	6.5	100
65.5	A	11.7	52	7.1	75
65.5	C	30.2	30	8.1	90
65.5	D	35.7	30	9.7	81
66	C	31.7	24		
66	E	30.7	48	6.9	96
66.5	B	16.3	76	7.0	90
66.5	C	26.7	47	7.4	95
66.5	D	39.4	26	14.1	69
67	B	8.0	90	6.8	96
67	E	27.4	43	6.9	93
67.5	D	26.3	40	6.7	90
68	E	28.0	42	5.9	93

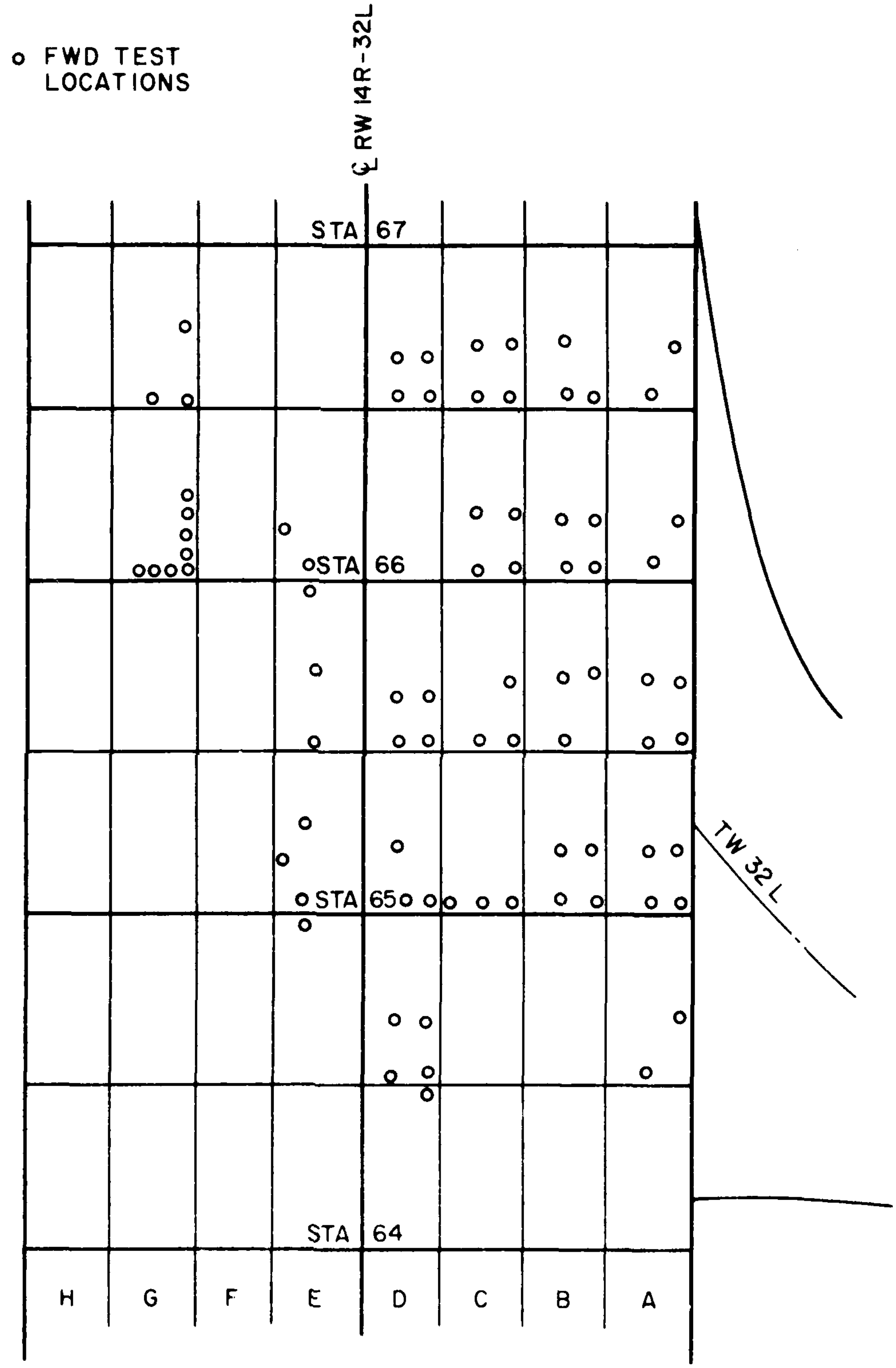


Figure 8. Locations of FWD Tests on RW 14R-32L

same joint at a point near midslab. Furthermore, it is seen that the average deflection near the corner is nearly 4 times as great at the deflection near midslab. Assuming a uniform LTE along the joint, the theoretical maximum ratio for these two deflections is 2.63. These data clearly show that there is a breakdown of the load transfer across these joints near the corners of the slab. The cause and effect of this breakdown is seen in the analysis of failures of pavements in the outer circle taxiway.

Several slabs which had cracked and faulted under traffic were partially removed as illustrated in Figure 9. Upon removal of portions of the slabs as shown, several causes and effects were immediately apparent. First, it was noted that the tie bars in the keyed longitudinal joint had rusted through and failed. Also, that most of the male portion of the keyway had failed or been removed. As a consequence the longitudinal joints had retained little or no load transfer ability. Second, it was observed that the three dowels in the transverse joint nearest the longitudinal joint had bent, with the dowel nearest the joint bent most severely, and the third dowel bent least of the three. Also, the dowels nearest the longitudinal joint had a significant "looseness" which permitted a vertical movement of these dowels in their socket of up to 0.15 inch. This looseness decreased nearly linearly to zero movement by the sixth dowel away from the longitudinal joint.

In an earlier report to FAA [7], it was shown that keyways tend to fail under the heavy gear loads of the modern aircraft. Certainly the experience at OIA would support this conclusion. It was also shown in this same report that dowels are very efficient load transfer devices but that the dowels tend to loosen in their sockets under repeated loads of heavy gear loads.

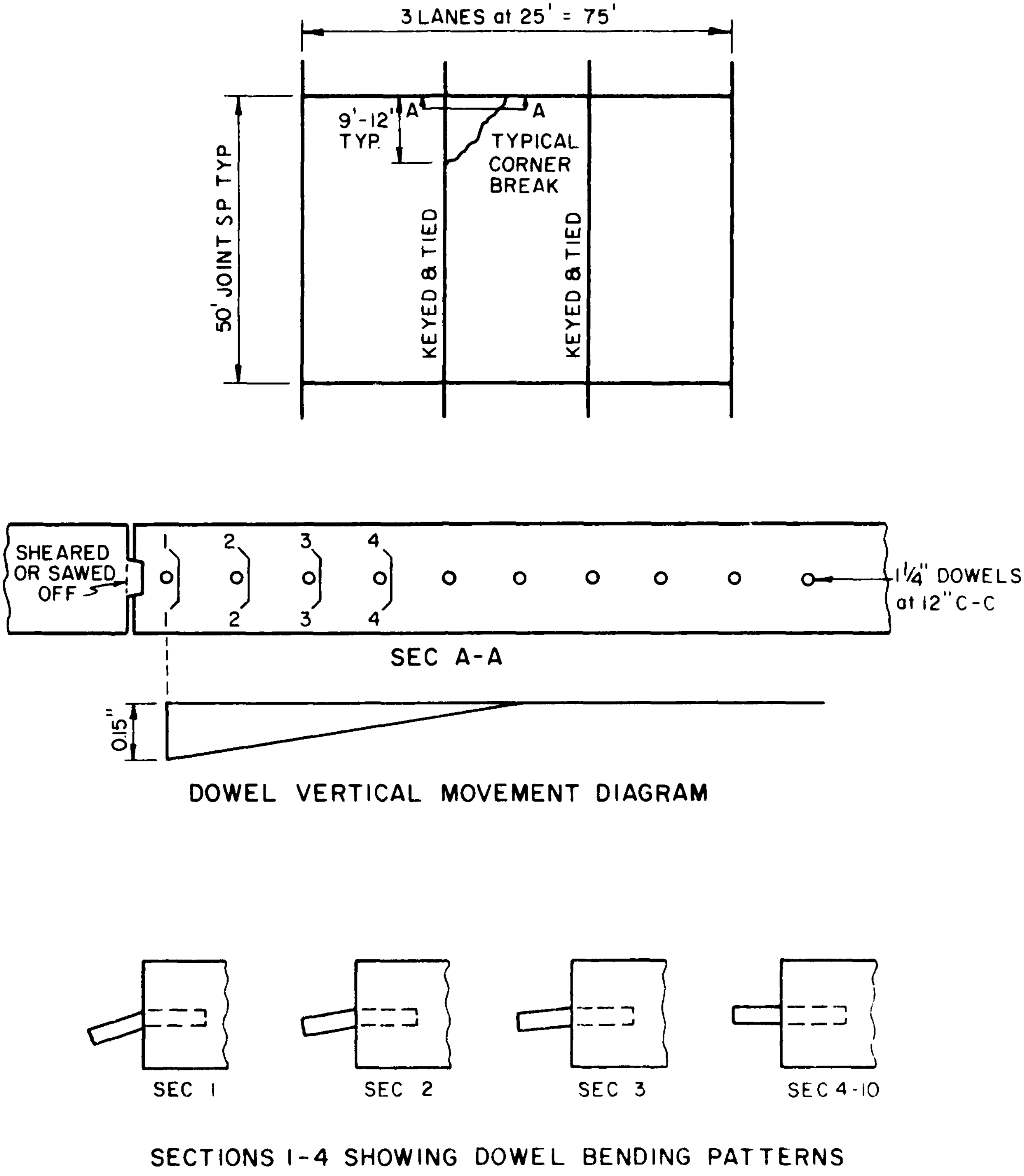


Figure 9. Failure Mechanism at Corner Breaks
in Outer Taxiway, OIA

As the dowel sockets in the concrete elongate and the dowel loosens, there is a decrease in support for the dowel, especially adjacent to the joint. When the loss of support progresses far enough back from the joint, the dowel must span a greater distance and bends under the load.

The effectiveness of new dowel systems can be seen in results from the crossover taxiway between the inner and outer circle taxiways constructed in 1978. A 21 inch thick PCC pavement was constructed using 2 inch diameter dowels and heavy ties (1 3/8) near the intersection of the longitudinal joints (at the corners), while the more normal 1 1/4 inch diameter dowels were used away from the corners. Load transfer measured in late October 1980 using the WES vibrator yielded LTE values of 100 percent across the transverse joints and approximately 90 percent across the longitudinal joints, both excellent values. Similarly it is seen in Table 2 that the LTE near the corners is somewhat better at Site 5 than at Site 4, which is probably due to the relatively low traffic volumes at Site 5 as compared with Site 4.

Presentation of Findings - Instrumented Sites

Instrumented pavements were loaded at different times with several types of load, including three different pieces of NDT equipment and a range of aircraft gear types and loads. The pieces of NDT equipment used were the U.S. Army Engineers WES 16 kip vibrator, two identical Road Raters, Model 2008, capable of maximum dynamic loads up to 8 kip, and a dynaflect with a maximum dynamic load of 1 kip. Aircraft loads consisted of B-727 and DC-10 aircraft under less than maximum gross load over the pavements near the instrumented sites. Results from the 26 strain and deflection gages were recorded simultaneously on magnetic tapes for all loading conditions.

The pavement at Test Site 4 consisted of a 15 inch thick PCC slab on 12 inches of granular subbase. The slabs were reinforced between joints with dowelled transverse joints at 50 foot intervals and tied longitudinal joints at 25 foot intervals. Dowels in the transverse joints were 1 1/4 inches in diameter, placed at 12 inch centers. Exact size and spacing of the ties for the longitudinal joints are not known.

Figure 10 shows the load patterns for several runs with the B-727 aircraft on the pavements at Site 4 and the locations of the deflection gages and the critical strain gages. With these loading patterns, the main gear of the aircraft passed over or near the deflection gages at locations 1, 2, 3 and 4, shown in Fig. 10.

Measured and calculated deflections at critical locations are shown in Table 4. Calculations were made using both the finite-element program, ILLI-SLAB [7], and with the influence charts developed by Pickett, et al. [8]. One of the advantages of the ILLI-SLAB program is that with this program it is possible to analyze slabs with joints having varying efficiencies of load transfer across these joints. Thus, the calculated deflections for loads applied near the joints and at the corners are given for slabs having varying load transfer efficiencies across both the longitudinal and transverse joints as appropriate. All calculations shown with both the ILLI-SLAB program and the influence charts for Site 4 were made with an assumed "k" value of 200 pci/in. on top of the granular subbase.

Figures 11, 12 and 13 show the results of measured deflection at the locations for Site 4 using the WES NDT vibrator and the B-727 as the load. For the locations shown, the deflections are shown from both the NDT sensors (geophones) normally used to measure surface deflections with the vibrator, and the deflection gages installed in the pavement. The results from the

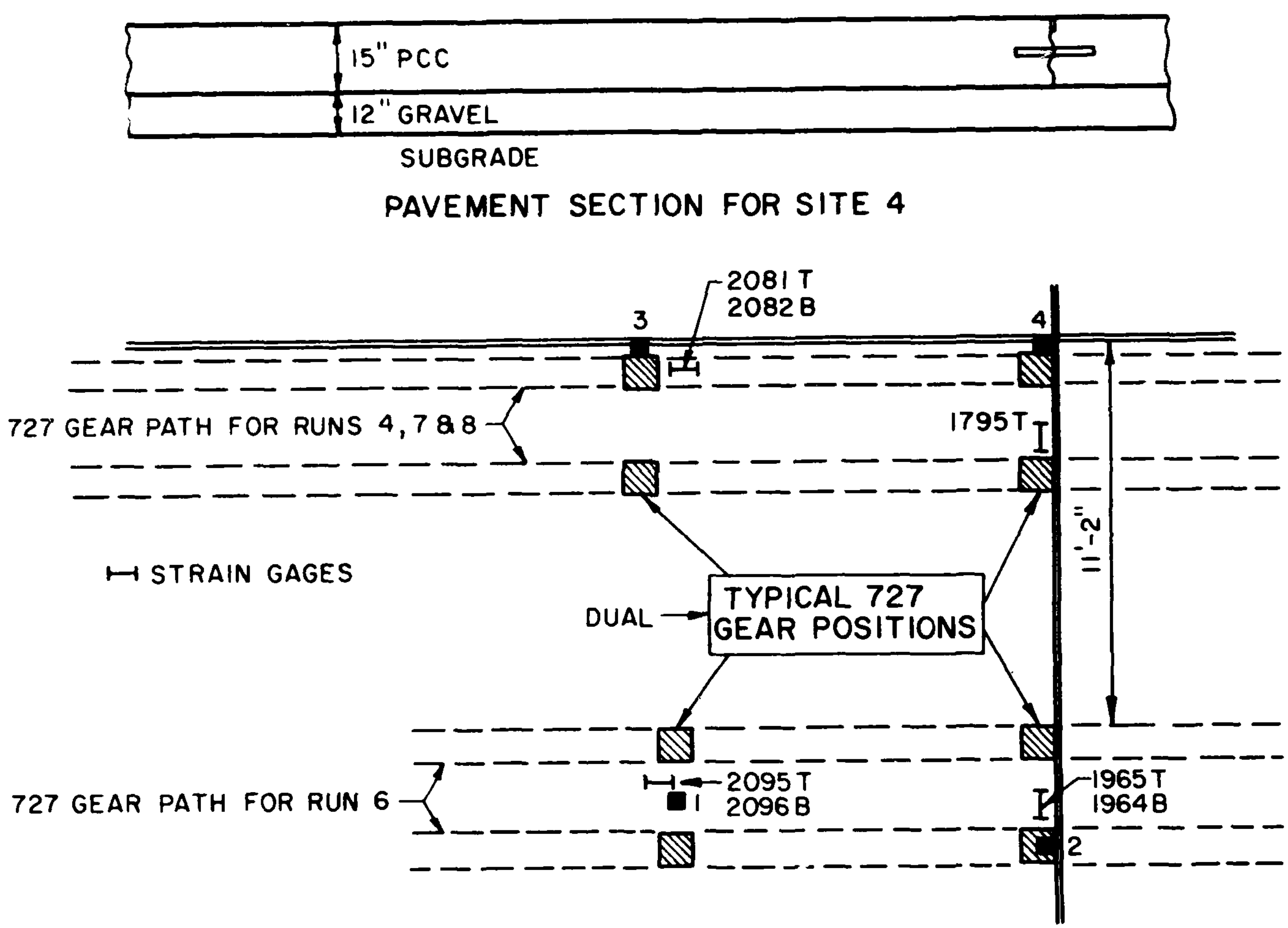


Figure 10. Pavement Section, Gear Location, and Typical Gear Paths for Aircraft Loading at Site #4

Table 4. Comparison of Measured and Calculated Deflection for Site 4

Gage Location (Fig. 10)	Measured Deflection (inches)	Calculated Deflection, inches, for Indicated Load Transfer Efficiency				
		0[a]	0[b]	50%[b,c]	67%[b]	90%[b]
Center of transverse joint	.017-.025	.0313	.0332	.0219	.0180	.0172
Center of longitudinal joint	.016-.021	.0258	.0264	.0177	.0147	.0141
Corner of slab	.065-.190	--	.073	.0374	--	--

a Calculated using influence charts developed by Pickett, et al [3] k = 200

b Calculated using ILLI-SLAB [1] k = 200

c Approximate efficiencies as the actual efficiency changes slightly with subgrade modulus, pavement thickness, and load locations

d 1 inch = 25.4 mm

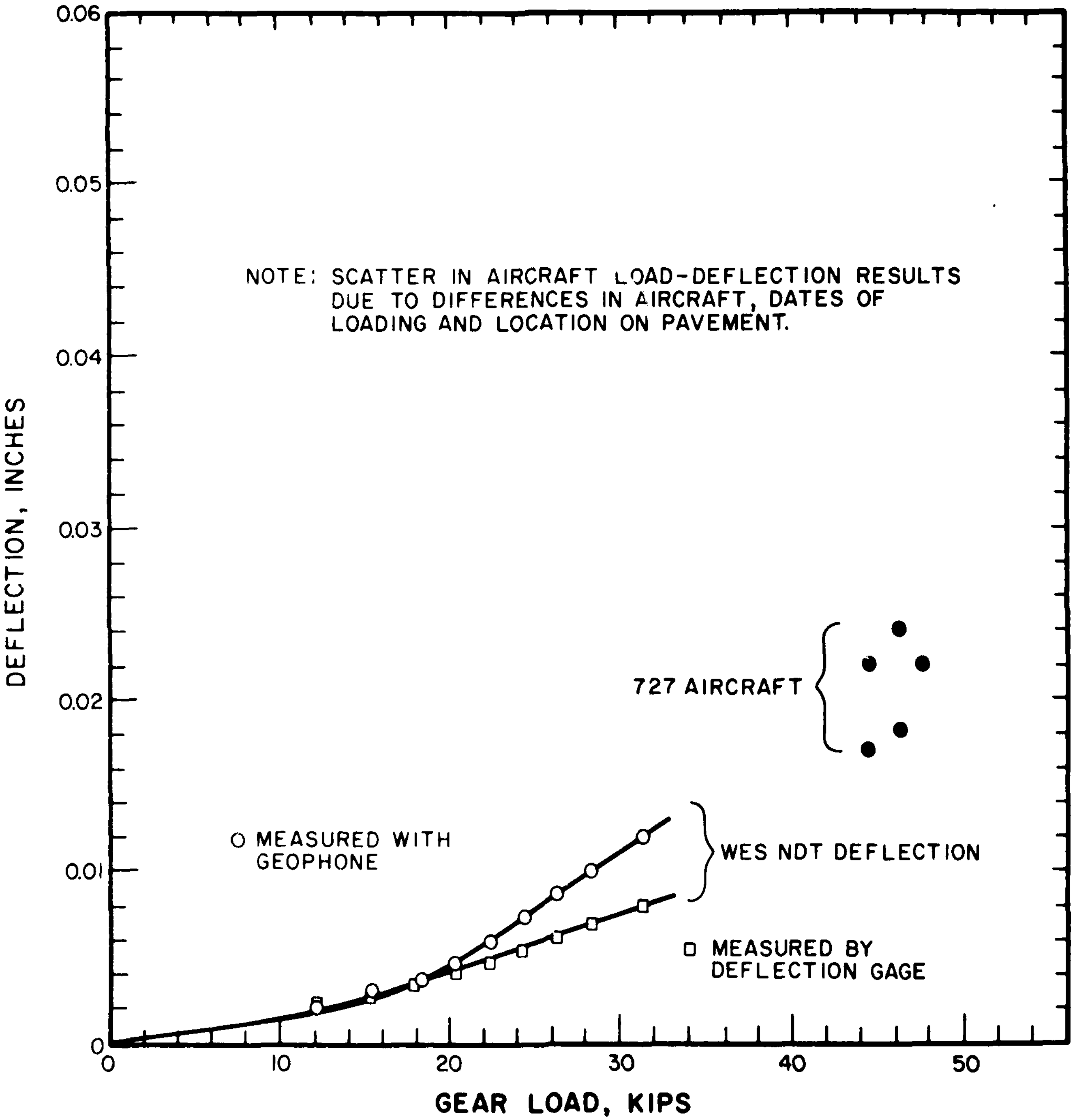


Figure 11. Typical Deflection Results Collected at Site #4; at Dowelled Transverse Joint

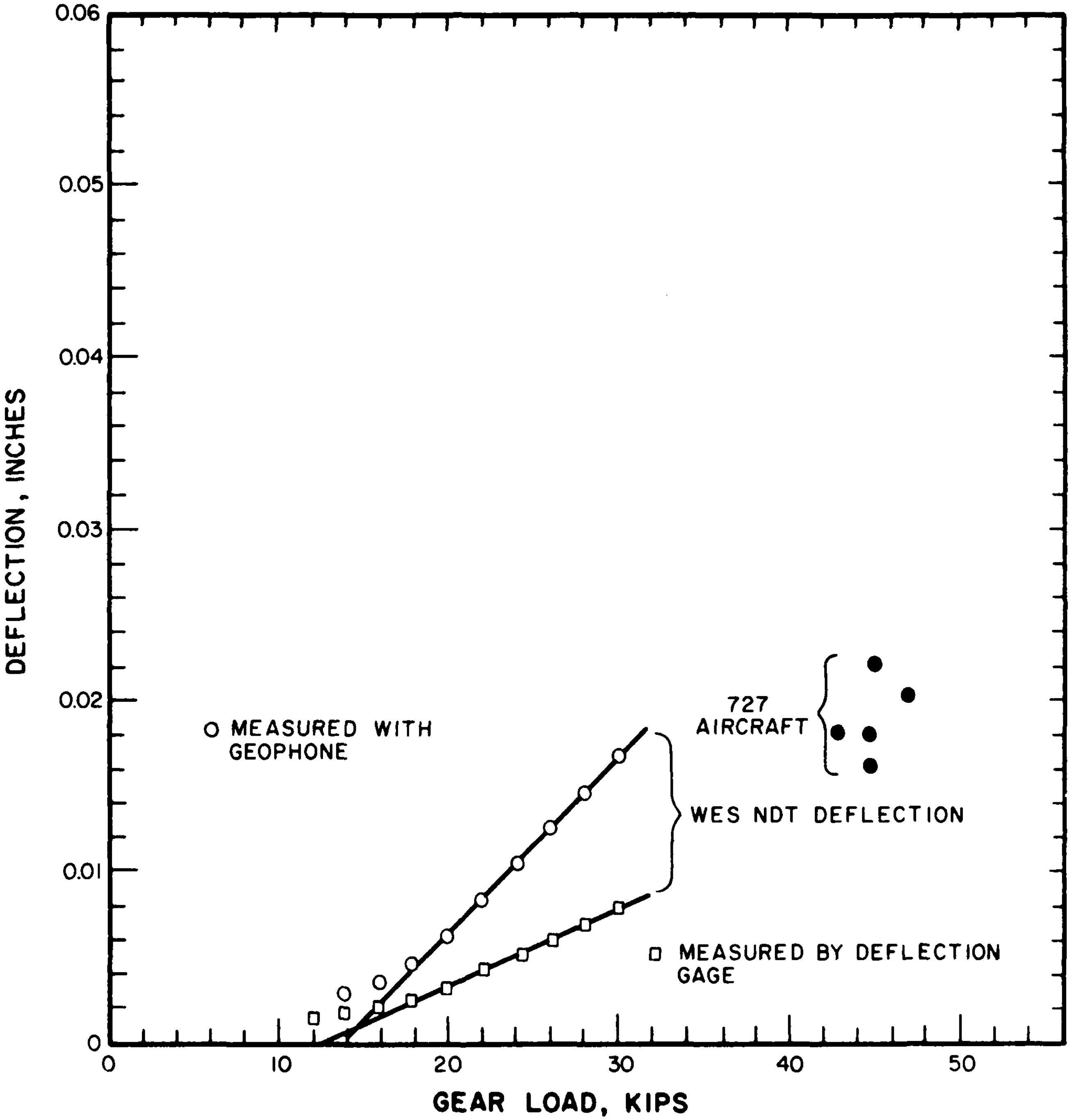


Figure 12. Typical Deflection Results Collected from Site #4, at Center of Longitudinal Keyed Joint

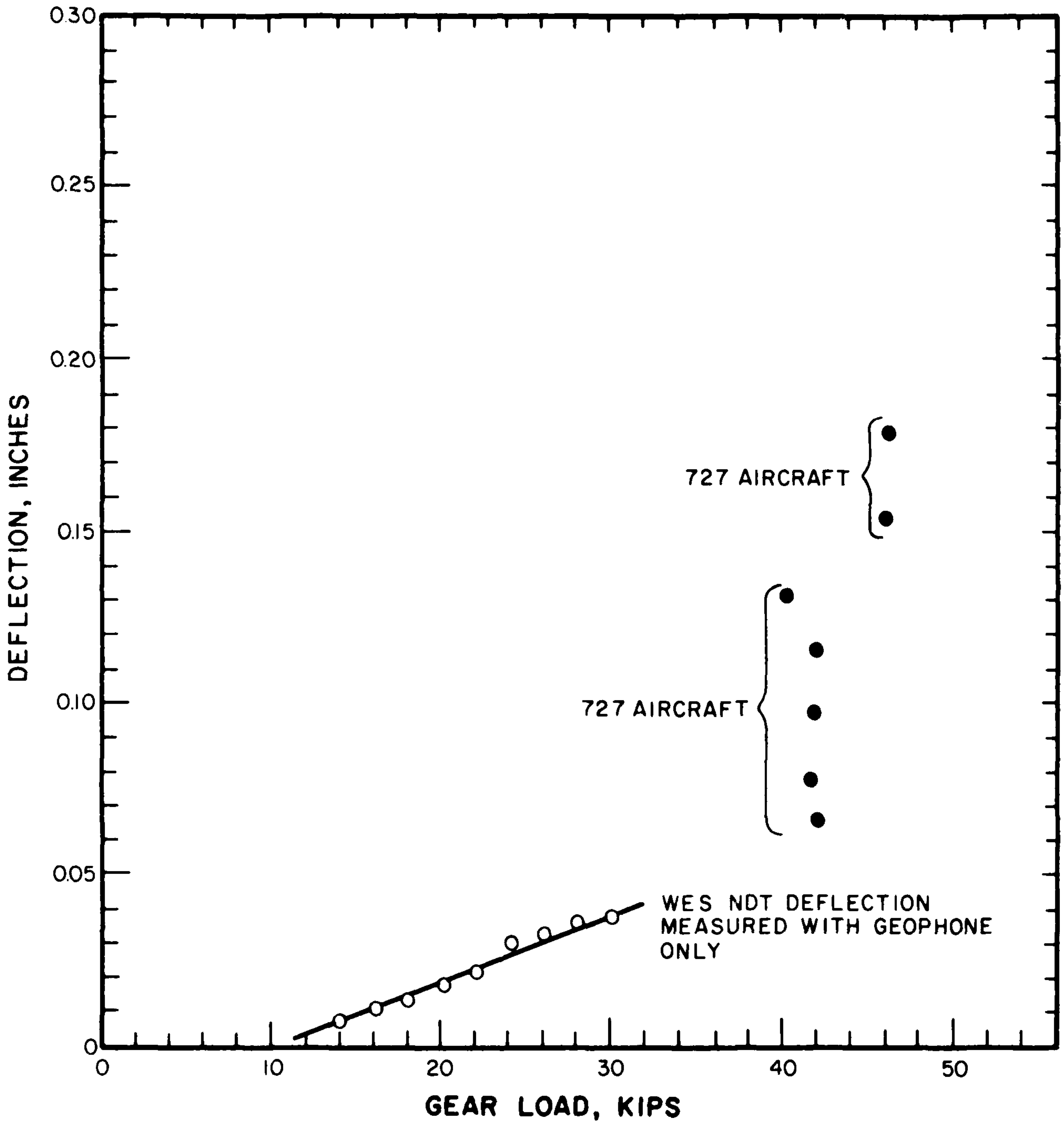


Figure 13. Typical Deflection Results Collected from Site #4, near a Corner of the Slab

gage installed at the center of the slab (location 1) are not shown because this deflection gage was not operative at the time of the test.

Measured strains in the pavements at Site 4 under both aircraft and vibratory loads are shown in Figures 14, 15 and 16. Results shown are the actual strains as measured without correction for load location with respect to the gages and for the location of the gages in the pavement system. It should be noted, for example, that the Carlson wire strain gages are located between two and one-half and three inches from the top and bottom faces of the slabs. To obtain maximum strains in the slab the measured strains will have to be adjusted to convert the measured strains to the theoretical maximum strains. This adjustment is discussed later in the section on the interpretation of the results.

The locations of the gear paths for the DC-10 aircraft at Site 5 are shown in Figure 17 along with the location of the principal gages at the site. Location of the wheel paths for the B-727 aircraft at Site 5 was similar to that for Site 4 as shown in Figure 10.

Figures 18 and 19 show the measured deflections at two locations for Site 5 under a range of vibratory loads and for the B-727 and DC-10 aircraft. Unfortunately, deflection gages at two other locations were not operating properly at the time these tests were run. Thus the NDT geophone results at these locations are also not shown as the results do not show any trends significantly different from those shown in Figures 18 and 19.

Measured strains from two locations at Site 5 under vibratory NDT and aircraft loads are shown in Figures 20 and 21. Again, these findsing must be adjusted for the location of the loads and the location of the gages in the pavements.

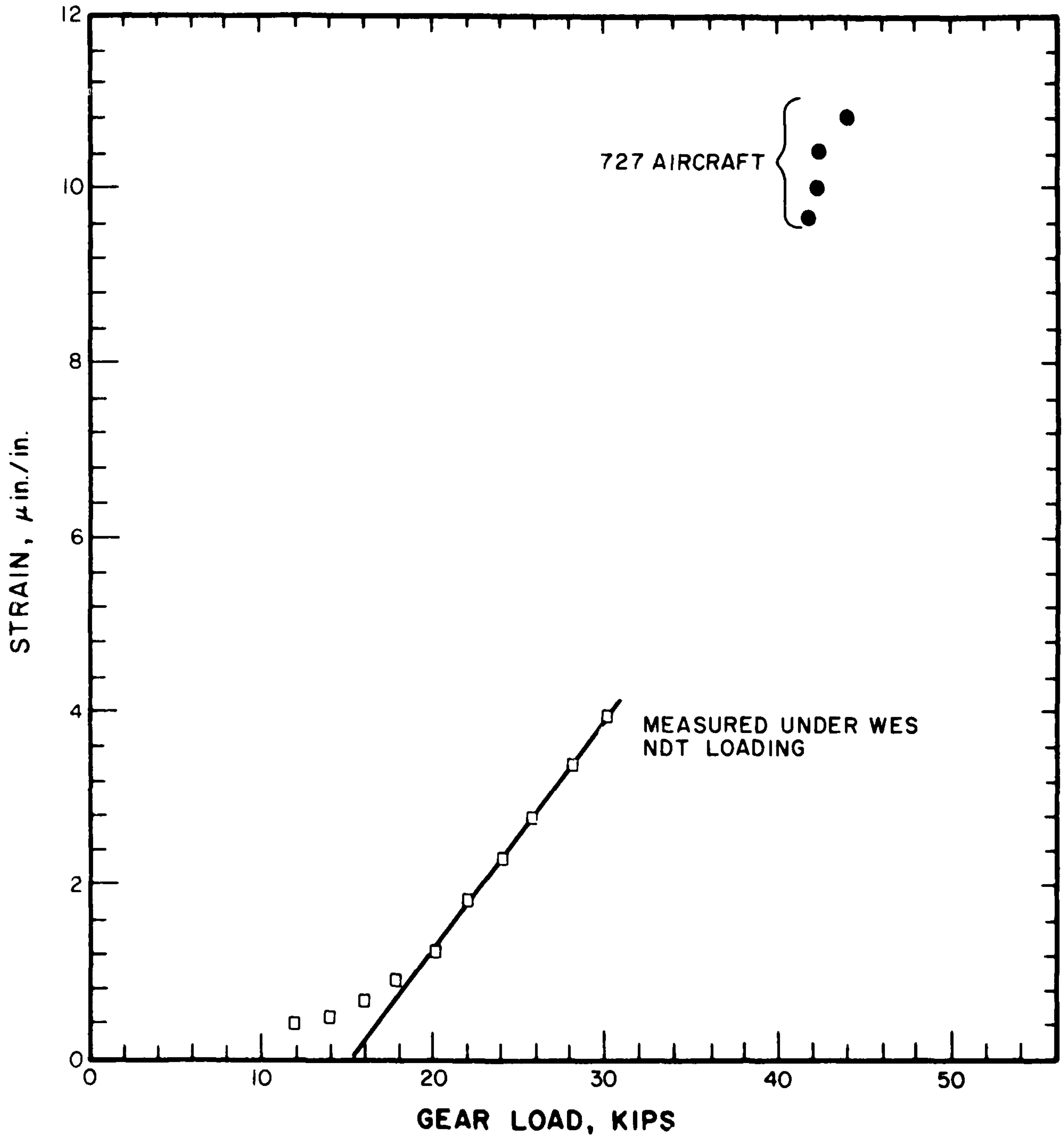


Figure 14. Typical Strain Results from Site #4, at Dowelled Transverse Joint

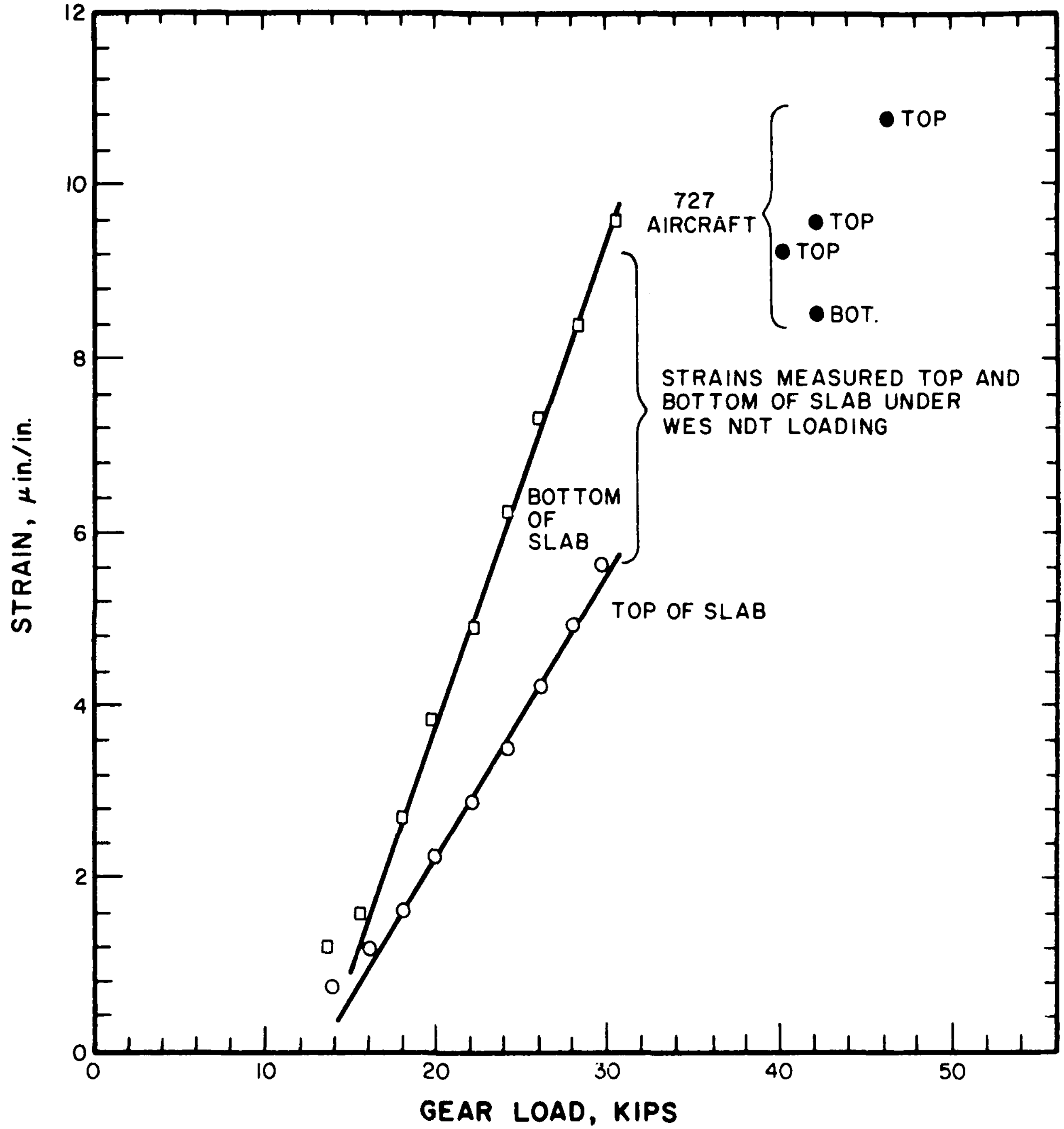


Figure 15. Typical Strain Results from Site #4,
at Center of Keyed Longitudinal Joint

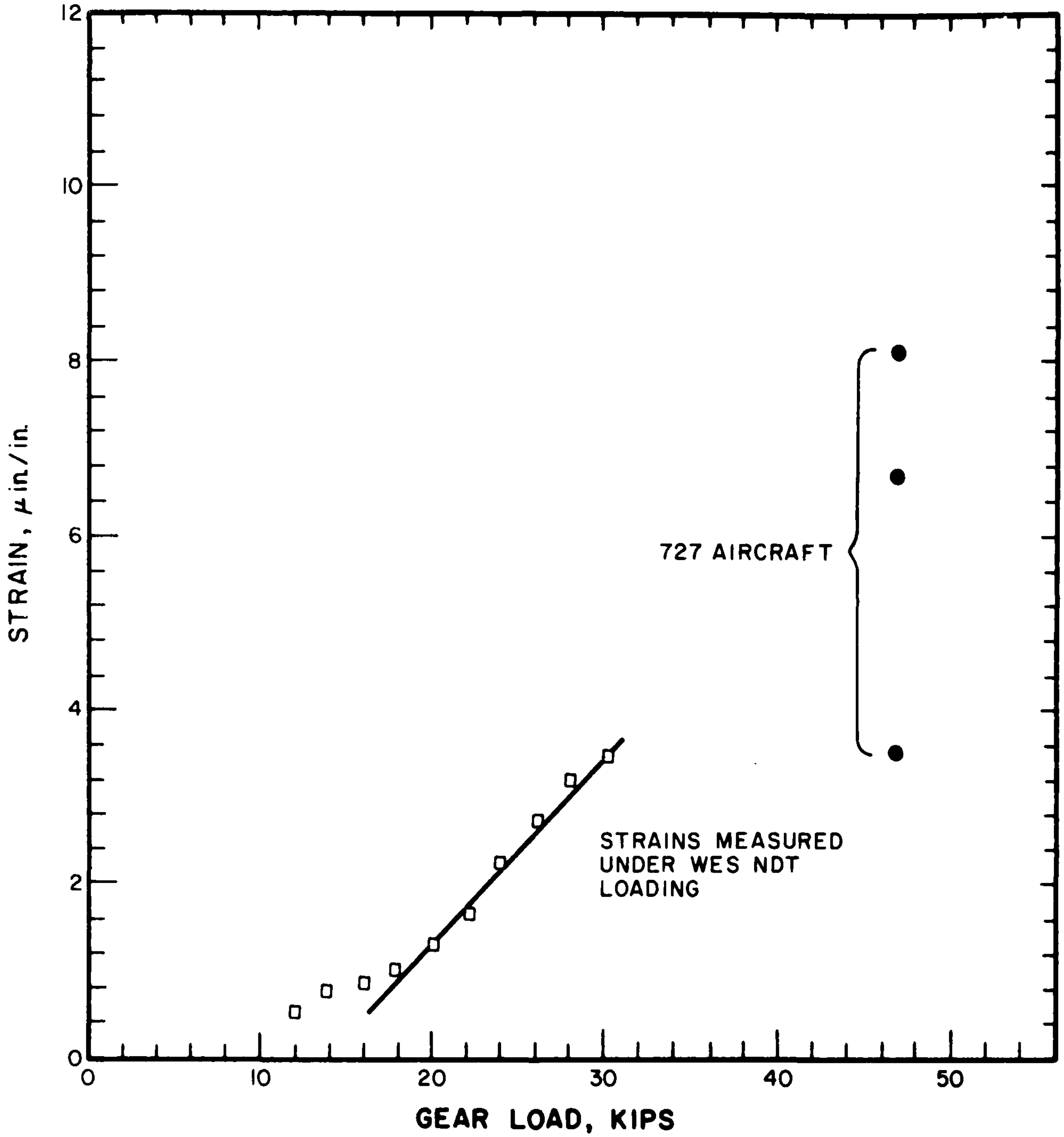


Figure 16. Typical Strain Results from Site #4, Load Placed near the Slab Corner

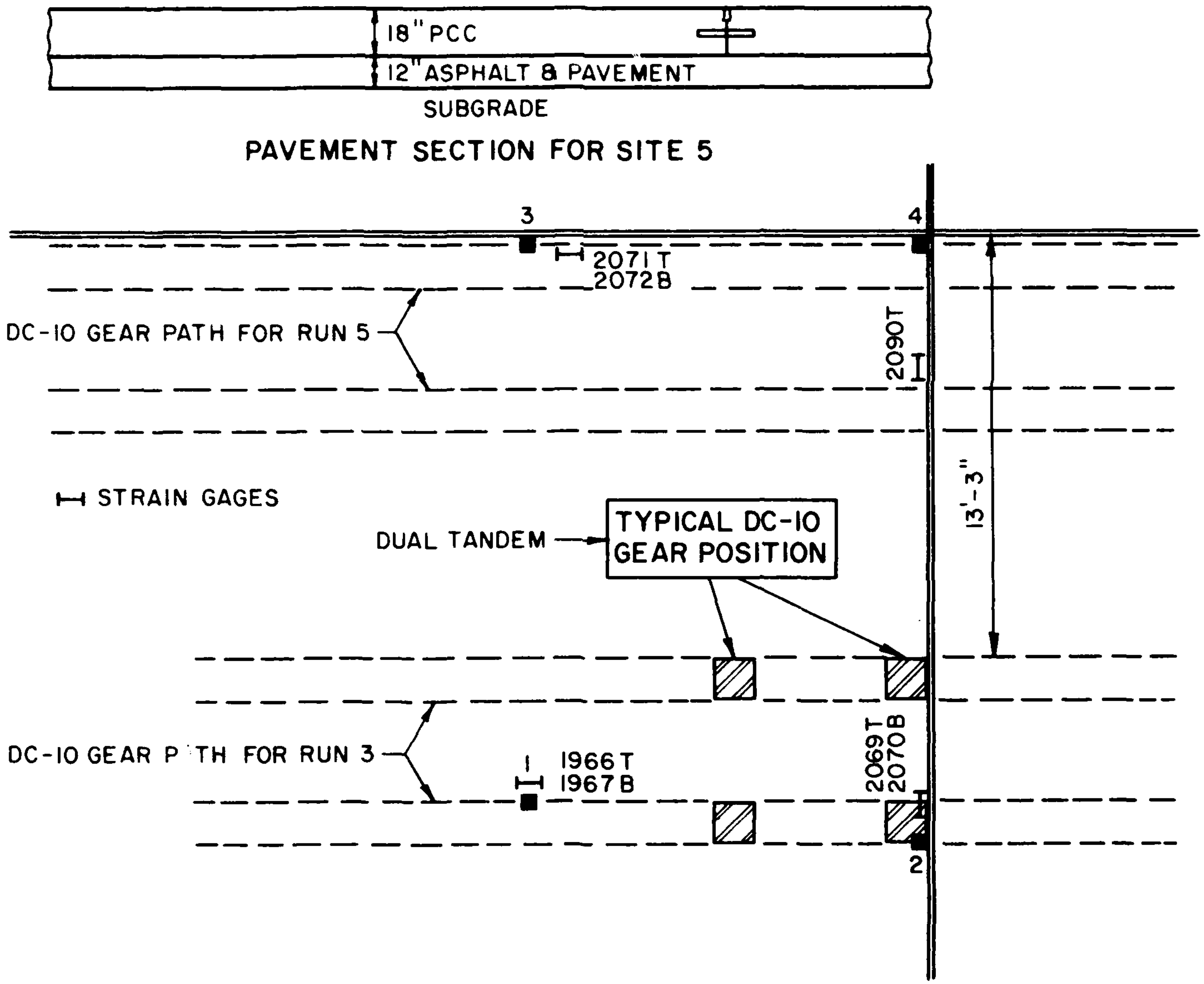


Figure 17. Pavement Section, Gage Locations, and Typical Gear Path for Aircraft Loading at Site #5

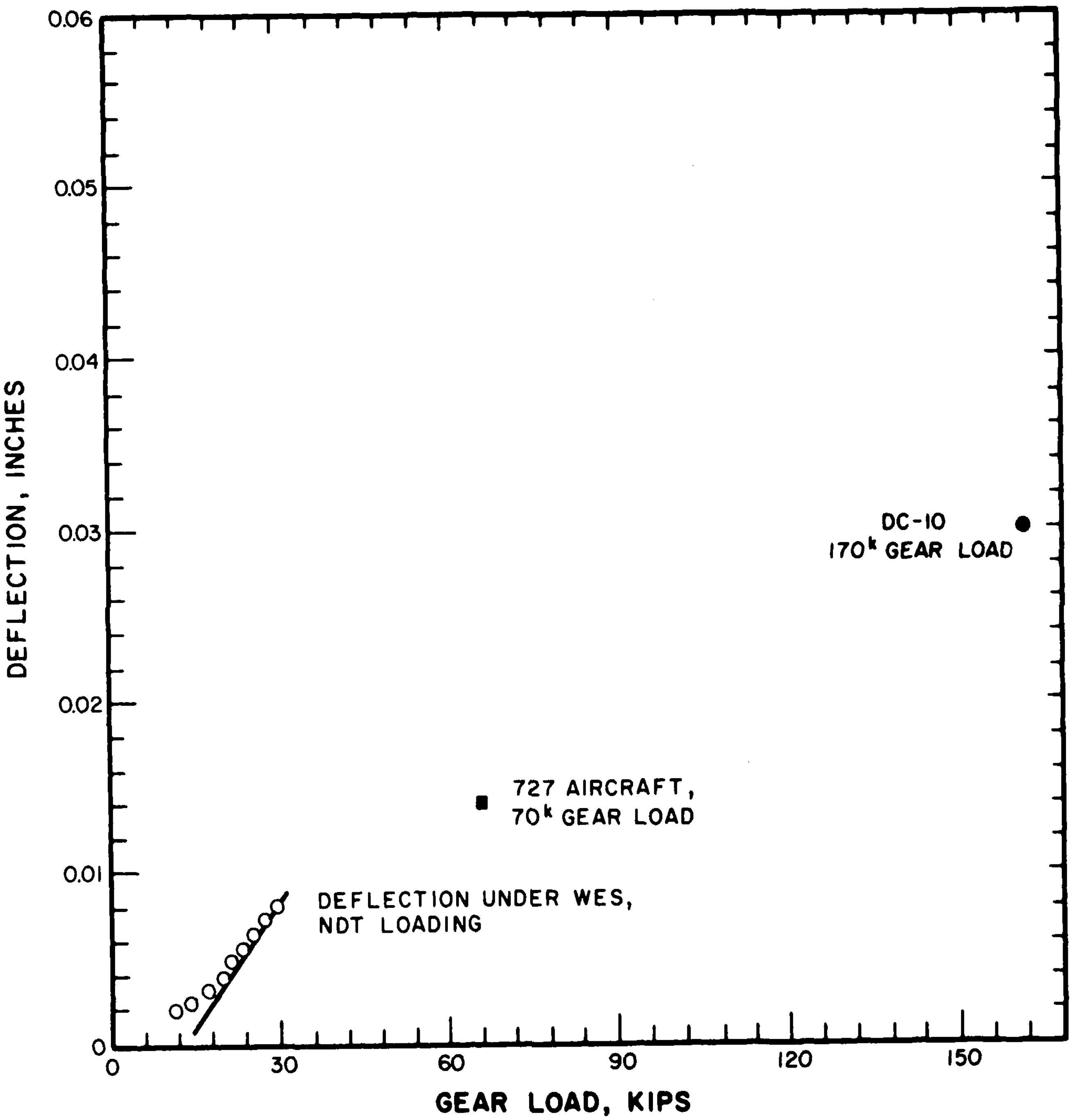


Figure 18. Typical Deflection Results Collected from Site #5, Load at Center of Dowelled Transverse Joint

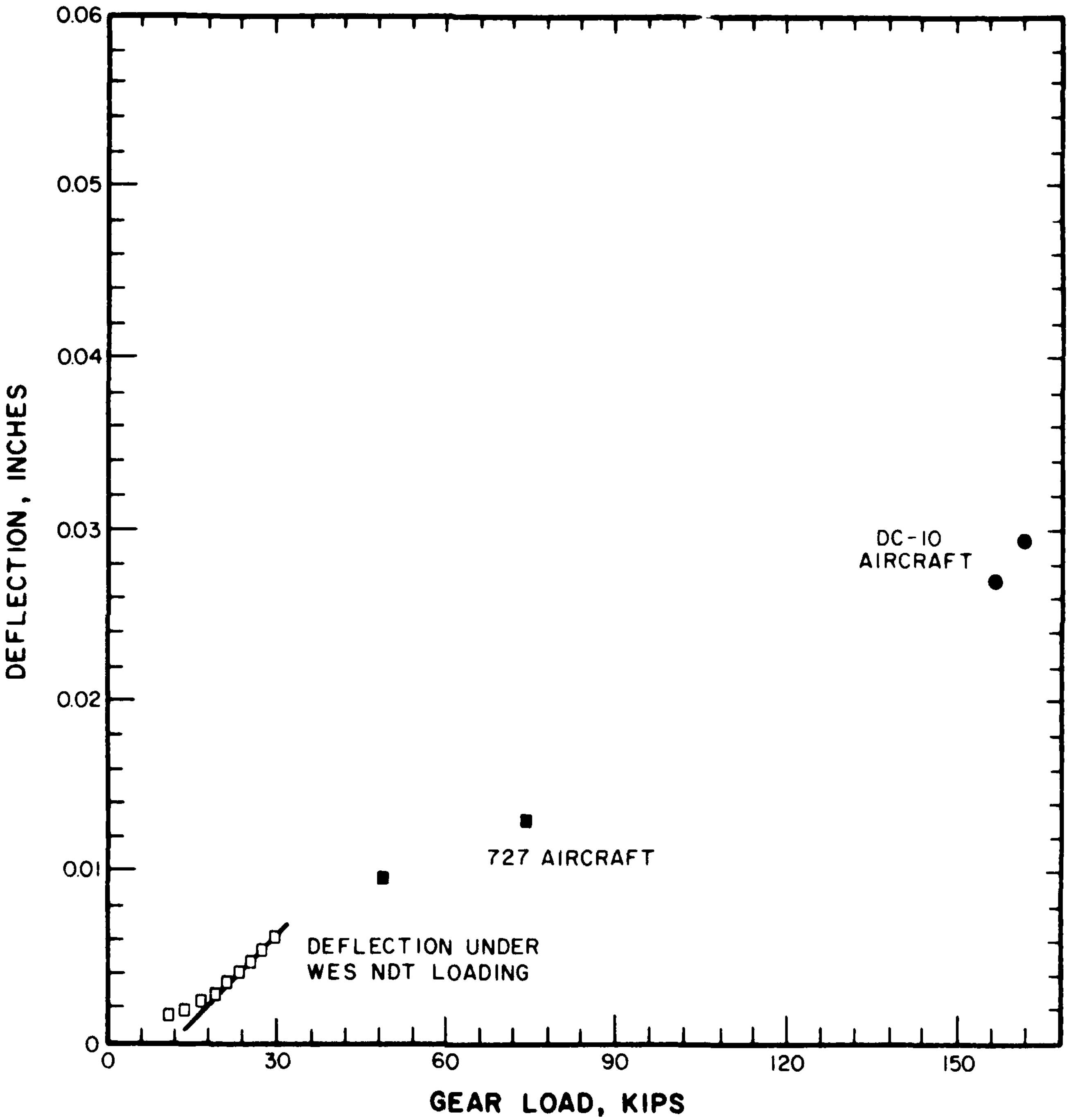


Figure 19. Typical Deflection Results Collected from Site #5, with Load at Center of Slab

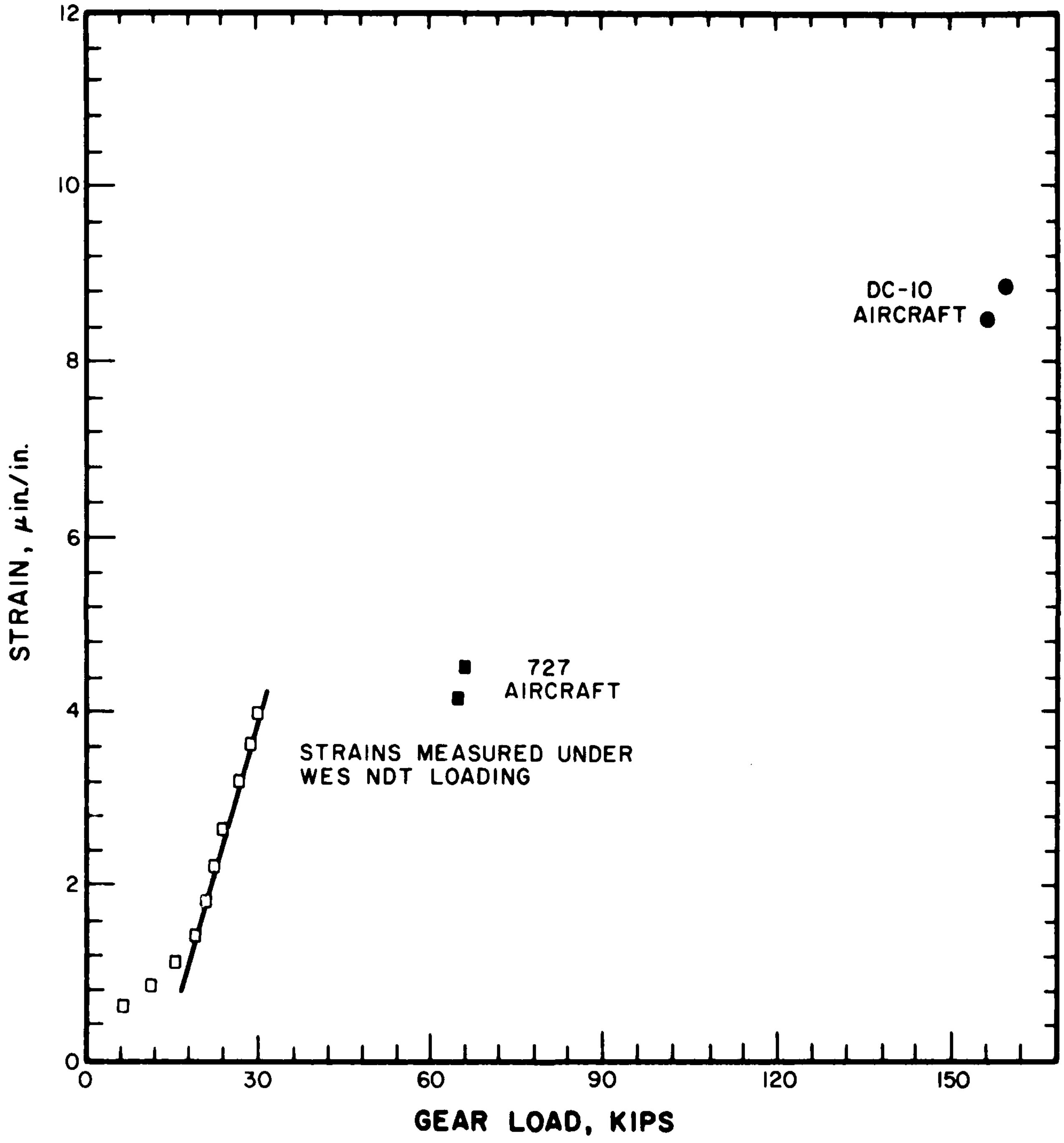


Figure 20. Typical Strain Results from Site #5, with Load at Center of Slab

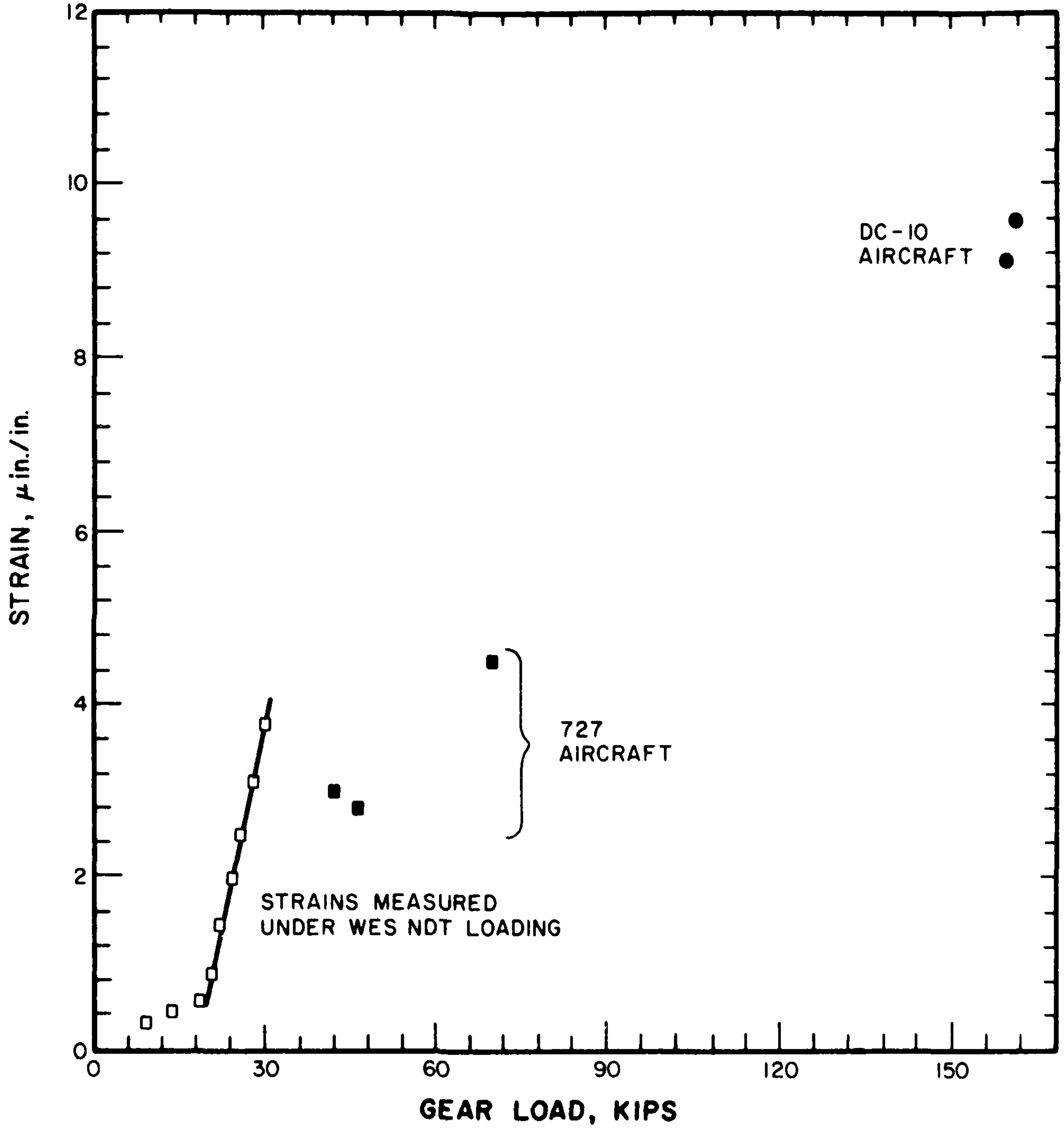


Figure 21. Typical Strain Results from Site #5, with Load at Center of Dowelled Transverse Joint

Measured deflections at Site 3 under loading with NDT vibratory devices are shown in Figure 22. Attempts to measure the deflections at Site 3 with aircraft under normal operations proved unsuccessful as the location of the actual wheel paths could not be determined with sufficient accuracy to make comparisons.

Interpretation and Discussion of Findings

The primary purposes of tests at O'Hare were to validate the analysis procedures, and especially to validate the ILLI-SLAB program used for analysis of jointed concrete pavements with varying load transfer efficiencies, and to evaluate the effectiveness and efficiency of various load transfer systems. This is a dual pronged validation as the measured results must be validated as well as the calculated values. Thus the major thrusts of this discussion will be the relationships between the measured and calculated responses of the pavements, and between measured values when measured by different types of instrumentation. Since loads nearly identical in magnitude, location and configuration were used for both the actual pavement loading and the loading conditions assumed for the calculations using the ILLI-SLAB program, the actual loading conditions are not critical as only relative values are needed for this evaluation. It is noted, however, that for the findings presented and discussed herein, the gross weight of the B-727 aircraft was between 95,000 and 120,000 pounds and the gross weight of the DC-10 used to generate these data was approximately 382,700 pounds. For the calculations, it was assumed that 95 percent of the gross weight was distributed equally to all wheels of the main gear, and 5 percent to the nose gear. All calculations were made with an assumed modulus for the concrete of 5×10^6 psi.

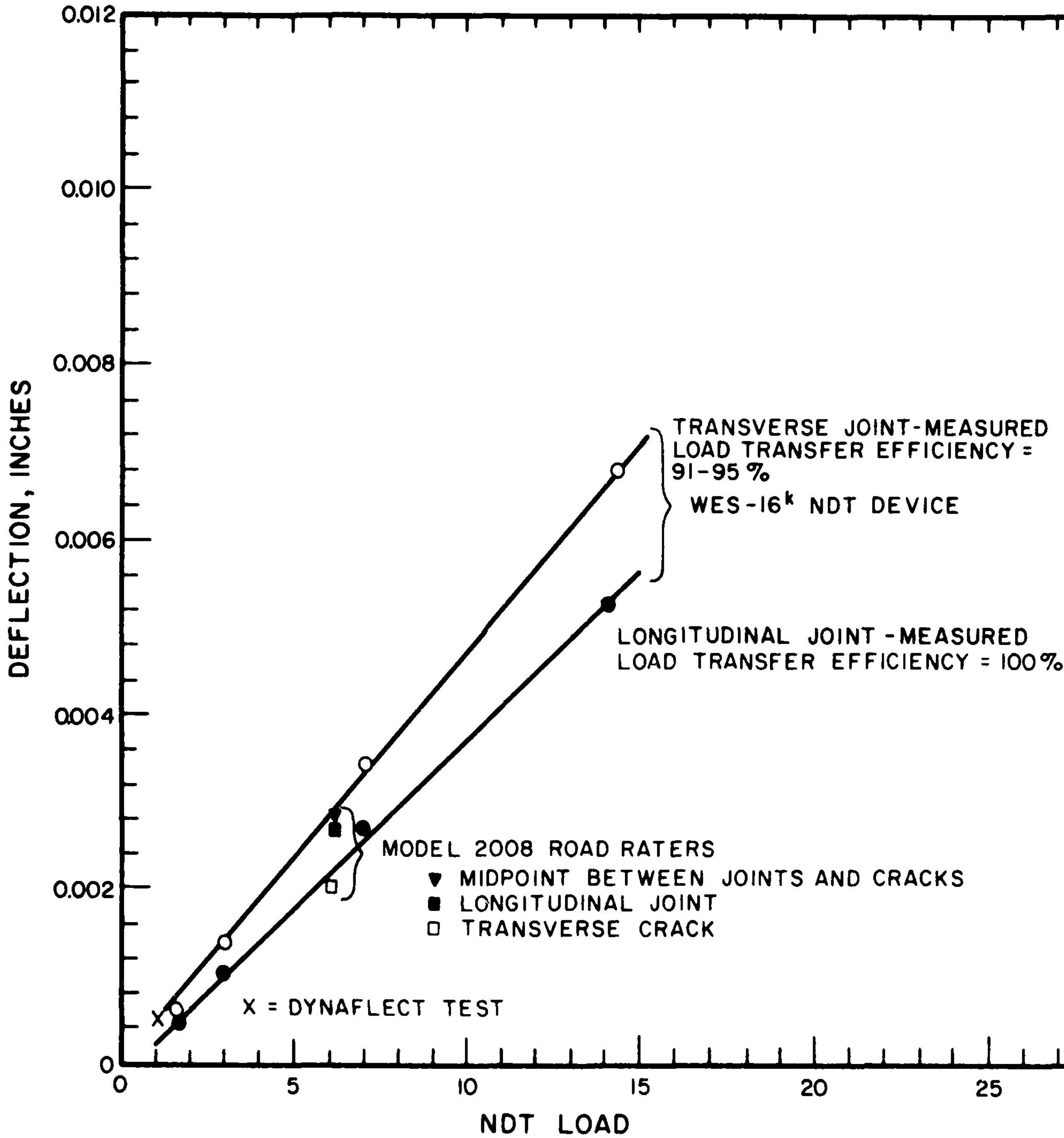
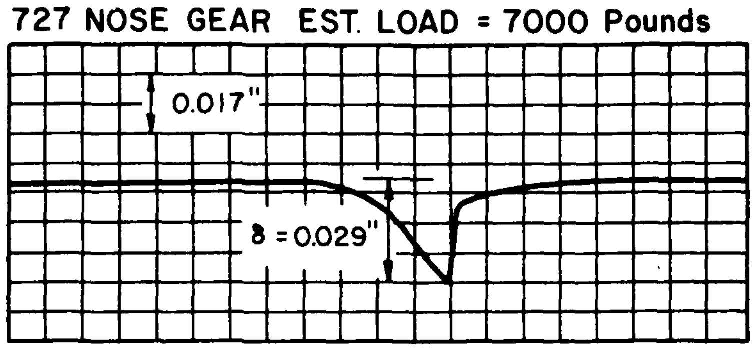


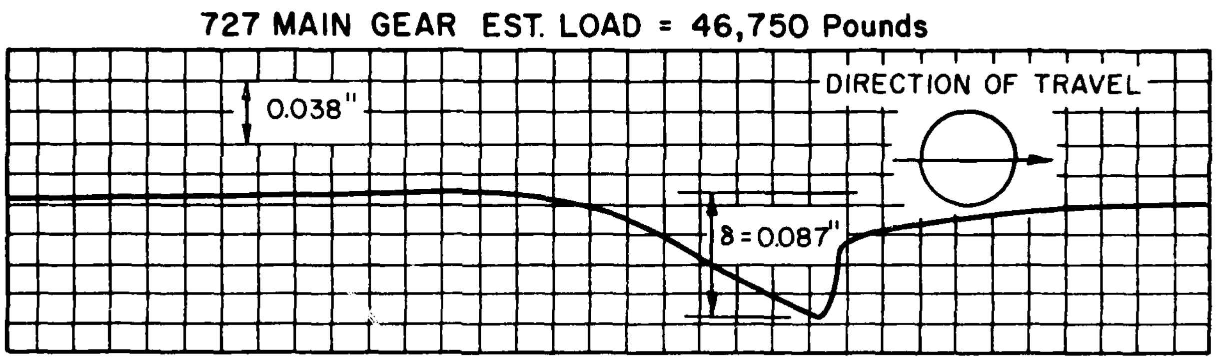
Figure 22. Typical Deflection Results at Site #3, with Loads at Longitudinal Joint and at Transverse Crack

Comparison of the measured and calculated values for the deflections at Site 4 is shown in Table 4. The calculations using both the ILLI-SLAB program and the Pickett and Ray [3] influence charts were made using several "k" values, but the results shown in Table 4 are for the k value of 200 pci as this is approximately the k value used in the design and seems appropriate for the subgrade and subbase conditions encountered.

Comparison of the measured deflection at locations 2 and 3, for Test Site 4, namely of the dowelled transverse and tied longitudinal joints, respectively, shows that the calculated deflection with load transfer efficiencies of between zero and fifty percent yielded deflection results in good agreement with the measured values under the moving aircraft. These assumed load transfer efficiencies are compatible with the load transfer efficiencies at this site determined by measurement with the WES vibratory equipment.

Measured deflections at location 4, Test Site 4, the corner location, are in excess of the calculated values; the exact amount being a function of the load transfer efficiencies assumed across the joints. Reason for this high deflection at the corner is believed to be due to a combination of poor load transfer across the joints, especially the transverse joint, and the probable presence of a void under the slab near the instrumented corner. The probability of such a void was first noted when it was observed that the maximum deflection at this location under the nose gear of the B-727 aircraft was nearly as large as the deflection under the main gear of the aircraft. The relative magnitudes of those deflections are shown in the traces in Figure 23, which were taken directly from the deflection gage installed at that corner. Note that if there were a





1 in. = 25.4 mm
1000 Pounds = 4.448 kN

Figure 23. Deflection Traces for Corner Deflection at Site #4 under Separate Loading Runs with 727 Aircraft

linear relationship between the magnitude of load and the deflection, the pavement deflection under the nose gear would have been .013 inches rather than .029 inches as shown in Figure 23. These values would suggest a void approximately .016 inches (16 mils) in depth.

The absence of load transfer across the transverse joint near the corner of the slab can also be seen from the pattern of the deflection traces. It is seen in Figure 23 that as the nose gear of the B-727 aircraft approaches the transverse joint, there is a gradual increase in the slab deflection. As the wheel crosses the transverse joint to the adjacent or leave slab, however, there is an abrupt reduction in deflection in the instrumented slab, indicating a low level of load transfer efficiency. This was confirmed by measuring the relative deflection across the joint at this location using the geophones and the WES vibrator.

It is believed that much of the difference in the deflections measured with the NDT sensors and the deflection gages at location 4, Site 4, is also due to the manner in which an unsupported pavement responds to loading with the WES vibrator. The initial 16 kip static load of WES equipment forces the pavement systems into contact with the subgrade prior to the start of the dynamic loading. Thus the dynamic response of the pavement as measured with a velocity meter (geophone) would likely be different from the response measured using an absolute deflection gage.

Comparison of the calculated and measured strains at Site 4 are shown in Table 5. The correlation between measured and calculated strains was not as good as for the deflections. For strain gages near the longitudinal and transverse joints (gages 1965T-1964B and 2081T-2092B), measured strains averaged 51.5 percent of the calculated values based on zero load transfer, and 62 percent based on an assumed load transfer efficiency of 50 percent. For the corner load condition, the measured strains were about 50 percent greater than the calculated values assuming a load transfer efficiency of 50 percent across both joints.

Comparison of measured strains produced by the NDT loading with that produced by the aircraft loading indicates some agreement. Figure 14 shows the strains produced by the NDT device and strain data produced from aircraft loading. Keep in mind when reviewing these results that the aircraft gear load is distributed over 2 wheels approximately 3 feet apart (37 inches) whereas with the NDT the load is applied through a single plate.

Deflection data from Site 5 are shown in Table 6. The data indicate the deflections were generally higher than the calculated values. At the transverse joint, for example, the load transfer efficiency at the transverse joint would have to be under 50 percent for the calculated deflection to be equal to the measured value. This is not unrealistic except that the measured load transfer efficiency of this joint was nearly 100 percent at the time of the test. The results from the gage at the center of the slab show the measured deflection higher than the calculated value. This can only be explained by assuming the support conditions for the pavement were highly nonuniform or that the slab had curled up at the time of testing. Obviously, assuming a uniform support condition, the slab deflection at the interior will never be equal to that under edge load, even if the load transfer at the joints was 100 percent effective.

The graphs in Figures 18 and 19 show some of the measured deflections at Site 5 under dynamic NDT and moving DC-10 and B-727 aircraft. It is apparent from these curves that there is a general agreement between the

Table 5. Calculated and Measured Strains for Site 4

Gage Number[a] (Fig. 10)	Maximum Measured Strain, μ in./in.		Calculated Strains, μ in./in., for Indicated Load Transfer Efficiency				
	Actual	Corrected	0[b]	0[c]	50[c,d]	67[c,d]	90[c,d]
1965T	11.98	18.0	41.0				
1964B	--		--	44.2	32.5	26.7	24.3
2081T	15.00	22.6					
2082B	18.00	27.5	46.3	55.2	37.9	34.1	32.3
1793T	14.53	27.5	--	--	13.75	--	--

a T = Top gage
B = Bottom gage

b Calculated using influence charts developed by Pickett, et al [3] and assuming $E_{conc} = 4 \times 10^6$ (28 GPa)

c Calculated using ILLI-SLAB with $E_{conc} = 4 \times 10^6$ (28 GPa)

d Approximate efficiencies only as efficiencies change slightly with subgrade support, slab thickness, and load location and direction

Table 6. Comparison of Measured and Calculated Deflection for Site 5 for DC-10 Aircraft

Gage Location (Fig. 17)	Measured Deflection (inches)	Calculated Deflection, inches, for Indicated Load Transfer Efficiency				
		0^a	0^b	$50^{b,c}$	$67^{b,c}$	$90^{b,c}$
Center of slab	.028-.030	--	.0160	--	--	--
Center of transverse joint	.030	.0377	.0389	.0292	.0229	.0215

a Calculated using influence charts developed by Pickett, et al [3] k = 200

b Calculated using ILLI-SLAB [1] k = 200

c Approximate efficiencies as the actual efficiency changes with subgrade modulus, slab thickness, and load location and direction

d Corner deflection gage not operational for this test deflections under the two types of loading, but that the deflection under the NDT type loading when extrapolated to the aircraft gear loads would be significantly higher for equivalent magnitudes of load. This is as expected as the NDT load was applied through a single load whereas the aircraft gear loads were applied through gears with either 2 or 4 tires spaced between 3 and 5 feet apart.

Some of the calculated and measured strains at Site 5 under the DC-10 aircraft gear load are shown in Table 7. As with the results from Site 4, the measured strains are somewhat less than the calculated strains. For the interior load condition the measured strains are, on the average, about 57 percent of the calculated values. For the transverse joint the measured values are approximately 37 percent of the calculated values, assuming a high LTE across the joint.

Comparison of measured strains under NDT and aircraft gear loading is shown in Figures 20 and 21 for the interior and transverse joints. These results indicate the measured strains under the NDT loading are considerably higher than the measured strains for comparable loads with the aircraft gear.

Taking an overview of all results presented, it is apparent that there is better agreement between the measured and calculated deflections than there is between the measured and calculated strains. In general, the measured deflections appear to fall within about 10 to 20 percent of the calculated values whereas the measured strain are only about 50 to 60 percent of the calculated values. At this time the author has no valid explanation for his discrepancy. It is not unusual, however, to have great difficulty in getting agreement between calculated and measured strain values, and there exists a question of whether these differences are due

Table 7. Comparison of Measured and Calculated Strains for Site 5

Gage Number[a] (Fig. 17)		Maximum Measured Strain, μ in./in.		Calculated Strains, μ in./in. for Indicated Load Transfer Efficiency				
		Actual	Corrected	0[b]	0[c]	50[c,d]	67[c,d]	90[c,d]
1966T 1967B	Center of slab	9.6 9.0	13.4 13.0	--	23.2	--	--	--
2069T 2070B	Transverse joint	10.3 10.4	14.4 15.0	59.1	63.1	50.2	43.9	41.7

a Calculated using influence charts developed by Pickett, et al [3] k = 200

b Calculated using ILLI-SLAB [1] k = 200

c *Approximate efficiencies as the actual efficiency changes with* subgrade modulus, slab thickness, and load location and direction

d Corner deflection gage not operational for this test to the method of measurement or to the analysis procedures. Unfortunately, there are no methods of independently checking the accuracy of the measurement system, so these have to be taken at face value.

Summary and Conclusions

A number of trends are apparent in the data and information provided herein and in the referenced reports. The data presented herein do not totally validate or fully confirm all of the trends and conclusions presented below, but as these data do confirm similar trends and conclusions from other observations it is believed that the data presented are sufficient to justify the conclusions given.

1. Testing PCC pavements with NDT equipment at some point in mid-slab, i.e., away from all cracks and joints, will not provide meaningful data for predicting the life of these pavements.
2. The present and future behavior and performance of PCC pavements is controlled almost exclusively by the thickness of the concrete slab and the joint conditions which exist.
3. To obtain meaningful test results from any NDT, the NDT equipment must apply sufficient force to the pavement to activate a response from the entire pavement system. That is, the force must be great enough to provide meaningful stresses on the subgrade as well as sufficient deflection of the pavement system so that these deflections can be properly monitored. (Note: Data from the dynaflect device are not shown as the deflections were too small to be monitored by independent gages.)
4. The relative deflections and strains measured using the NDT and the aircraft loads indicate a reasonable agreement in the pavement's

response to these two methods of loading provided the NDT loads were large enough to fully activate the pavement responses.

5. Breakdown of the structure of jointed PCC pavements starts with a breakdown of the load transfer capacity at the corners and progresses along the joints towards the midpoints of the slabs.

6. Where the PCC slabs had retained adequate support near the edges and corners, the strains measured in the slabs under both the NDT and aircraft loadings were generally below that predicted by appropriate theory, such as with the ILLI-SLAB program or with the Pickett and Ray influence charts.

7. With the progressive breakdown of load transfer at the corners of the PCC slabs, stresses applied to the subgrade due to the heavy gear loads are sufficient to cause a gradual loss of support for the slab and a concomitant increase in distress in the slabs, especially near the corners.

Based on the above observations it seems to follow logically that to effectively evaluate the condition and structural capacity of PCC pavements the test procedure must include an evaluation of the load transfer efficiency across all joints and especially near the intersections of two joints or at the intersections of joints and cracks (i.e., near corners of the slabs). Also, since the breakdown appears to be strongly correlated with the number of loads the pavement has carried, the testing must be done near the center of the aircraft wheel patterns.

As a follow-up on the test procedures, it follows that to prolong the life of those pavements exhibiting some breakdown of load transfer but no other distress, attempts should be made to reestablish load transfer near the corners, and the slabs should be pressure grouted to reduce slab deflection near the corners.

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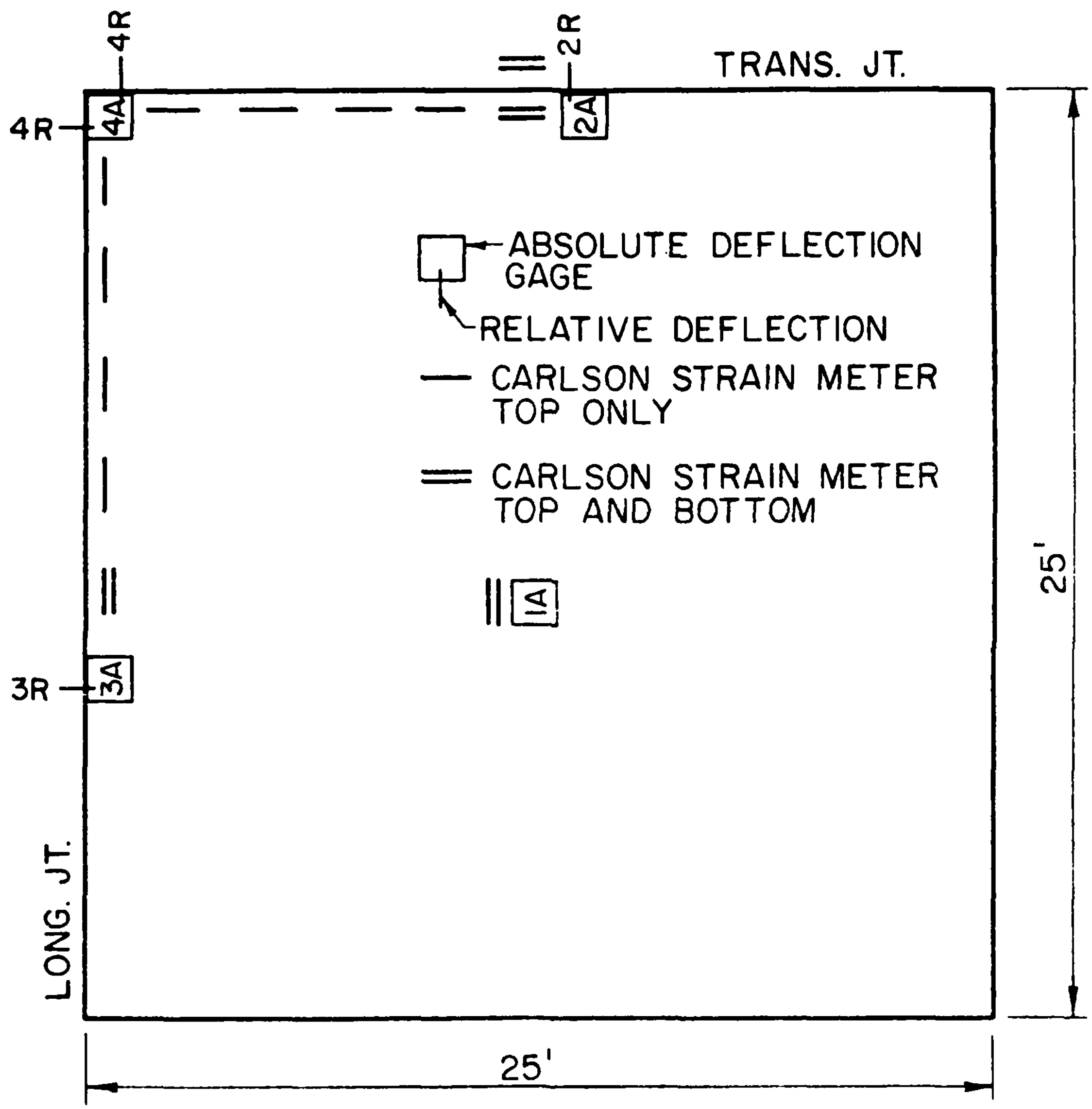
DOT/FAA/RD-79/4 - LONGITUDINAL JOINT SYSTEMS IN SLIP FORMED RIGID PAVEMENTS: The following volumes are available from the National Technical Information Service, Springfield, Virginia 22161:

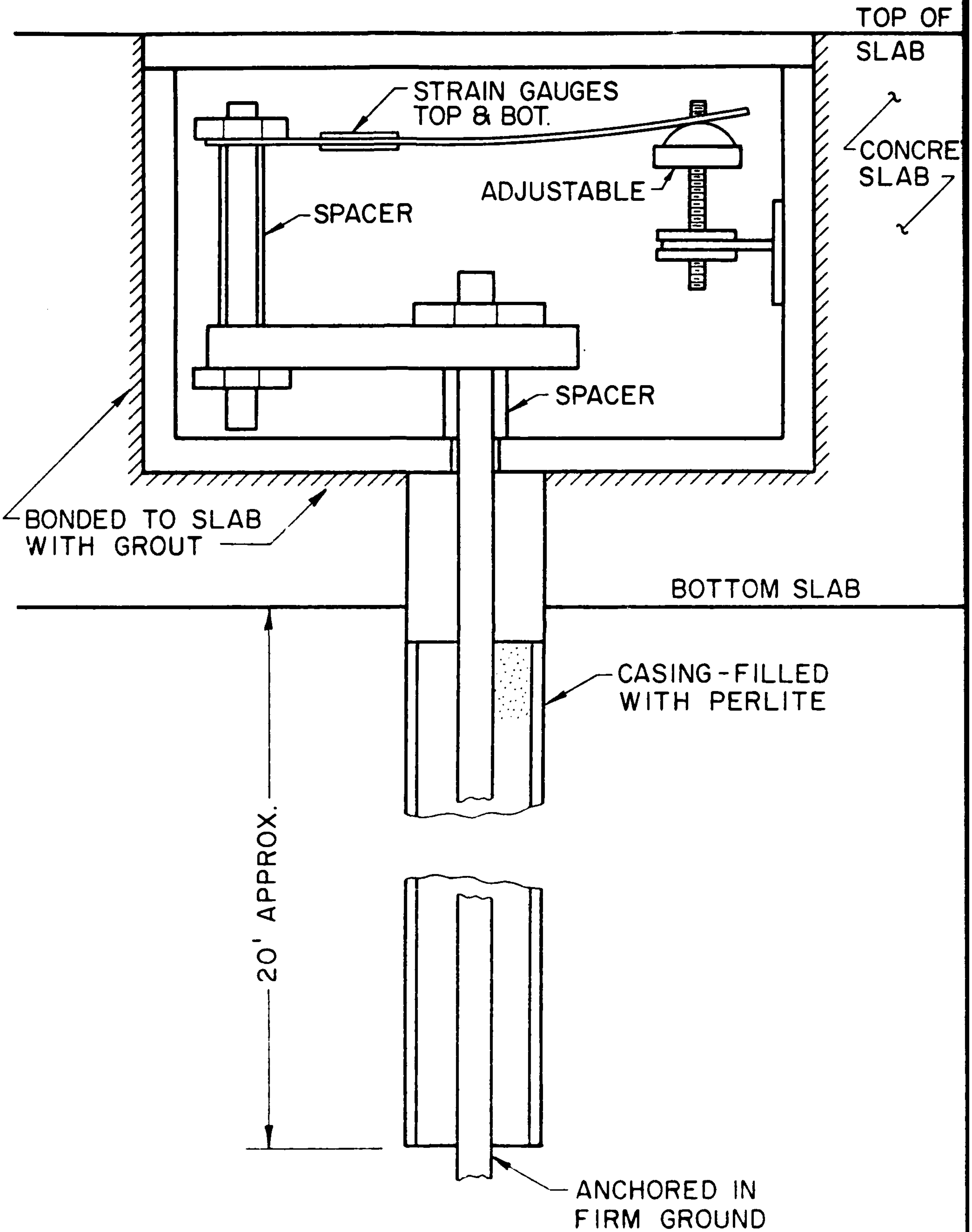
Vol. I - Literature Survey and Field Inspection, ADA 066320
Vol. II - Analysis of Load Transfer for Systems for Concrete Pavements, ADA 078836
Vol. III - User's Manual, ADA 078850
Vol. IV - Recommendations for Alternate Joint Systems and for Strengthening Existing Joints, ADA 112970

APPENDIX

DETAILS OF INSTRUMENTS INSTALLED AT O'HARE INTERNATIONAL AIRPORT



TYPICAL SLAB INSTRUMENTATION


TOP OF
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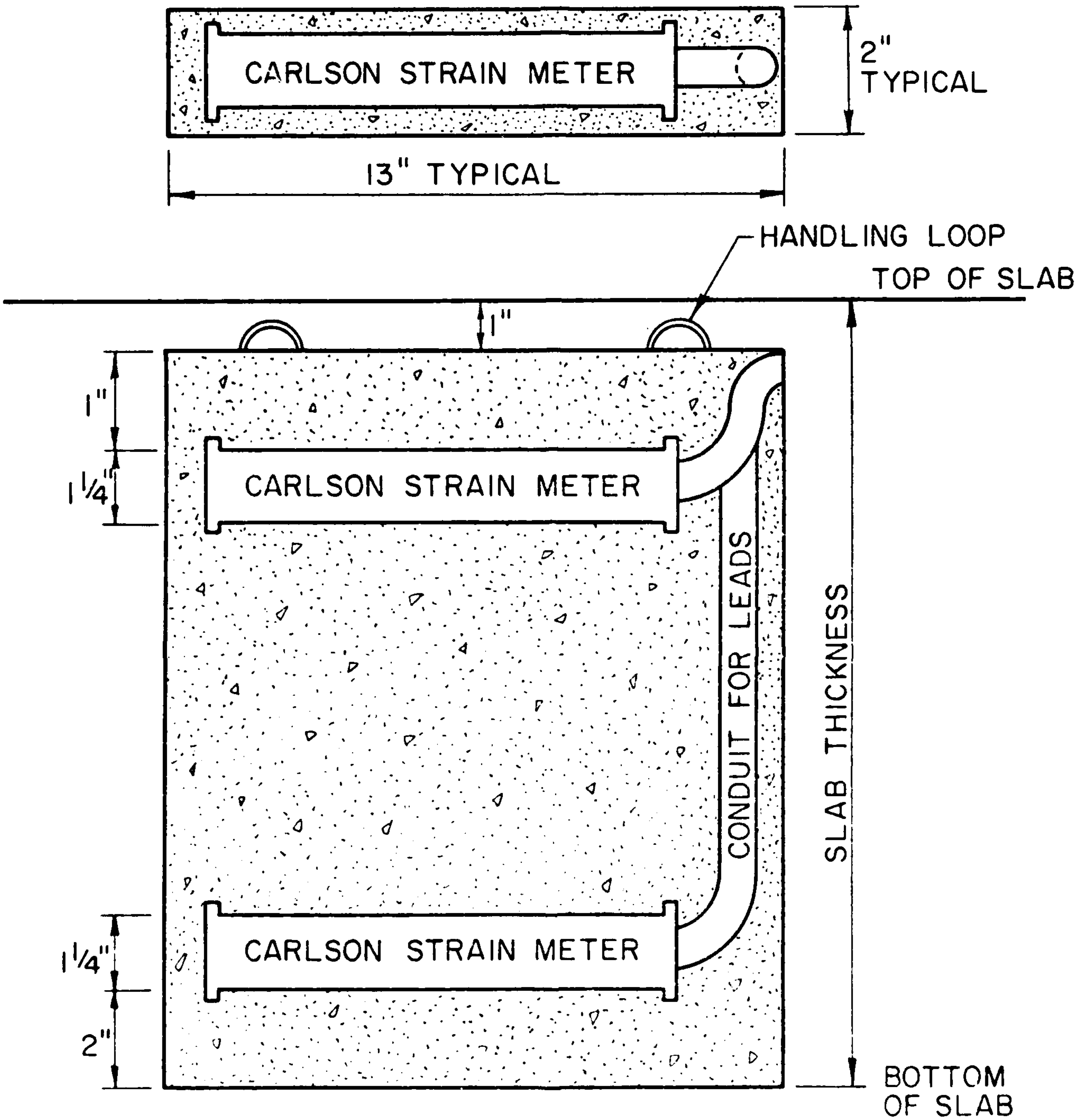


INSERTS FOR INSTALLATION OF STRAIN METERS

CARLSON ELASTIC WIRE STRAIN METER*

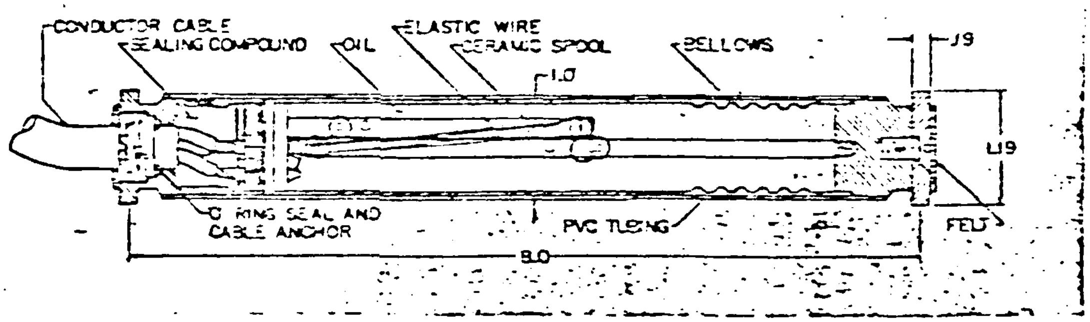
The standard strain meter can be embedded in concrete or it can be attached to a surface with saddle mounts. It measures change in length (strain) and temperature with the help of a simple Wheatstone-bridge type testing set, or the new *Carlson* Digital Test Set. The meter contains two coils of highly elastic steel wire, one of which increases in length and electrical resistance when a strain occurs, while the other decreases. The ratio of the two resistances is independent of temperature (except for thermal expansion) and therefore the change in resistance ratio is a measure of strain. The total resistance on the other hand is independent of strain since one coil increases the same amount as the other decreases due to a change in length of the meter. Thus, the total resistance is a measure of temperature. The improved strain meter is a better thermometer than the earlier ones, which had one coil within the other and therefore were of different lengths.

The strain meter is furnished in three different lengths, from 8 inches to 20 inches, but all with the identical sensing element. The end away from the cable has a tapped hole (1/4-28 UNF) to permit attachment to a spider for mass concrete embedment, or for adding an extender to increase the length and sensitivity. The body is covered with PVC sleeving to break the bond with the concrete.

The conductor cable most commonly used is neoprene rubber-covered, portable cord with either three or four conductors. The four-conductor cable permits the testing set to make automatic subtraction of cable resistance for the determination of temperature only. If the user does not specify cable length, the meter is supplied with 30 inches of 16/3 SO cord. However, it is often preferred to attach the cable at the job site in the full length to be needed.

Carlson Strain Meters are now covered with a "sleeve" of PVC tubing which is lubricated internally with silicone. A result of this process is that the sealing chamber is covered. Due to the difficulty of slipping the PVC over the meter, it is recommended that the user order the meters with 30" of 16/3 or 16/4 SO cable attached and splice his cable at the job site. Several companies offer excellent splicing kits for this purpose. It is recommended that no greater than 600 feet of 16 AWG cable be used. Larger wire should be used with longer lengths.

The strain meter frame is all steel, making the temperature correction (for thermal expansion of the frame) 6.7 microstrains per degree. This value is nearly the same as the thermal expansion of the concrete and is advantageous in that little of the range of the meter is lost due to temperature change.



SPECIFICATIONS—"A" SERIES

(See "Accuracy of Carlson Strain Meters")

Model Number	A8	A10	A10S**	A20
Range (micro-strain)*	2500	2100	2100	1050
Least reading (micro-strain), max.	3.5	2.9	2.9	1.5
Least reading, temperature (°F)	0.1	0.1	0.1	0.1
Gauge length (inches)	8	10	10	20
Weight (lbs.)	.8	1.3	1.3	1.8

*Normally set at factory for 2/3 of range in compression. Within limits, other settings may be specified.

**Saddle mount, mounting diameter is 1-1/16 inches.

CARLSON INSTRUMENTS

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Manufacturer's literature regarding strain meter instrumentation

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