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THE KANSAS TEST TRACK

PART I - ANALYSIS OF TEST DATA

NOVEMBER 1979

FINAL REPORT

Prepared for

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL RAILROAD ADMINISTRATION
Office of Research and Development
Washington, D.C. 20590

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16. Abstract

This report presents the results of an experimental project to compare the performance of different track support systems.

Data obtained during the project are summarized, presented, and analyzed. Based on these data, conclusions are made regarding the performance of the different track support systems.

Details of instruments used for data measurement, their location in track, test procedures, and test data are presented in Part II of the report.

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PREFACE

This report has been prepared as a part of a subcontract between the Atchison, Topeka, and Santa Fe Railroad and the Construction Technology Laboratories of the Portland Cement Association.

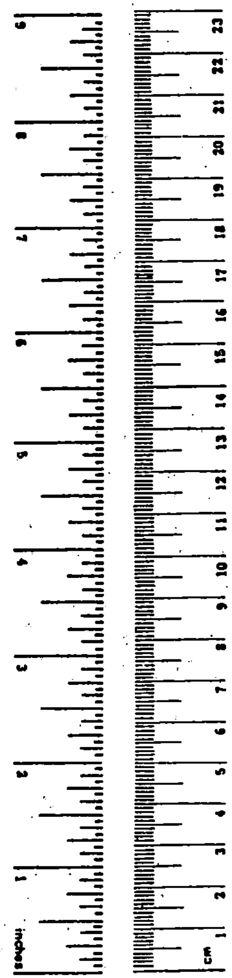
Analysis of test data obtained from the Kansas Test Track is reported. Work on the project was carried out by the Transportation Development Section under the direction of Bert E. Colley and the Structural Development Section under the direction of W. Gene Corley and Henry G. Russell. Particular recognition is given to Richard D. Ward, William Hummerich, Jr., George Fessler, and other project staff for their assistance and suggestions in the design, installation, and monitoring of the instrumentation.

Dr. R. M. McCafferty and Mr. W. B. O'Sullivan of FRA were the technical monitors for the work reported herein. Their cooperation and suggestions are gratefully acknowledged. Also, the cooperation and assistance provided by the Atchison, Topeka, and Santa Fe Railroad, in particular that of Mr. W. S. Tuinstra and W. S. Autrey is acknowledged.

METRIC CONVERSION FACTORS

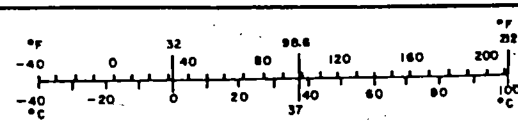
Approximate Conversions to Metric Measures

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



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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HIGHLIGHTS AND RECOMMENDATIONS

In 1971-72 a 1.8 mile test track containing concrete tie, slab, beam, and wooden tie test sections was constructed on the Atchison, Topeka, and Santa Fe Railway Company mainline near Aikman, Kansas. Test sections were instrumented with load sensors, soil pressure cells, deflection meters, strain gages, and accelerometers.

Shortly after the test track was opened to traffic, numerous fastener anchorages pulled out of the slab and beam sections forcing suspension of service. A new replacement anchorage system was developed and special construction details and alignment jigs were prepared. The replacement system increased anchorage capacity by 5 times and performed satisfactorily throughout the remainder of the test.

Field data were obtained on three separate occasions during late 1974 and early 1975. In addition, limited data were obtained in April 1973 prior to the installation of the new slab and beam anchorage system.

Initial track modulus values ranged from 2,900 lb/in./in. for the wooden tie section to 16,100 lb/in./in. for the concrete slab section. Modulus of each tie section remained relatively constant with time and traffic. Beam and slab section modulus values decreased 62 to 82% by end of test.

Special load cell ties constructed to simulate wooden and concrete ties showed that ballast reaction on the ties was initially highest directly under the rail seats, but gradually tended toward a uniform distribution with time and traffic. Largest pressures measured for wooden and concrete tie sections were 60.2 psi and 50.4 psi, respectively.

Tie strains and computed bending moments were consistently low and could not be used to explain the presence of cracks observed during the final inspection of the test track. Computed bending moments were never more than 2/3 of the established cracking moment and typically were less than 1/3 of the established cracking moment.

Maximum recorded tensile stresses in the slab and beam sections were 77 and 404 psi, respectively. Maximum recorded compressive stresses were less than 300 psi. Maximum reinforcing steel stress was 8,200 psi.

Rail stresses at and between fasteners averaged approximately the same for all test sections. Average of the maximum stresses measured for the various sections and recording periods was 8,180 psi. Maximum calculated stress was 18,400 psi.

In June 1975 the test track was closed and the project was terminated due to subgrade failure.

Laboratory tests of the concrete ties showed that they met 1971 AREA specification requirements. Tests on ballast showed that the concrete tie withstood 8.25 million load cycles without any visible damage, while the wooden tie became spongy under the tie plates, spikes loosened slightly, and one steel tie plate cracked.

Based on the work performed in this project, the following recommendations are made concerning future test tracks:

1. Special attention should be given to the selection and use of suitable foundation materials.
2. Large amounts of instrumentation should not be clustered into one small area. Subgrade instrumentation should not be grouped together in such a manner to weaken or disturb normal subgrade condition. All instrumentation should be laid out so that data obtained are representative of a typical track section.
3. If the effect of tonnage and time are to be investigated, the condition of the test track should not be changed unless it is absolutely necessary. When work is done on the test track, the time and extent of work should be documented. When major changes are made, data should be obtained just prior to and immediately after the changes are made.

INTRODUCTION

In the early 1970's the Federal Railroad Administration (FRA) and the Atchison, Topeka and Santa Fe Railway Company (ATSF) jointly sponsored an investigation to evaluate the performance of different track support systems. For this purpose, a 1.8-mile long test track consisting of nine sections was constructed adjacent to the mainline track of the ATSF, between Aikman and Chelsea, Kansas. These test sections included continuously reinforced concrete slabs, reinforced concrete twin beams, prestressed concrete ties, stabilized ballast, and standard wood-tie track sections. The site was chosen because of abundant rail traffic, a long tangential section of track, relatively flat uniform grade, and climatic conditions typical of vast areas in the continental United States.(1)

The test track was designed and constructed by several organizations under sub-contract to ATSF, the project administrator. The Construction Technology Laboratories (CTL), a division of Portland Cement Association (PCA), was engaged to instrument the test track, obtain data periodically, reduce and analyze data, and submit a report covering the findings. The design, development, and preparation of installation specifications for a new rail fastening anchorage system, to replace the one originally installed in the slab and beam test sections, also was done by PCA.

Static and dynamic data collected from embankment instrumentation at the Kansas Test Track (KTT) were analyzed by Shannon and Wilson, Inc.(2) Embankment instrument studies were performed by the U.S. Army Engineer Waterways Experiment Station. Mitre Corporation conducted post-mortem static and dynamic tests to validate a dynamic track structure model for the KTT beam section.

Objective

The objective of the project was to compare the performance of eight track structures of differing stiffness with each other, and with the performance of a standard ATSF wood-tie track structure.

Analysis and evaluation of data obtained from instrumentation installed in the different test sections were used to accomplish the objective.

Track Details

The test track was constructed parallel to the existing single ATSF mainline track as shown in Figure 1, just west of Mile Post 161. Center-to-center distance between track was 30 ft. The length of the tangential portion of the track was approximately 1.8 miles. Within this length were located 9 test sections. These sections were designated Section 1 through 9 in sequence from east to west as shown in Figure 2, and consisted of the following types of track:

1. Concrete ties spaced at 30 in. center-to-center and 10 in. ballast below the ties.
2. Concrete ties spaced at 27 in. center-to-center and 10 in. ballast.
3. Concrete ties spaced at 24 in. center-to-center and 10 in. ballast.
4. Continuously reinforced concrete twin beams.
5. Continuously reinforced concrete slab.
6. Stabilized or "glued" ballast. This section was similar to ATSF standard track (Section 9), except for ballast treatment.
7. Precast, reinforced concrete twin beams.
8. Concrete ties spaced at 27 in. center-to-center and 15 in. ballast.
9. Standard ATSF mainline track construction, to serve as a reference. This track structure consisted of 7 in. x 8 in. x 9 ft hard wood ties spaced at 19.5 in. center-to-center and 10 in. ballast.

All test sections were 800-ft long, except Section 6, which was 545-ft long. A 136 RE, continuously-welded rail, and crushed, ferrous metal slag ballast were used in all sections.

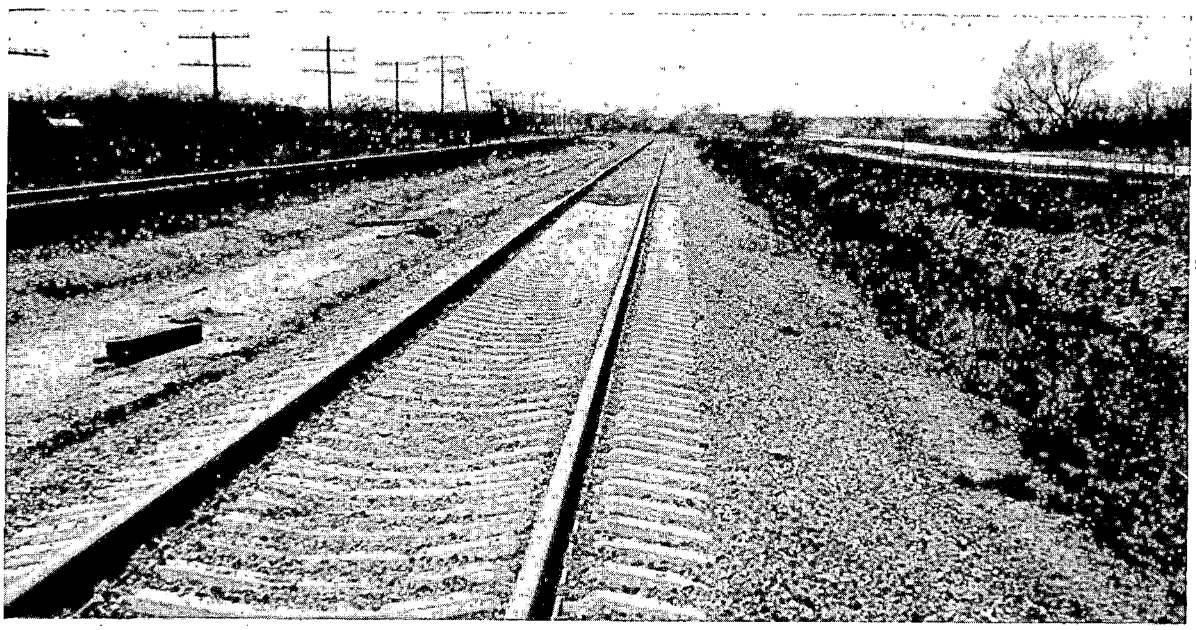


FIGURE 1 - TEST TRACK ADJACENT TO MAINLINE

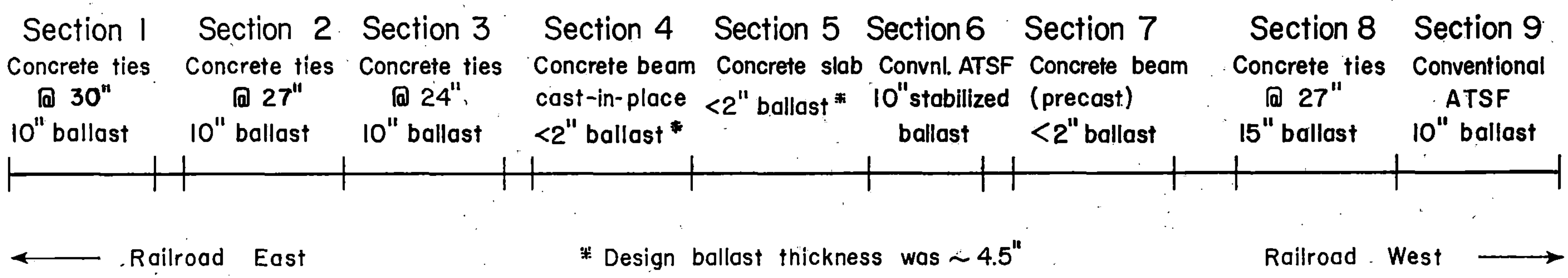


FIGURE 2 - SCHEMATIC PLAN OF TEST TRACK

General views of the wood tie, concrete tie, twin beam, and concrete slab sections are shown in Figures 3, 4, 5 and 6, respectively.

Track support systems were arranged in the test track in order of increasing stiffness in Sections 1 through 5, and varying stiffness in Sections 6 through 9. Special 40-ft long transitions, consisting of sub-ballast, slabs of varying thickness, and concrete ties at different spacings, were located at the east end of Sections 4 and 7 and at the west end of Sections 5 and 7. These transitions provided a gradual change in track stiffness between the rigid structures and the cross-tie portions of the track.

Additional details of the test facility and its physical characteristics are described in another report.[3]

Chronology

Project construction started on July 26, 1971. Completion of construction and diversion of ATSF traffic to the test track was expected by November 1972. During this period appropriate instrumentation was to be procured or fabricated and installed in the various test sections. Data was to be gathered on a quarterly basis for the first year of traffic, and once a year thereafter for three years until project completion late in 1976. The initially planned schedule of the project is illustrated in Figure 7.

Due to construction delays, traffic was not diverted to the test track until May 1973. This was to be followed by a two-week shakedown period for track adjustments. During this period, numerous fastener anchorages pulled out of the slab and beam sections, forcing suspension of service after 28 hours. A new fastening anchorage system was installed during the summer of 1974. The test track was placed in normal service on October 31, 1974.

The track was closed and scheduled testing terminated on June 11, 1975, after approximately six months of service. This

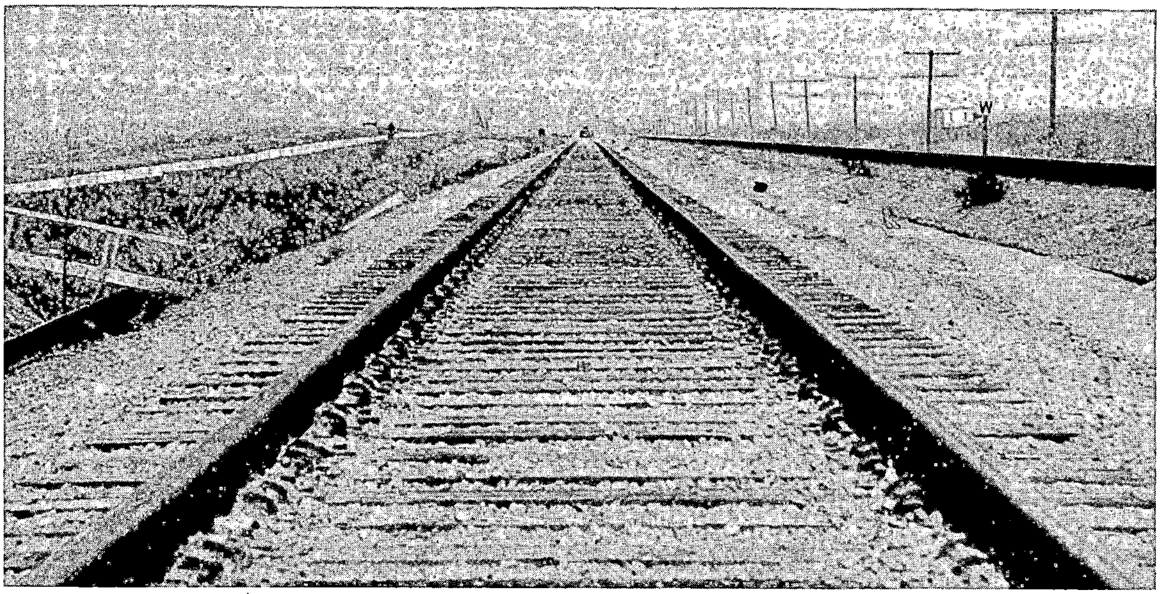


FIGURE 3 - GENERAL VIEW OF WOOD TIE SECTION

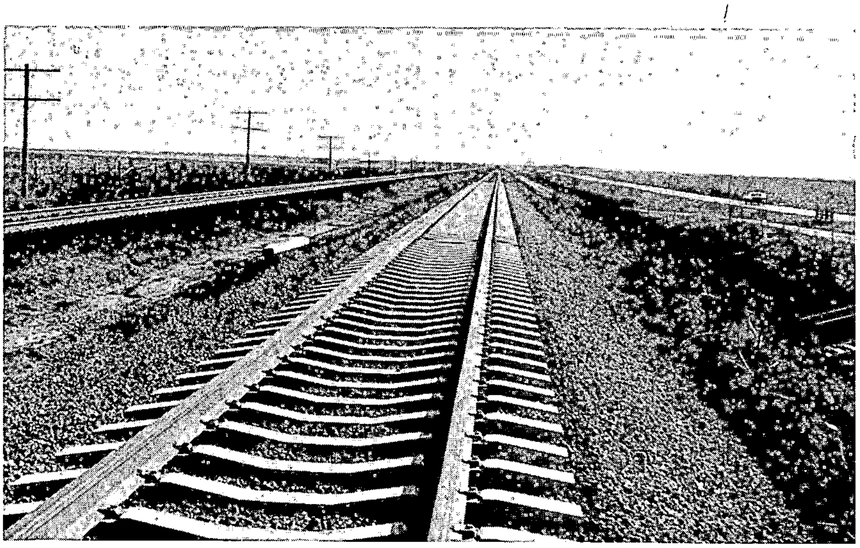


FIGURE 4 - GENERAL VIEW OF CONCRETE TIE SECTION

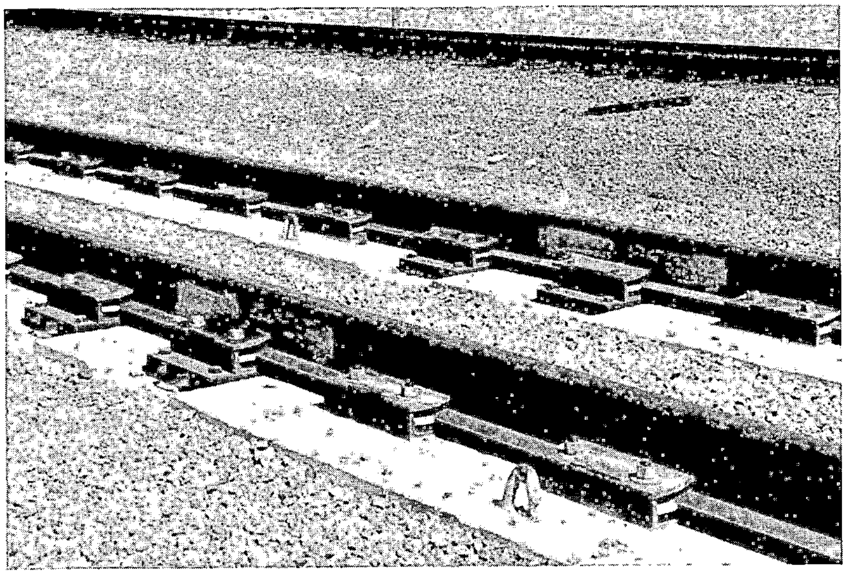


FIGURE 5 - GENERAL VIEW OF CONCRETE BEAM SECTION

FIGURE 6 - GENERAL VIEW OF CONCRETE SLAB SECTION

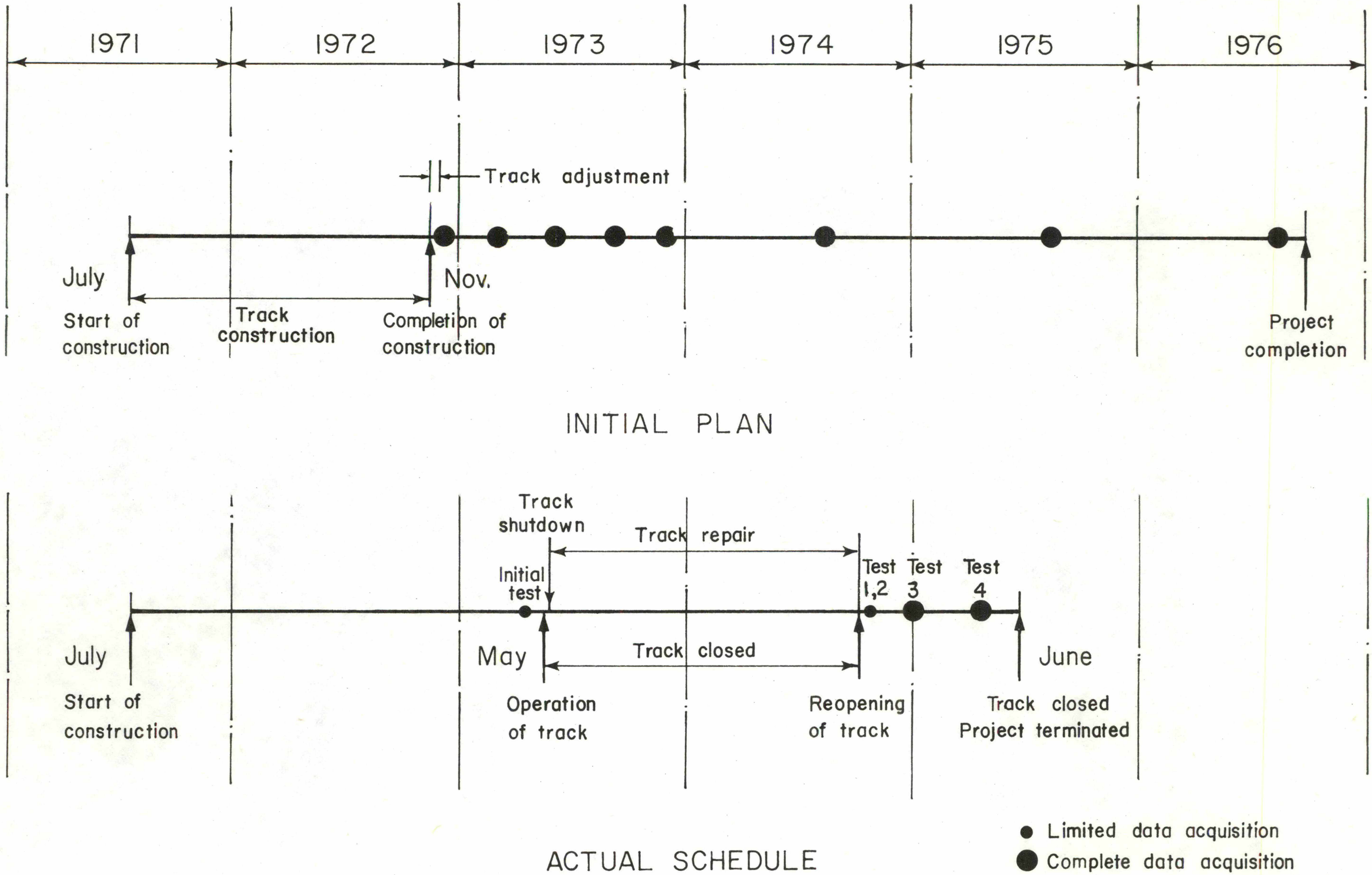


FIGURE 7 - PROJECT SCHEDULE

action was taken due to subgrade failure that resulted in excessive track deflections and mud pumping throughout most of the test sections.

As illustrated in Figure 7, the actual schedule consisted of five tests as follows:

- An initial test was made in April 1973 prior to opening track to traffic. In this test, limited data were obtained at creep speed.
- Tests 1 and 2 were made in October 1974 for the slab and beam track section after installing the new anchorage system, and in November 1974 for the cross-tie track sections. In these tests limited data were obtained at creep speed and 30 mph.
- Test 3 was made in December 1974 for the slab and beam track sections and in January 1975 for the cross-tie sections. All data were obtained at 30 mph.
- Test 4 was made in April 1975 on all track sections. Data were obtained for normal traffic at a speed of 50 mph.

EXPERIMENTAL PROGRAM

To accomplish the project objective, an experimental program was implemented to obtain track performance data. Type of data obtained, layout and details of instrumentation used, test procedures followed, and the constraints that influenced implementation of the test program are described.

Track Performance Data

Data obtained from instrumentation in the various track sections included:

1. Rail stresses at and between fastenings.
2. Vertical and longitudinal rail deflections.
3. Accelerations of rails and ties.
4. Rail seat loads and flexural strains in ties.
5. Ballast pressure distribution at the tie-ballast interface.
6. Pressure distribution at the ballast-subgrade interface.
7. Vertical deflections of slabs and beams at and between joints.
8. Strains in concrete and reinforcing steel in slabs and beams.
9. Fastener loads on beams and slabs.
10. Joint opening and settlement of beams and slabs.
11. Embankment data. These and other data were obtained from instrumentation installed in the embankment and were given, in a raw form, to Shannon and Wilson, Inc. for analysis and report.(2)

Part of the original program was to monitor changes in instrumentation response to determine the effect of time and traffic on the performance of track structure components. However, since only two scheduled data-acquisition periods were made, field data showing the influence of time and traffic are limited.

Instrumentation Layout

Instrumentation installed in the embankment by Shannon and Wilson, Inc. was identical for each track section. Main array instrumentation consisted of multi-position vertical extensometers, soil pressure cells, moisture and temperature cells, and plastic tubes for horizontal extensometers. Single-position vertical extensometers were located 100 ft, 200 ft and 300 ft east, and 100 ft west of the main array.(4)

Instrumentation installed in the track sections by PCA was located so as not to influence or be influenced by those installed by Shannon and Wilson. Figure 8 shows instrumentation locations in cross tie track sections. Instrumentation layout for slab track and beam track sections is illustrated in Figures 9 and 10, respectively.

In addition to track instrumentation near the main array, a secondary array was installed in the slab track section near its west end. This array was to provide information on the performance of slab sections near the ends. Also the east end of the cast-in-place beam track section contained a secondary array as shown in Figure 11. This array was to provide information on the effects caused by change in track structure stiffness.

Instrumentation arrays were installed in the embankment, 600 ft from the east end of all sections, except Section 6 where this distance was 460 ft. Locations of main instrumentation arrays were chosen to enable westbound traffic to assume similar riding characteristics on each section by the time it reached the arrays.

Details of Instrumentation

Instrumentation installed in the Kansas Test Track consisted of strain gages, pressure cells, load cells, accelerometers, deflectometers, and reference devices such as monuments, rods, and plugs.

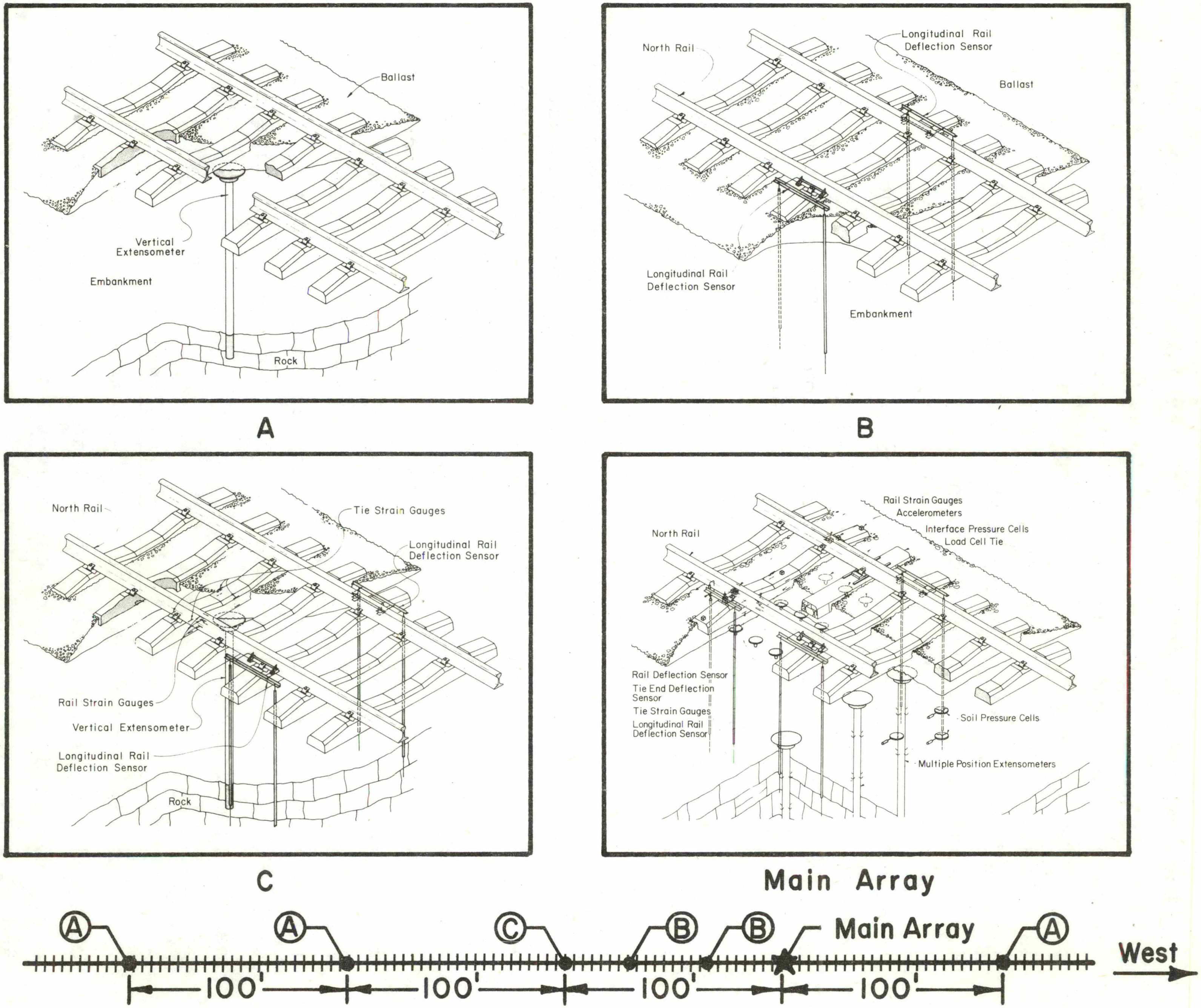
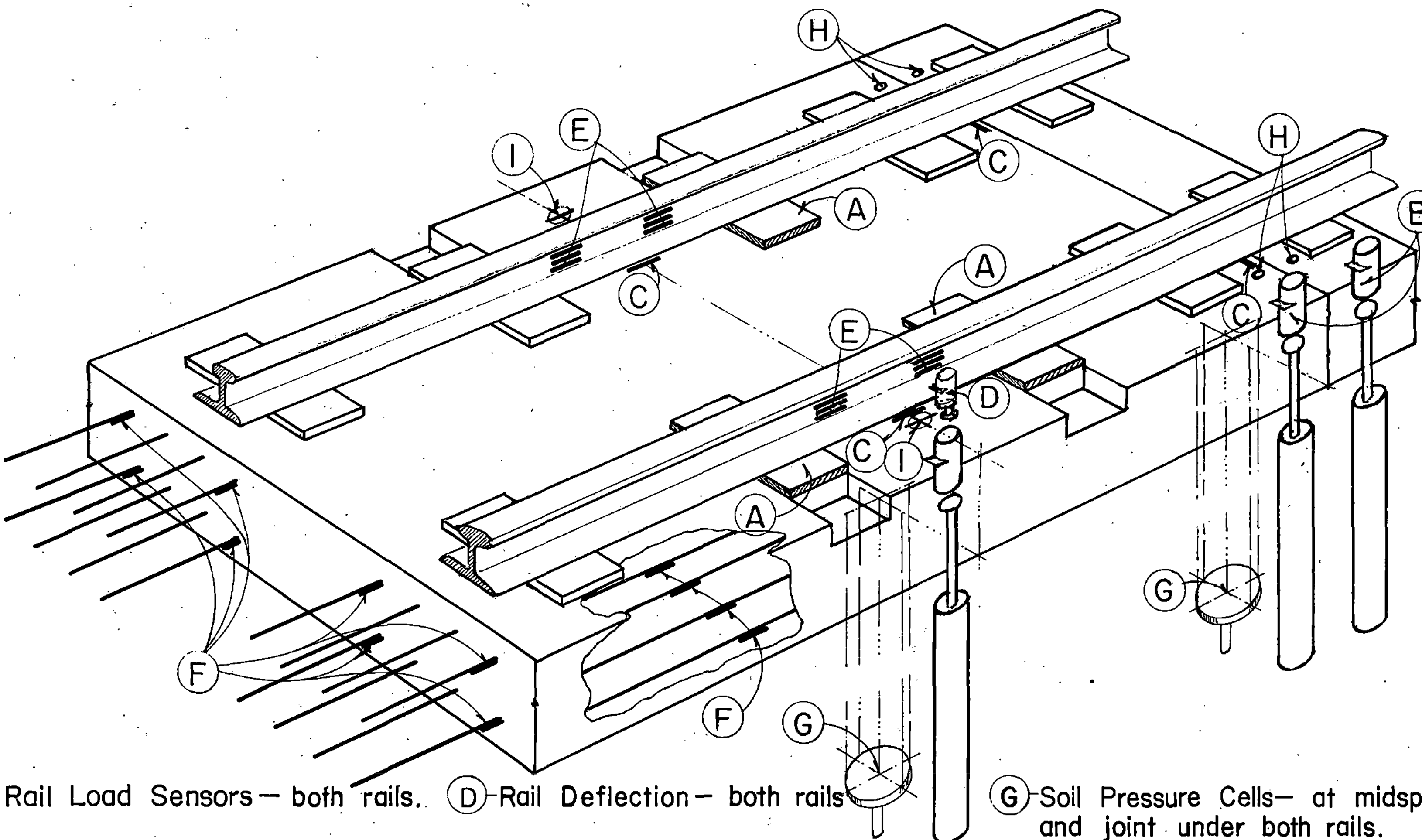


FIGURE 8 - INSTRUMENTATION LAYOUT FOR TIE TRACK SECTIONS



(A)-Rail Load Sensors — both rails.

(B)-Slab Deflection Meters — both sides of joint and midspan.

(C)-Concrete Strain Gages — top, mid and bottom at midspan — top and bottom at joints.

(D)-Rail Deflection — both rails

(E)-Rail Strain Gages — top, mid and bottom at both sides of both rails.

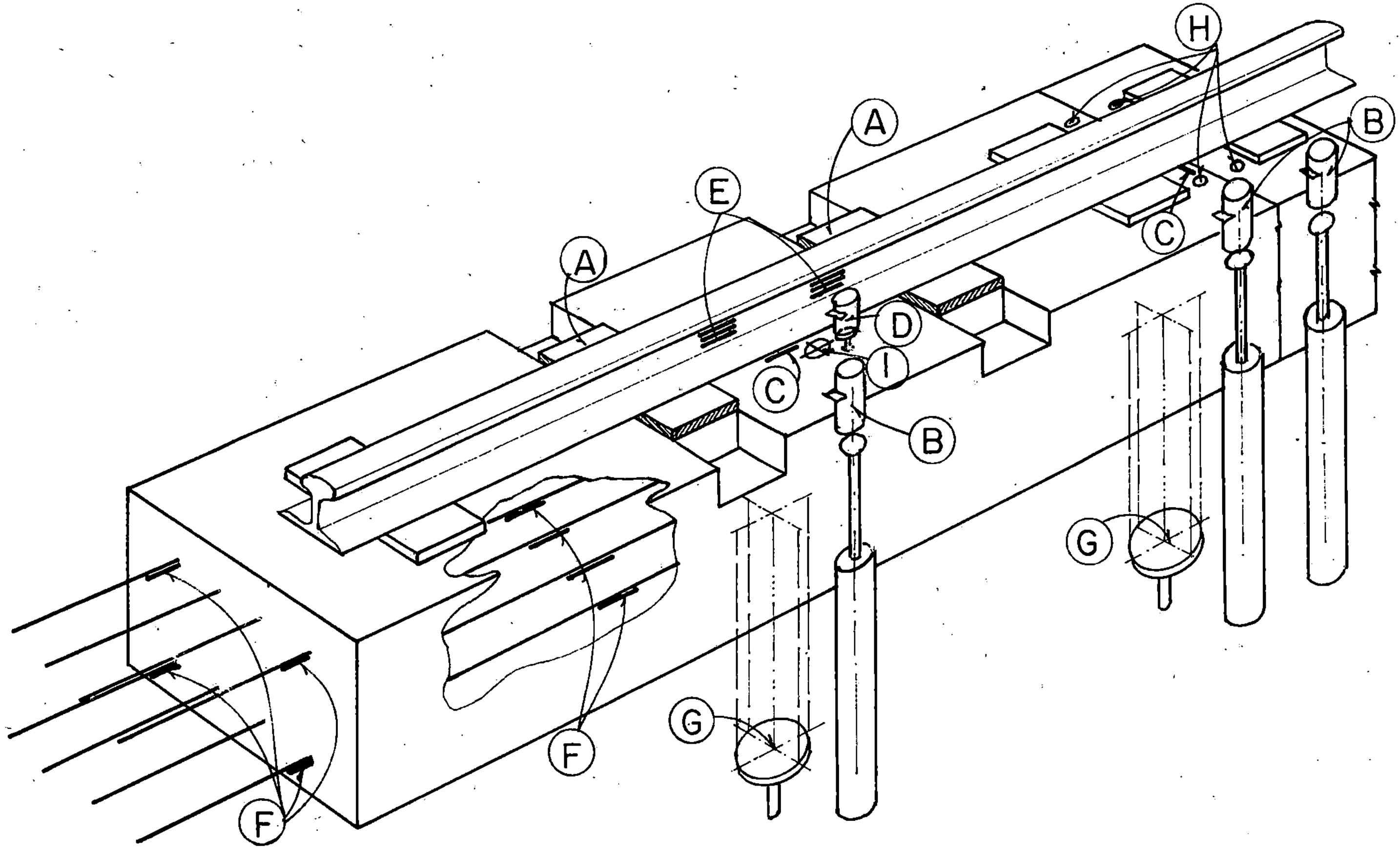
(F)-Rebar Strain Gages — top and bottom bars at midspan and joint under both rails.

(G)-Soil Pressure Cells — at midspan and joint under both rails.

(H)-Joint Width Plugs — both sides of slab.

(I)-Settlement Targets — both sides of slab.

FIGURE 9 - INSTRUMENTATION LAYOUT FOR SLAB TRACK SECTION



Ⓐ-Rail Load Sensors— both rails.

Ⓑ-Beam Deflection Meters— both sides of joint and midspan.

Ⓒ-Concrete Strain Gages— top, mid and bottom at midspan— top and bottom at joints.

Ⓓ-Rail Deflection— both rails.

Ⓔ-Rail Strain Gages—top, mid and bottom at both sides of both rails.

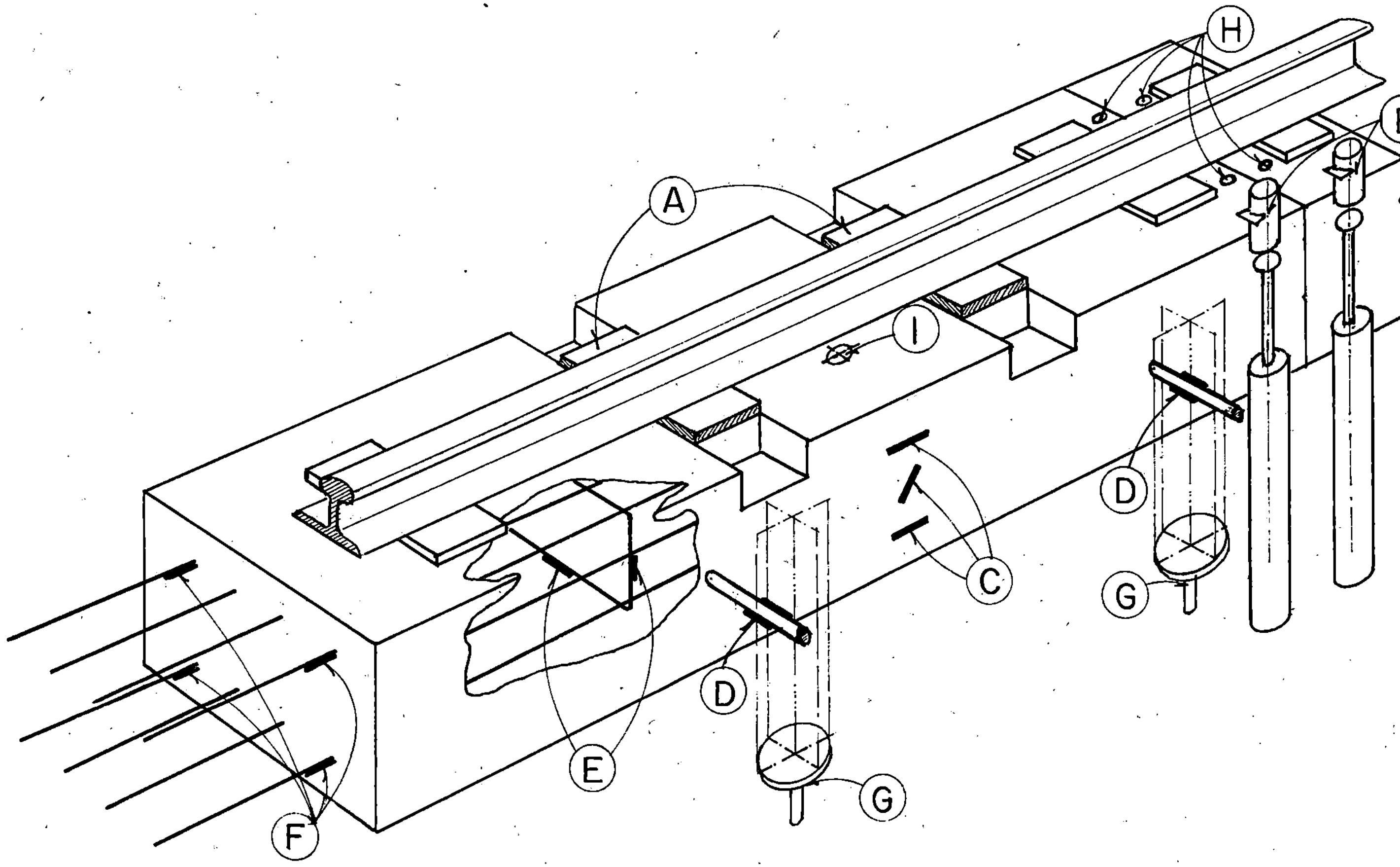
Ⓕ-Rebar Strain Gages—top and bottom bars at midspan and joint under both rails.

Ⓖ-Soil Pressure Cells— at midspan and joint under both rails.

Ⓗ-Joint Width Plugs— both sides of each beam.

Ⓘ-Settlement Targets — both sides of each beam.

FIGURE 10 - MAIN ARRAY INSTRUMENTATION FOR BEAM SECTIONS



Ⓐ Rail Load Sensors — both rails.

Ⓑ Beam Deflection Meters — both sides of joint.

Ⓒ Concrete Strain Gages — both sides of both beams.

Ⓓ Gage Bar Strain Gages.

Ⓔ Stirrup Strain Gages — midspan at two locations.

Ⓕ Rebar Strain Gages — top and bottom bars at midspan and joint under both rails.

Ⓖ Soil Pressure Cells — at midspan and joint under both rails.

Ⓗ Joint Width Plugs — both sides of each beam.

Ⓘ Settlement Targets — both sides of each beam.

FIGURE 11 - SECONDARY ARRAY INSTRUMENTATION FOR BEAM SECTIONS

Electrical resistance strain gages were used to measure strains on rails, reinforcement, and ties, and also internal and external concrete strains in slab and beam test sections.

Pressure cells were used to measure ballast pressure at the ballast-embankment interface. Cells were installed in groups of nine in the main array of cross tie track sections, and in groups of four in the main and secondary arrays of slab and beam track sections.

Resistance bridge-type deflectometers were used to measure dynamic deflections at main and secondary arrays in slab and beam track sections. Linear variable differential transformer (LVDT) deflectometers were used to measure dynamic deflections of ties and rails in cross tie track sections.

Piezoelectric-type accelerometers were employed for vertical and lateral acceleration measurements of both rails and at the ends, middle, and both rail seats of a cross tie at each main array. Accelerometer output was recorded on magnetic tape.

Load-cell ties and rail-pressure sensors were designed and fabricated at PCA. Four load-cell ties simulating the bending stiffness of concrete ties were used in the concrete tie track sections, one in each of Sections 1, 2, 3, and 8. Two load-cell ties simulating the bending stiffness of oak ties were used in wood tie track section, one in each of Sections 6 and 9.

Each load-cell tie contained a load cell at the rail seat to determine load transfer from the rails to the tie. In addition, the bottom surface of the ties was subdivided into ten sections, each consisted of a load cell, to measure pressure distribution at the tie-ballast interface.

Instrumented studs having top and bottom plates were originally proposed for use in the slab and beam track sections to measure load transfer between the rails and the structures at fastener locations. These sensors required 3-1/2 in. deep recesses cast in the top surface of the slabs or beams. As a maximum recess depth of 2 in. was specified, sensors were redesigned so that the top plates were supported on rollers. A fastening channel was bolted directly to the top plate that was gaged to measure bending strain. Also, the load sensors were instrumented to measure lateral forces.

Detailed information on instrumentation is given in Appendix A.

Appendix F discusses instrumentation and methods of data acquisition. Also, track conditions that may have affected performance, shortcomings of certain instrumentation, and recommendations to eliminate these shortcomings are outlined.

Test Procedures

Train loadings used for data collection and procedures for data acquisition and reduction are described.

Train Loading

As listed in Table 1, three types of train loadings were used for data collection. Loadings for initial test in April 1973 and Tests 1 and 2 in October-November 1974 were produced by test trains. Each train consisted of a locomotive, two hopper cars, and a caboose. A view of a test train is shown in Figure 12 and its details are illustrated in Figure 13. Data were obtained at creep speed for both the initial tests and Test 1, and at 30 mph for Test 2.

Quarterly data collection started with Test 3 in December 1974-January 1975. In addition to normal traffic, at least one preweighed train crossed the test track each day so as to monitor its effects on each test section.

A histogram of the wheel loads of all six trains used on tie track sections is shown in Figure 14. These data show that about 45% of the wheel loads were less than 12 kips, while the mean value was 15.7 kips. About 6% of the wheel loads were between 32 and 36 kips. The average locomotive wheel load was 33 kips.

Other trains used for data collection in Test 3 during December 1974-January 1975 and Test 4 in April 1975 were not preweighed. Locomotive numbers were recorded and wheel loads determined from ATSF locomotive description sheets. Wheel

TABLE 1 - TEST SCHEDULE AND TRAFFIC

Test No.	Date	Section No.	Traffic and Speed
Initial	April 73	All sections	Test train at creep speed
1	October 74 November 74	4,5,7 1,2,3,6,8,9	Test train at creep speed
2	October 74 November 74	4,5,7 1,2,3,6,8,9	Test train at 30 mph
3	December 74 January 75	4,5,7 1,2,3,6,8,9	One preweighed train/section plus normal traffic
4	April 75	All sections	Normal Traffic

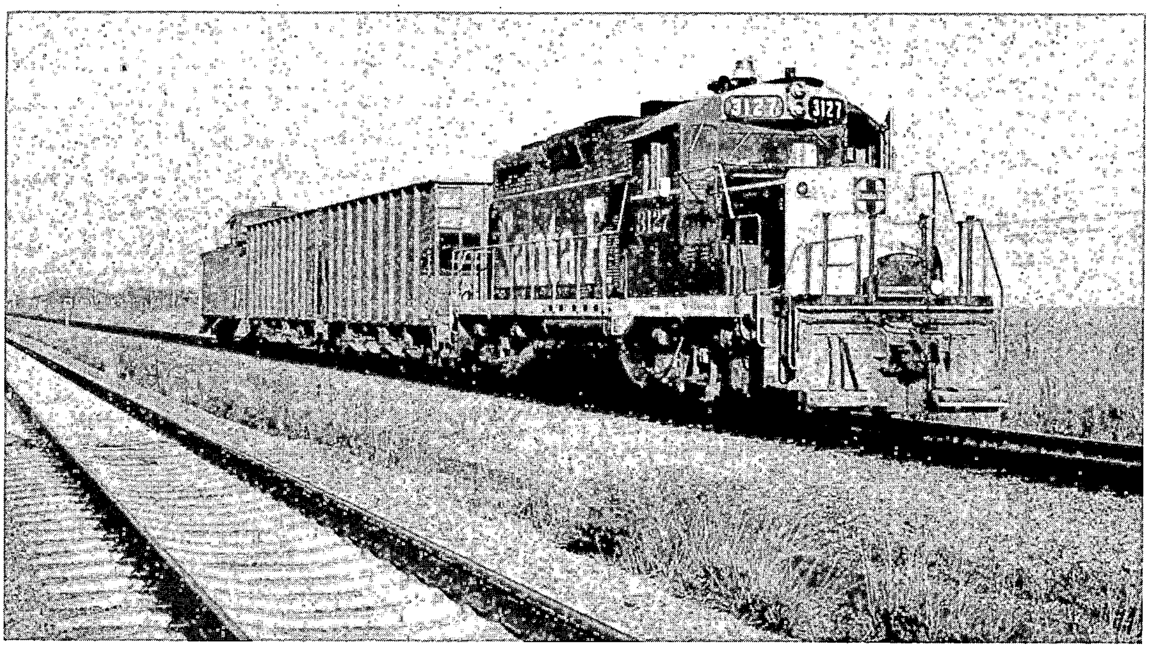


FIGURE 12 - TEST TRAIN USED IN TESTS 1 and 2

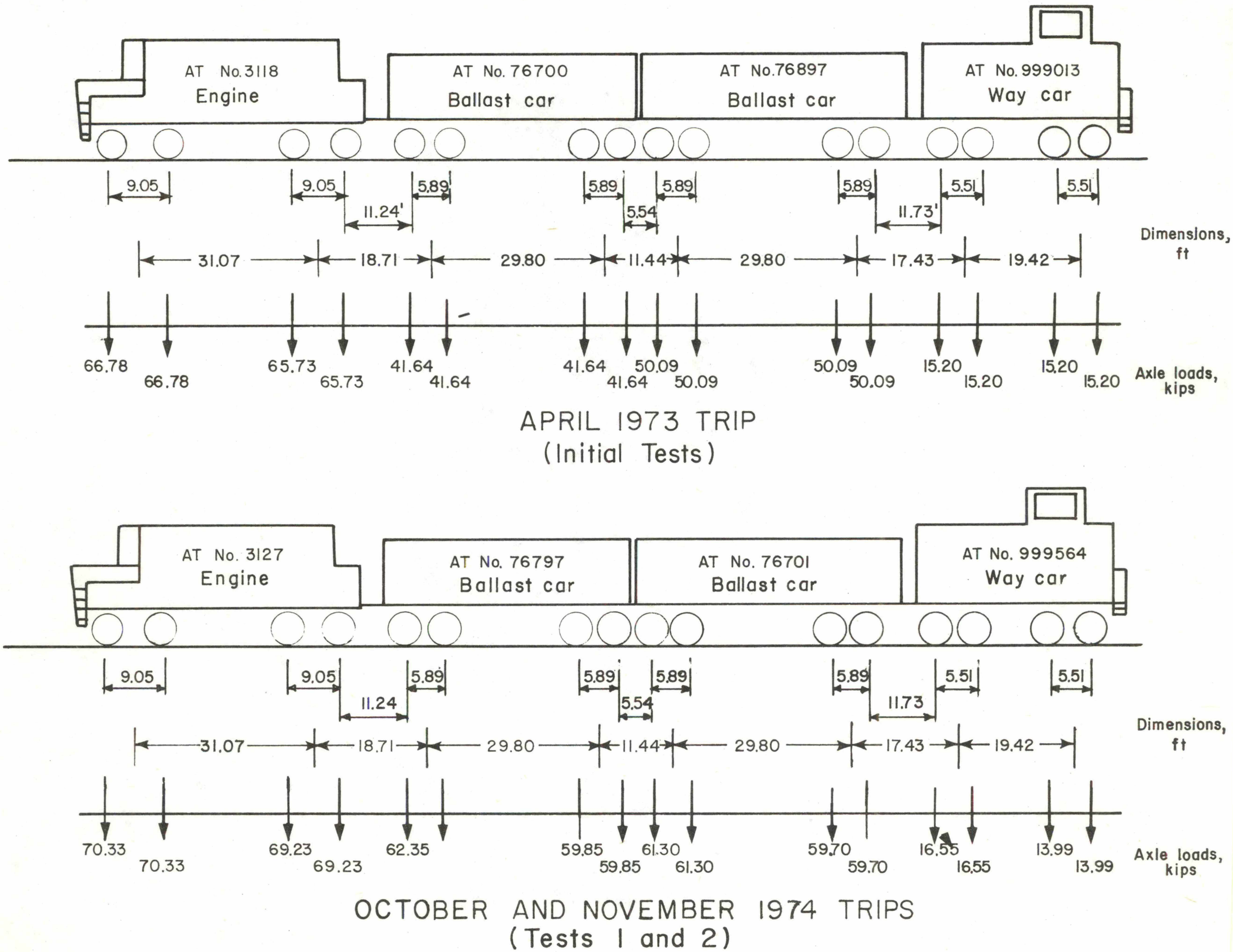


FIGURE 13 - DETAILS OF TEST TRAINS

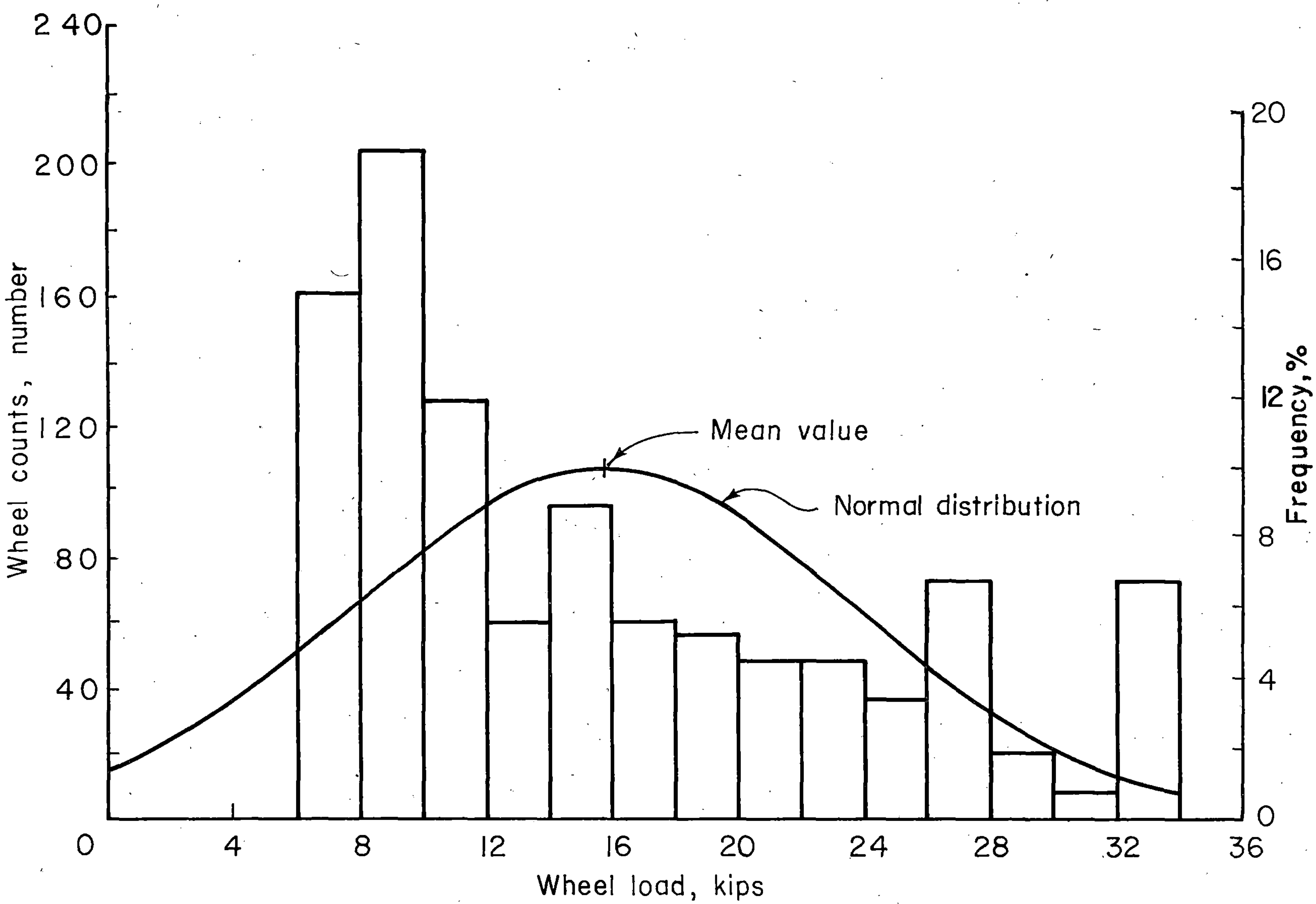


FIGURE 14 - HISTOGRAM OF WEIGHED TRAINS CROSSING TIE TRACK SECTIONS, JANUARY 1975

loads for these trains are presented in the bottom lines of data listings tabulated in Appendix C.

Data Acquisition

Three different train passages were recorded to complete the data acquisition for each test section. Preparation for data recording consisted of connecting cables to the proper sensors, balancing and calibration of recorder channels, and setting attenuation factors. Connecting cables and intermediate devices are described in Appendix F.

Oscillographic recording equipment was balanced by adjusting resistance and capacitance controls, so that the zero position of the pen was common for all attenuation settings of the carrier amplifier. Using a special "balance" configuration of the amplifier, each channel was balanced in 30 seconds.

Channels were calibrated by adjusting amplifier gain so that a calibration signal produced a known pen displacement on the recording chart. The attenuator was set at 1.0 for calibration. On most channels, the calibration signal was obtained from a one megaohm shunt resistance across one arm of the input bridge. On 12 channels, a gage factor dial was used to obtain the calibration signal. In either case, the resulting calibrated pen displacement was directly related to sensor units by sensitivity factors obtained from precalibration tests or by calculation. Sensitivity factors are discussed in Appendix A.

Attenuation of amplifiers was adjusted so that the recorded trace remained within the available chart width.

The passage of a wheel over a test section appears as a series of pen displacements on a moving strip of paper in the recorder. Normal paper speed was 25 mm per second. Conversion of pen displacement due to each wheel load into usable information such as strain, deflection, etc. was calculated from the following equation:

$$R = \frac{(A - B)CD}{E}$$

Where R = magnitude of data, e.g. strain, deflection, etc.

(A - B) = net pen displacement, mm

C = sensitivity factor

D = attenuation factor

E = calibration pen displacement, mm

Data were collected from embankment instruments for one of the three trains. Raw data traces together with calibration information was delivered to Shannon and Wilson, Inc. for interpretation.(2)

Constraints on Data Collection

The premature failure of the subgrade and subsequent termination of traffic over the test track severely limited the quantity and, in some cases, the quality of data collected. Cancellation of five data-acquisition periods prevented the taking of field measurements necessary to determine the effects of time and traffic on tests section performance. Also, measurements of the influence of high and low ties on load distribution and rail stress, scheduled after the fourth data-acquisition period, could not be taken. Subgrade softening and the resulting mud pumping created an environment detrimental to certain instrumentation. Some instruments were made inoperable by the unfavorable conditions while others produced extraneous data.

An inspection of the concrete tie test sections was conducted in November 1975. The inspection revealed that all ties had flexural cracking beneath the rails. No conclusive evidence is available to determine when these cracks had occurred.

Soil pressure cells were installed to measure distribution of ballast pressure at the ballast-embankment interface. Inspection of the instrumentation in November 1975 indicated that these cells were displaced vertically. For each 9-cell group located at the main array in each cross tie test section, the variation in elevation was as much as 4 in. For example, one cell in a group may have been covered with 6 in. of ballast while adjacent cells were covered with 8 or 10 in. of ballast.

Also, some cells were found to be tilted by as much as 15 degrees. This condition did not yield the desired pressure distribution data.

SUPPLEMENTAL INVESTIGATIONS

A few hours after opening the test track to traffic in May 1973, it was noted that anchor bolts of the rail fastening system used in the beam and slab sections started to pullout of the concrete. Therefore, an investigation was made to develop repair procedures and prepare installation specifications for a new anchorage system.

In addition, after termination of the project in June 1975, information was obtained on the condition and strength of the concrete and wood ties used in the test track. For this purpose laboratory tests were made on several ties.

Replacement of Fastening Anchorage System

After opening the test track to mainline traffic, an inspection on the first day indicated that anchor bolts of the rail fastener system used in the beam and slab sections had started to pullout of the concrete. After 28 hours of service, several anchor bolts had pulled out of the concrete. Therefore, traffic was suspended to analyze the problem and develop a solution.

Pullout tests were conducted on 4 anchor bolts in an unused panel stored near the test track. Test results shown in Table 2 indicate an average pullout force of 4,788 lbs. A typical pullout failure of an anchor bolts is shown in Figure 15.

Specially instrumented studs were fabricated to monitor track forces transmitted to anchor bolts. Four studs were calibrated and installed in a fastener in Section 7. Tests were conducted to determine the following:

1. Initial force exerted in anchor bolts during fastener installation and bolt tightening to the specified 450 ft-lb torque, and
2. Force exerted in anchor bolts due to a moving train.

Five test conditions were investigated to evaluate the effect of height variation between adjacent fasteners on anchor bolt forces. Results shown in Table 3 indicate that maximum

TABLE 2 - PULLOUT FORCE OF ORIGINAL KTT FASTENER ANCHOR BOLTS

Test No.	Pullout Load, lb
1	5,481
2	4,746
3	4,200
4	4,725
Average	4,788

FIGURE 15 - PULLOUT FAILURE OF FASTENING ANCHOR BOLT

TABLE 3 - FORCES TRANSMITTED TO FASTENER ANCHOR BOLTS

Test Sequence	Shim Condition	Average Force in Each Anchor Bolt, lb[(1)]		
		Due to Installation[(2)]	Due to Traffic[(3)]	Installation and traffic
1	As shimmed during construction	11,595	0 to -890	11,595 to 10,705
2	1/32" shim removed	11,780	25 to -900	11,805 to 10,880
3	3/32" shim removed	12,250	65 to -1,395	12,315 to 10,855
4	3/32" shim removed, and 1/16" shim removed from each of two fasteners on one side of instrumented fastener	13,530	300 to -1,415	13,830 to 12,115
5	3/32" shim removed, and 1/8" shim removed from each of two fasteners on one side of instrumented fastener[(4)]	11,565	450 to -1,235	12,015 to 10,330

(1) (-) indicates reduction in force.
(2) Anchor bolts were torqued to 450 ft-lb.
(3) Train moved across a fully torqued fastener at 30 mph.
(4) Fastener anchor system had failed on two fasteners on one side of the instrumented fastener. These fasteners provided no hold down force. The rail was allowed to float approximately 1/16" to 1/8" above its normally anchored position.

anchor-bolt forces occurred primarily during installation and were not significantly influenced by traffic. Maximum anchor-bolt forces ranged from 10.3 to 13.8 kips.

Anchor bolts were made with a shoulder support for the fastener. The shoulder was designed to prevent transmission of installation forces to the concrete. To prevent transmission, the fastener and necessary shims had to fit tight against the shoulder. This tight fit was virtually impossible to accomplish during installation. Therefore, forces were transmitted to the concrete anchoring the bolt. Since these forces created stresses that exceeded the anchorage pullout strength, pullouts had occurred. These pullouts had cracked both the concrete and the asbestos-cement bearing panels used in slab and beam test sections.

Therefore, it was concluded that a new anchoring and bearing system should be developed. PCA developed a system of securing the fastener to the concrete. This system involved (a) removal of the asbestos panels and filling their recesses with an epoxy concrete, and (b) replacement of the existing anchor bolts by new ones placed in holes drilled in the concrete and then filled with an adhesive.

Several adhesives, hole sizes, and hole preparation procedures were investigated and one was selected. The effects of method of cleaning the hole, curing temperature, and curing time on the load required to pullout a threaded rod embedded in the selected adhesive are presented in Table 4. The effect of curing temperature on adhesive setting time is shown in Figure 16. Figure 17 illustrates pull load versus slip relationship for the new anchor bolt system.

Procedures for properly locating and installing the new anchorage system were identified. For this purpose, special grooving and alignment equipment, illustrated in Figures 18 and 19 were designed and fabricated by PCA. Details and specifications for this work were presented in previous reports.[5,6]

TABLE 4 - PULLOUT LOAD FOR A ROD EMBEDDED IN ADHESIVE

Method of Hole Cleaning	Curing Temp., °F	Curing Time, hr.	Pullout Load, kip
Compressed Air*	72	24	32.0
Compressed Air*	72	48	36.0
Compressed Air*	72	72	36.0
Compressed Air*	72	120	32.0
Compressed Air*	135	48	42.0
None*	72	72	30.0
None**	72	72	15.0

*Dry hole
**Wet hole

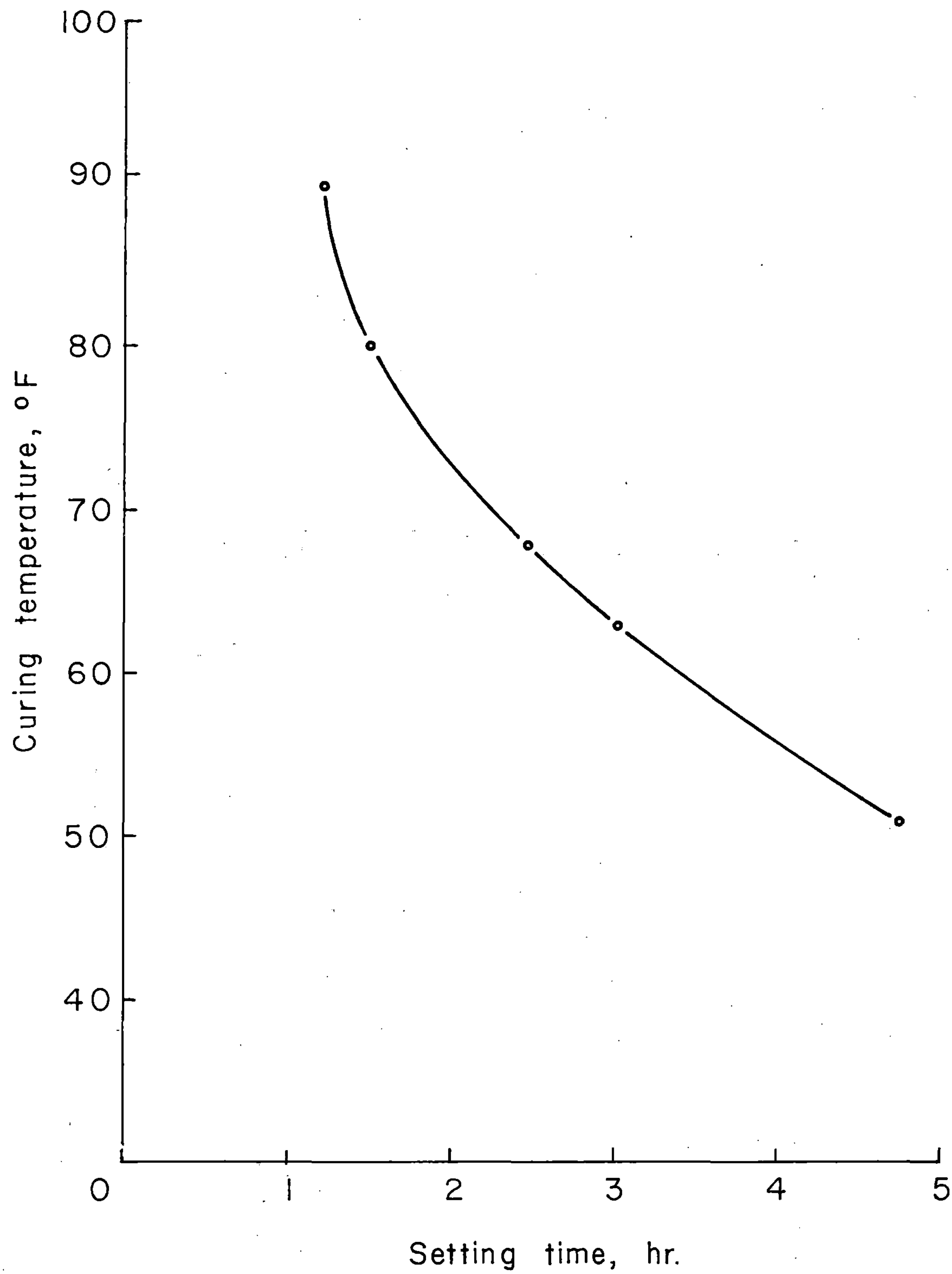


FIGURE 16 - EFFECT OF CURING TEMPERATURE ON SETTING TIME

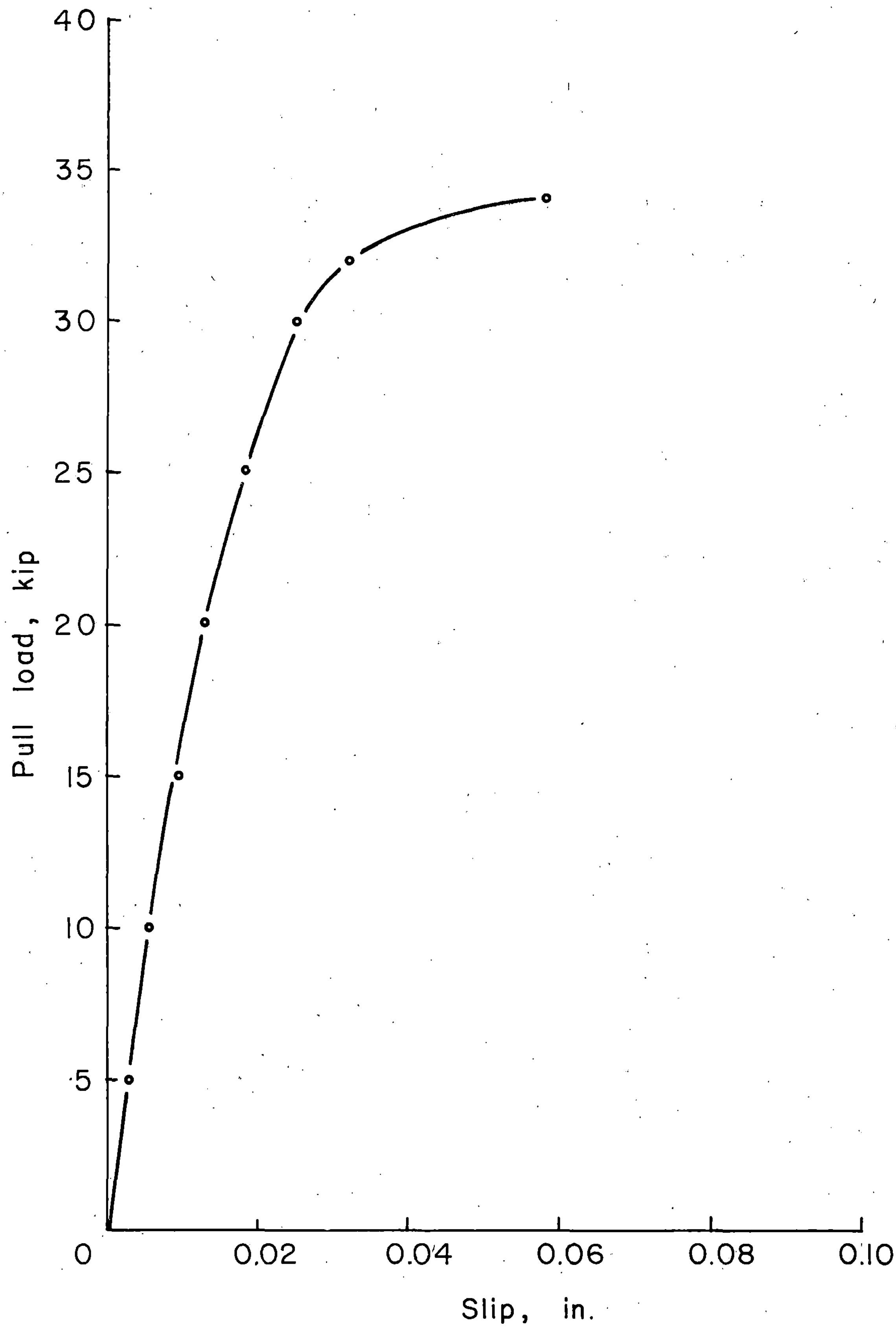


FIGURE 17 - PULL LOAD VERSUS SLIP RELATIONSHIP

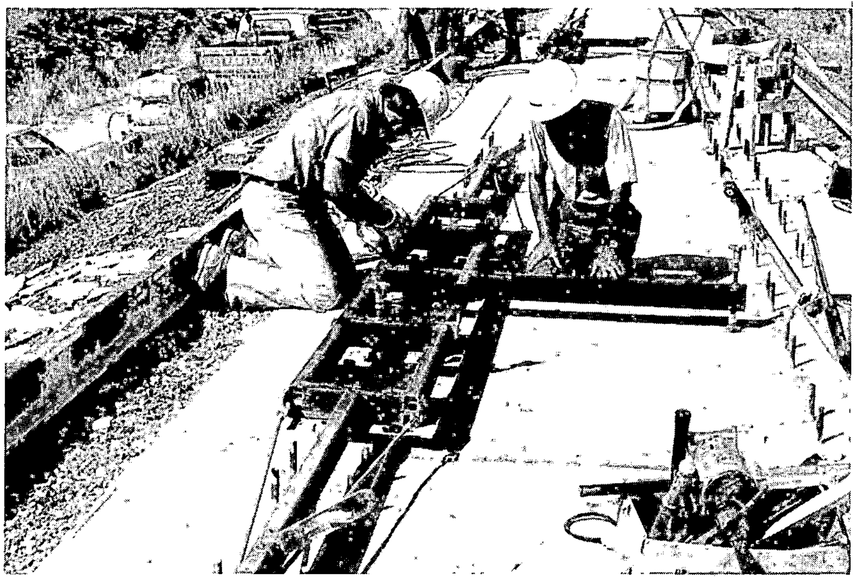


FIGURE 18 - GENERAL VIEW OF JIGS USED FOR LOCATING AND INSTALLING ANCHOR BOLTS

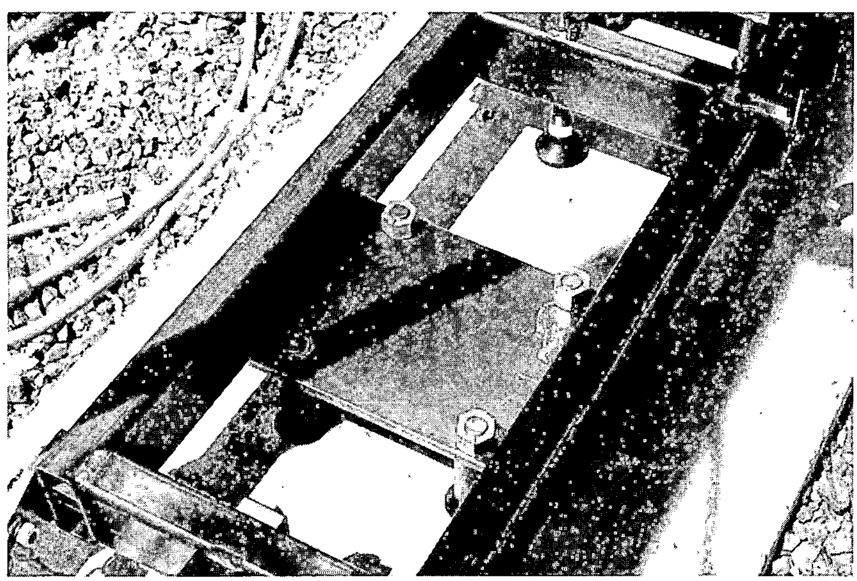


FIGURE 19 - CLOSE-UP OF JIG

Laboratory Tie Tests

When the test track was closed in June 1975, additional information was obtained on the strength properties of the concrete and wood ties used in the test sections. Laboratory tests were conducted on several ties to (a) determine the strength of uncracked concrete ties, (b) determine the effect of cracking on the ultimate strength of concrete ties, and (c) study the behavior of concrete and wood ties under load and ballast support conditions similar to those encountered in the test track.

Test ties included cracked concrete ties removed from Sections 1 and 8, unused concrete ties obtained from a stockpile, and an unused wooden tie similar to those used in Section 9. Tests included those specified by the American Railway Engineering Association (AREA)[7] and repeated load tests on ties supported on ballast. Type and number of tests are listed in Table 5.

AREA Tests

AREA tests were conducted on concrete ties in accordance with specified procedures. The ties were loaded in increments to obtain initial and structural cracking moments. Initial cracking is defined as "the first visible crack as seen through an illuminated 5-power mangifying glass."[7] Structural cracking is defined as "a crack that, when observed through illuminated 5-power magnifying glass, extends to the prestress layer nearest the tensile surface."[7]

Results of the rail seat vertical load and center bending tests, together with the AREA specificiation requirements, are summarized in Table 6. Strength requirements recommended by the 1975 AREA specifications[7] for ties spaced 24- and 30-in. on centers, and by the 1971 AREA specifications[8] for 30-in. spacing are listed. The 1971 specification was in effect when the ties were manufactured and installed in the test track.

Tests conducted on uncracked concrete ties indicated that the ties conformed to 1971 AREA bending moment requirements for

TABLE 5 - TYPE AND NUMBER OF CONCRETE TIE TESTS

Type of Tie	TYPE OF TEST						
	Ultimate Load	Rail Seat Vertical Load*	Tie Center Positive Bending Moment	Tie Center Negative Bending Moment	Rail Seat Repeated Load	Post-Fatigue Ultimate Load	Tie on Ballast
Concrete (unused)	2	4	2	2	1	1	2
Concrete (cracked, Section 1)	1				1	1	
Concrete (cracked, Section 8)	1				1	1	
Wood (unused)							1

*Each test consists of a positive and a negative test.

TABLE 6 - TIE BENDING MOMENTS

Type of Test	Specified AREA Moment Without Structural Cracking, in.-kip			Test Number	Test Results	
	1975 AREA, 24 in. Spacing	1975 AREA, 30 in. Spacing	1971 AREA, 30 in. Spacing		Moment At Initial Crack, in.-kip	Moment at Structural Crack in.-kip
Rail Seat Positive Moment	300	350	220	1	261	275
				2	213	241
				3	282	296
				4	282	303
				Avg.	260	278
Rail Seat Negative Moment	115	115	-	1	163	176
				2	176	195
				3	189	221
				4	221	234
				Avg.	187	206
Tie Center Positive Moment	105	125	135	1	122	135
				2	135	149
				Avg.	128	142
Tie Center Negative Moment	200	200	150	1	189	203
				2	189	203
				Avg.	189	203

30-in. spacing. Also, the ties conformed to 1975 AREA bending moment requirements for 30-in. spacing, except for positive bending moment at the rail seat. This value was 13 to 31% below the specified value.

Results of AREA bond development-ultimate load tests are summarized in Table 7. Ultimate loads obtained for cracked and uncracked ties were practically the same, the variation was 3 percent. Of the five ties tested, four indicated an ultimate strength exceeding 1971 AREA requirement for 30-in. spacing. The other tie indicated an ultimate strength 2% below the specified value. None of the ties met 1975 AREA requirement for 24- or 30-in. spacing.

Selected ties were instrumented with electrical resistance strain gages in the same manner as instrumented concrete ties used in the test track. Positive rail seat, bond development-ultimate load, negative rail seat, and center bending tests were conducted in accordance with AREA procedures. During the tests, load was applied in increments and strain recorded. Strain values obtained during the tests and corresponding bending moments are presented in Tables 8, 9, and 10.

AREA rail seat repeated-load tests were conducted on an unused tie, a tie removed from Section 1, and a tie removed from Section 8. AREA specifications require that test ties be structurally cracked at the beginning of the repeated-load test. Both ties from Sections 1 and 8 were structurally cracked when received. The unused tie was loaded statically until structural cracking occurred.

As discussed with and agreed upon by FRA, the load cycle applied to the ties varied between 4 and 48 kips. This loading range corresponds to the 1975 AREA requirement for a 9-ft long tie spaced at 24 in. Loading ranges corresponding to the 1971 and 1975 AREA requirements for 30-in. spacing are 4 to 35.2 and 4 to 56 kips, respectively.

The three ties were not affected appreciably by the 3 million load cycles. No progression of cracking was observed in the initially uncracked tie or the tie removed from Section 8.

TABLE 7 - BENDING MOMENTS FOR BOND DEVELOPMENT AND ULTIMATE LOAD

KTT Test Section	Tie Condition	Specified AREA Moment Without 0.001 in. Strand Slippage*, in.-kip			Test Results		
		1975 AREA, 24 in. Spacing	1975 AREA, 30 in. Spacing	1971 AREA, 30 in. Spacing	Structural Cracking Moment, in.-kip	Moment at 0.001 in. Strand Slippage, in.-kip	Ultimate Moment, in.-kip
Unused	Uncracked	525	613	330	275	385	447
Unused	Uncracked	525	613	385	241	378	461
1	Cracked	525+	613+	330+	-	481	481
8	Cracked	525+	613+	330+	-	447	481
8	Cracked	525+	613+	330+	-	454	461
Average					258	429	466

*Specification values are based on the structural cracking strength of the tie.

+Since these ties were already structurally cracked, the specification values are based on the average structural cracking strength of the two uncracked ties tested.

TABLE 8 - CONCRETE STRAINS FOR POSITIVE MOMENT RAIL SEAT AND BOND DEVELOPMENT-ULTIMATE LOAD TESTS

Applied Load, kip	Equivalent Moment, in.-kip	Concrete Strain, millionths*								
		Uncracked Tie			Cracked Tie from Section 1			Cracked Tie from Section 8		
		Top1)	Mid2)	Bot.3)	Top1)	Mid2)	Bot.3)	Top1)	Mid2)	Bot.3)
0	0	0	0	0	0	0	0	0	0	0
5	34	- 34	4	47	- 51	2	50	- 54	4	60
10	69	-107	7	110	-115	7	122	-109	7	123
15	103	-163	10	167	-168	14	194	-178	8	204
20	138	-224	13	228	-232	23	299	-248	12	314
25	172	-278	20	293	-307	48	441	-334	19	515
30	206	-339	26	373	-375	128	596	-446	52	962
35	241	-419	54	435	-482	326	951	-579	179	-
40	275	-547	477	-	-571	664	-	-715	511	-
45	309	-581	-	-	-595	1288	-	-850	974	-
50	344	-	-	-	-	-	-	-928	-	-
55	378	-	-	-	-	-	-	-976	-	-

*(-) Indicates compressive strain

1) Strain at the top surface of the tie
2) Strain at the neutral axis of the tie
3) Strain at the bottom surface of the tie

TABLE 9 - CONCRETE STRAINS FOR NEGATIVE MOMENT RAIL SEAT TESTS OF UNCRACKED TIES

Applied Load, kip	Equivalent Moment, in.-kip	Concrete Strain, millionths*		
		Top 1)	Mid 2)	Bot. 3)
0	0	0	0	0
5	33	37	1	- 39
10	65	85	1	- 90
15	98	141	1	-143
20	130	198	1	-195
25	163	256	2	-250
30	195	360	16	-309
35	228	-	81	-431

*(-) Indicates compressive strain

1) Strain at the top surface of the tie
2) Strain at the neutral axis of the tie
3) Strain at the bottom surface of the tie

TABLE 10 - CONCRETE STRAINS FOR CENTER BENDING TESTS OF UNCRACKED TIES

Applied Load, kip	Equivalent Moment, in.-kip	Concrete Strain, millionths*					
		Postive Bending			Negative Bending		
		Top1)	Mid2)	Bot.3)	Top1)	Mid2)	Bot.3)
0	0	0	0	0	0	0	0
2	27	- 71	8	89	75	-14	- 93
4	54	-144	18	182	147	-25	-180
5	68	-180	23	222	-	-	-
6	81	-222	29	282	228	-35	-275
7	95	-254	33	324	-	-	-
8	108	-297	40	392	301	-42	-368
9	122	-353	58	513	-	-	-
10	135	-420	120	525	401	-49	-462
11	149	-483	210	-	450	-50	-512
12	162	-	-	-	519	-50	-573
13	176	-	-	-	575	-42	-638
14	189	-	-	-	-	-11	-728
15	203	-	-	-	-	99	-908

*(-) Indicates compressive strain

1) Strain at the top surface of the tie
2) Strain at the neutral axis of the tie
3) Strain at the bottom surface of the tie

However, crack length in the tie removed from Section 1 increased about 1-1/4 in. Thus, all three ties met 1975 AREA rail seat repeated load test requirement for 24-in. spacing and also met 1971 AREA requirement for 30-in. spacing.

After completion of 3 million load cycles, static tests were made to determine the post-fatigue ultimate strengths of the ties. Tests data indicated ultimate loads 10 to 28 percent above those obtained from bond development-ultimate load tests.

Tests of Ties on Ballast

Three tie tests on ballast were conducted to evaluate the behavior of concrete and wood ties under load conditions similar to those encountered in the Kansas Test Track. These included two concrete ties and a wood tie. In the tests, the tie and ballast was representative of a section of the track with full transverse width including ballast shoulders and edge slope. One concrete and one wood tie was tested on slag ballast. The second concrete tie was tested on granite ballast. A general view of the test setup is shown in Figure 20. Type of tie, thickness and type of ballast, together with load magnitude and number of cycles are presented in Table 11.

Repeated loads were applied to the rails directly over the tie rail seats. Load cycles were selected to simulate 55-ft long cars moving across the tie at a speed of approximately 55 mph. Each load cycle ranged from a minimum of 500 lb to the maximum load specified in Table 11. A total of 8.25 million load cycles were applied. The initial 250,000 load cycles were used to seat the tie in the ballast.

Electrical resistance strain gages were applied to the ties in the same manner and locations as for instrumented ties in the test track. Strains and deflections of the tie at the rail seat were measured and recorded periodically during the test. Strain and deflection data are presented in Tables 12 and 13 for the concrete tie on slag and on granite ballast, respectively, and in Table 14 for the wood tie.

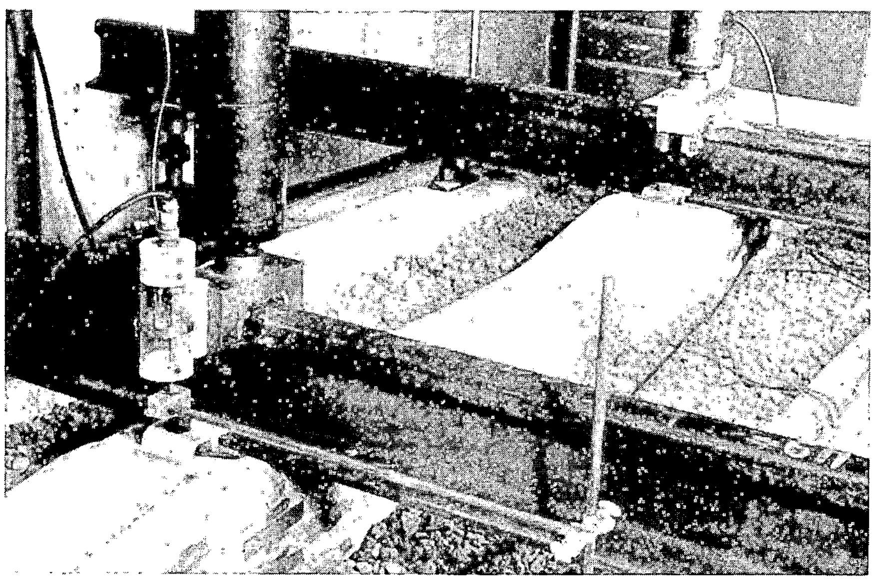


FIGURE 20 - GENERAL VIEW OF TIE TEST ON BALLAST

TABLE 11 - LOAD CYCLES FOR TIE TESTS ON BALLAST

Type of Tie	Thickness and Type of Ballast	Number of Cycles, millions				
		Rail Seat Vertical Load, kips				
		26	30	33	36	39
Concrete (uncracked)	15" slag	0.25	2.0	2.0*	2.0	2.0
Concrete (uncracked)	12" granite	0.25	2.0	2.0	2.0	2.0
Wood (unused)	12" slag	0.25	2.0	2.0	2.0	2.0

*Plus a 0.5 to 10 kip lateral load cycle for 1.7 million cycles.

TABLE 12 - STRAIN AND DEFLECTION DATA FOR CONCRETE TIE TEST ON 15-in. SLAG BALLAST

Rail Seat Load, Kip	Load Cycles, Millions	Strain, millionths*						Deflection, in.	
		East Rail Seat		West Rail Seat		Tie Center		Rail Seats	
		Top[1]	Bot.[2]	Top[1]	Bot.[2]	Top[1]	Bot.[2]	East	West
26	0.00	-115	168	-158	230	104	- 92	0.038	0.035
	0.25	-	171	-	206	69	- 95	0.038	0.053
30	0.00	-	203	-	237	65	- 74	0.034	0.048
	0.30	-	168	-	197	177	-209	0.035	0.050
	0.30[3]	-	129	-	198	218	-206	-	0.060
	2.00[3]	-	131	-	171	198	-182	0.054	0.070
33	0.00	-125	204	-138	74	149	-147	0.033	0.068
	1.00[4]	-185	243	-147	213	-	-308	0.070	0.050
	1.03[5]	-195	248	-	-	227	-342	0.040	0.055
	2.00	-201	out	-615	395	158	-170	0.040	0.040
36	0.00	- 89	-	-293	164	368	-342	0.045	0.055
	1.00	-198	-	-657	368	93	-114	0.039	0.049
	2.00	-192	-	-608	362	75	-108	0.040	0.070
39	0.00	-215	-	-645	411	89	- 78	0.041	0.073
	1.50	-213	-	-717	-	86	-119	0.038	0.073
	2.00	-224	-	-515	-	110	-171	0.059	0.058

*(-) Indicates compressive strain
1) Strain at top surface of tie
2) Strain at bottom surface of tie
3) A 0.5 to 10 kip lateral load was also applied to the east rail seat.
4) Tie was center bound.
5) Ballast was adjusted to alleviate center bound condition.

TABLE 13 - STRAIN AND DEFLECTION DATA FOR CONCRETE TIE TEST ON 12-in. GRANITE BALLAST

Rail Seat Load, Kip	Load Cycles, Millions	Strain, millionths*						Deflection, in.	
		East Rail Seat		West Rail Seat		Tie Center		Rail Seats	
		Top 1)	Bot. 2)	Top 1)	Bot. 2)	Top 1)	Bot. 2)	East	West
26	0.00	-120	189	-116	203	171	-164	0.096	0.073
	0.25	-117	194	-143	212	156	-174	0.071	0.066
30	0.00	-147	224	-188	245	140	-170	0.043	0.050
	1.00	-156	221	-108	240	207	-227	0.048	0.033
	2.00	-159	234	-171	252	131	-147	0.060	0.040
33	0.00	-173	251	-187	272	134	-156	0.058	0.033
	1.00	-179	249	-183	287	287	-141	0.046	0.038
	2.00	-182	249	-239	285	183	-227	0.048	0.065
36	0.00	-198	270	-254	306	179	-231	0.051	0.068
	1.00	-194	267	-329	282	203	-255	0.054	0.054
	2.00	-189	2.7	-242	284	180	-224	0.056	0.080
39	0.00	-203	296	-254	305	186	-234	0.060	0.083
	1.00	-224	308	-252	327	191	-239	0.051	0.071
	2.00	-204	297	-204	297	209	-269	0.048	0.085

1) Strain at top edge of tie.
2) Strain at bottom edge of tie.

TABLE 14 - STRAIN AND DEFLECTION DATA FOR WOODEN TIE TEST ON 12-in. SLAG BALLAST

Rail Seat Load, Kip	Load Cycles, Millions	Strain, millionths*						Deflection, in.	
		East Rail Seat		West Rail Seat		Tie Center		Rail Seats	
		Top[1)]	Bot.[2)]	Top[1)]	Bot.[2)]	Top[1)]	Bot.[2)]	East	West
26	0.00	239	560	104	241	241	386	0.053	0.083
	0.25	329	532	155	277	330	347	0.053	0.085
30	0.00	386	573	155	315	343	351	0.063	0.090
	1.00	633	601	458	417	463	378	0.065	0.085
	2.00	767	615	424	417	319	279	0.056	0.056
33	0.00	818	650	413	437	337	295	0.059	0.058
	1.05	819	655	602	587	440	365	0.071	0.078
	2.29[3)]	482	566	480	440	680	504	0.078	0.135
	2.29[4)]	333	630	812	564	88	62	0.080	0.093
36	0.00	314	575	869	595	88	80	0.090	0.100
	1.00	228	570	435	669	452	294	0.088	0.083
	2.00	out	486	620	659	510	389	0.117	0.095
39	0.00	out	925	598	669	540	398	0.113	0.100
	1.02	out	875	538	605	554	412	0.094	0.085
	2.08	out	787	546	638	638	447	0.082	0.102

*(-) Indicates compressive strain
1) Strain at top surface of tie
2) Strain at bottom surface of tie
3) Tie was center bound.
4) Ballast adjusted to alleviate center bound condition.

Inspection of ties during and after tests indicated the following:

1. No vertical cracks occurred in the concrete ties. However, small longitudinal cracks developed on the surfaces along the tie length.
2. Wood tie fibers became damaged under one tie plate. The damage occurred in the upper 1 in. of the tie and was observed after 6 million load cycles. In addition, a crack extending diagonally between two spike holes occurred in a tie plate.

A center-bound condition developed during testing of both the concrete tie and the wood tie on slag ballast. However, this condition did not develop during testing of the concrete tie on granite ballast.

A comparison of measured tensile strains in the concrete ties to those obtained from static tests indicate that the maximum rail seat bending moments were 186 and 233 in.-kip for the concrete tie on granite and slag ballast, respectively. The maximum center bending moments were 103 and 125 in.-kip for the concrete tie on granite and slag ballast, respectively. These bending moments are lower than structural cracking moments determined from static tests.

TEST RESULTS - TIE SECTIONS

Data obtained for cross tie sections during three acquisition trips are tabulated in Appendix C. The first trip, made in October-November 1974, produced data for creep speed and 30 mph. These data are labeled Tests 1 and 2, respectively. The second trip, made in January 1975, produced data for 30 mph and is labeled Test 3. The third trip, made in April 1975, produced data for 50 mph and is labeled Test 4.

A weighed train passing each test section during Test 3 provided data for a wide range of wheel loads. Two examples of data obtained for weighed wheel loads of a single train are shown in Figures 21 and 22. Wheel load was calculated on the assumption that each car load was equally distributed between all wheels.

Rail strains obtained from the bottom gage on the north rail located above the instrumented tie at the main array are shown in Figure 21. Rail seat loads measured for the north rail seat of the load cell tie are shown in Figure 22. These data indicate a nearly linear relationship from no effect for no load to a maximum effect for locomotive wheel loads. Therefore, data trends were assumed to be linear and to pass through the origin. Slopes of the trend lines were determined using the origin and locomotive data only. Discussions of data will be based on the trend line slope that expresses the relationship between wheel load and the discussed effect.

Deflection and Track Modulus

Rail deflection between ties, and tie-end deflection were measured at main arrays. These deflections, expressed in inches per kip of wheel load, are presented in Table 15 for the different test sections and acquisition trips.

Track modulus was calculated from these data using the following formula:[9]

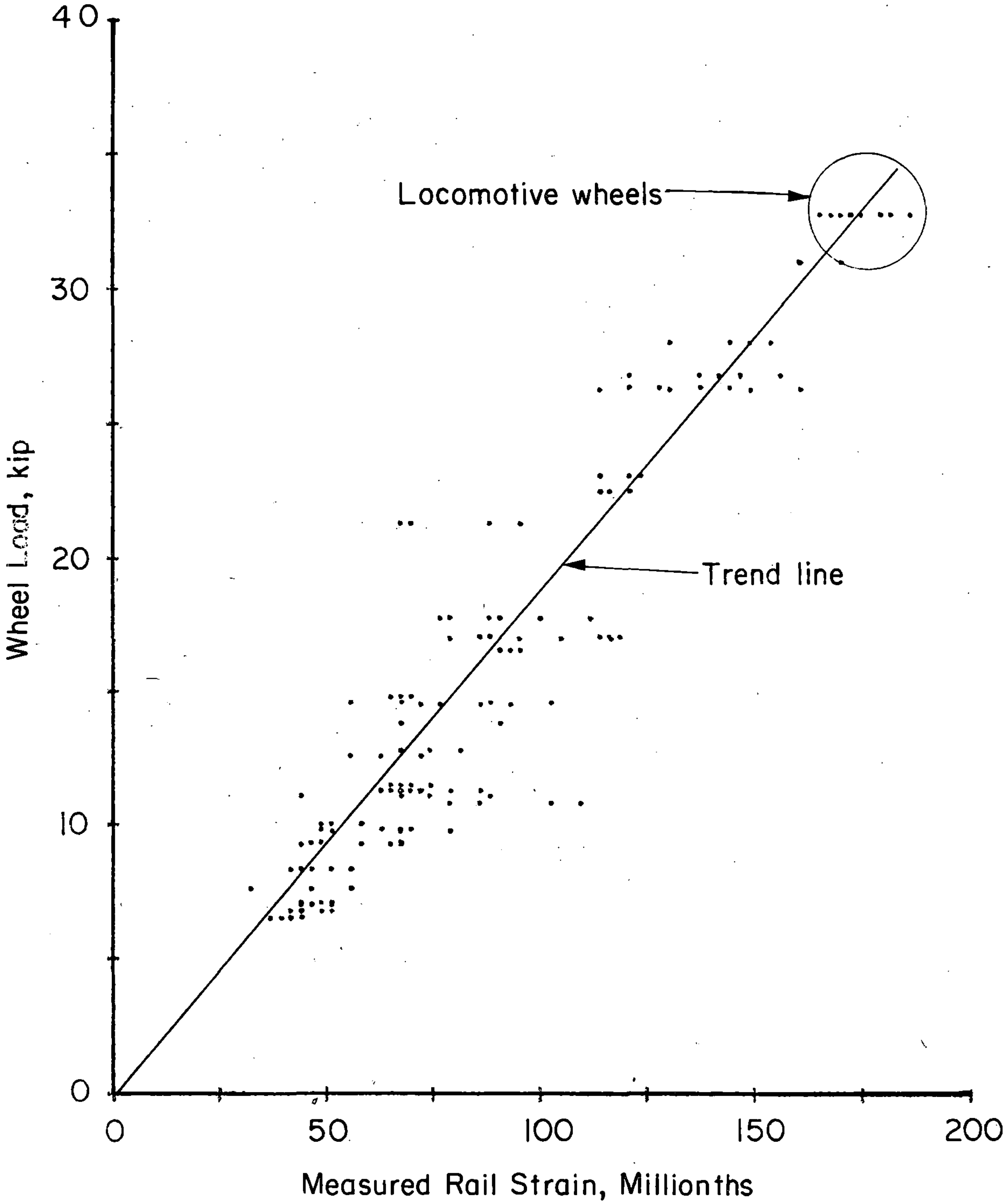


FIGURE 21 - RAIL STRAIN VERSUS WHEEL LOAD

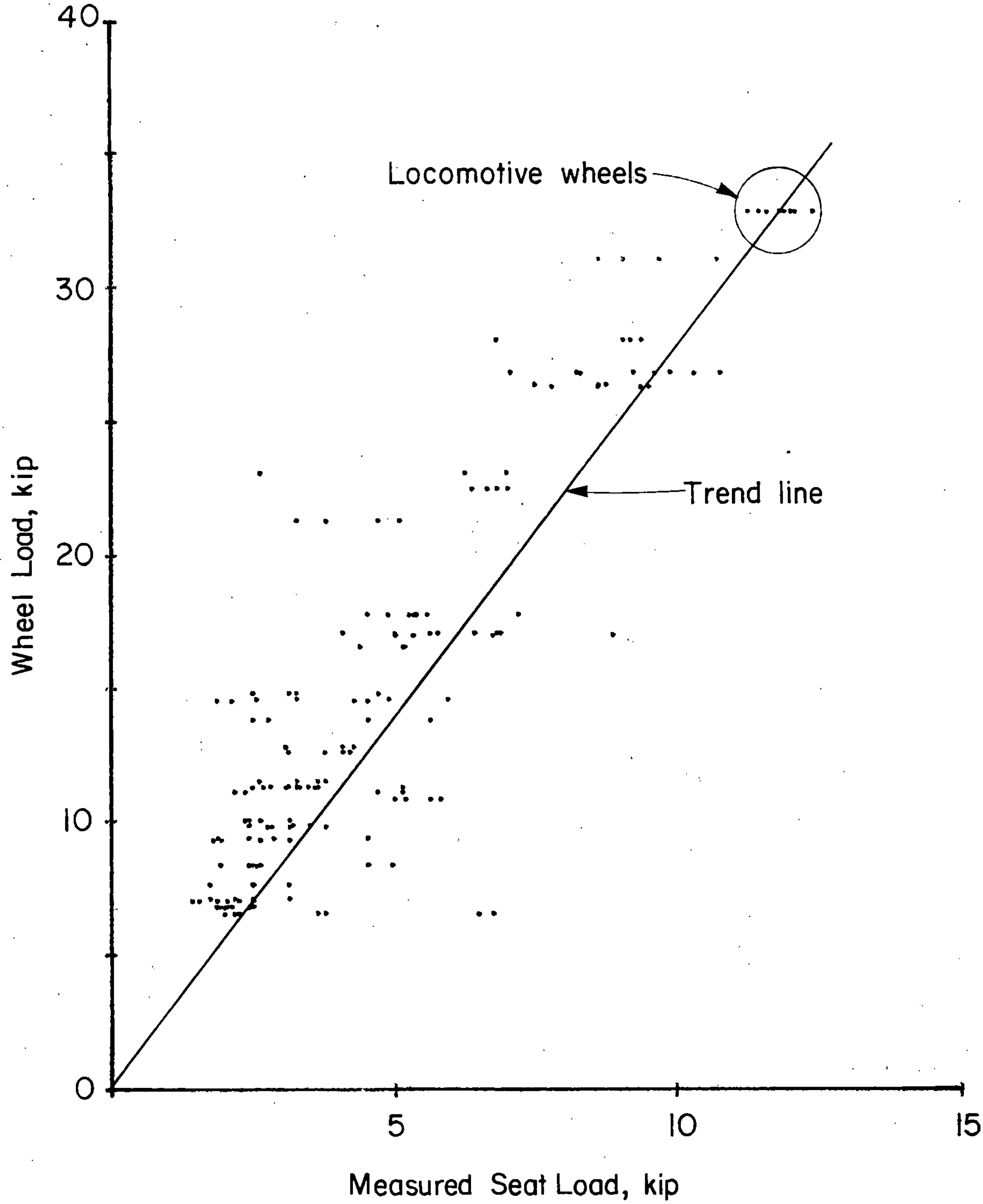


FIGURE 22 - RAIL SEAT LOAD VERSUS WHEEL LOAD

TABLE 15 - DEFLECTION DUE TO WHEEL LOAD

Test Section	Test	Deflection, in./kip of Wheel Load					
		Tie End			Rail Between Ties		
		Maximum	Minimum	Average	Maximum	Minimum	Average
1	1	0.0090	0.0076	0.0079	-	-	-
	2	-	-	-	-	-	-
	3	0.0044	0.0035	0.0041	0.0023*	0.0022*	0.0023*
	4	0.0080	0.0067	0.0071	0.0057	0.0043	0.0051
2	1	0.0069	0.0055	0.0059	0.0056	0.0045	0.0048
	2	0.0072	0.0057	0.0060	0.0059	0.0046	0.0049
	3	0.0032	0.0028	0.0030	-	-	-
	4	-	-	-	0.0038	0.0033	0.0036
3	1	0.0056	0.0045	0.0047	0.0047	0.0038	0.0040
	2	0.0055	0.0042	0.0045	0.0047	0.0036	0.0039
	3	0.0025*	0.0018*	0.0023*	0.0026*	0.0018*	0.0022*
	4	-	-	-	0.0073	0.0062	0.0069
6	1	-	-	-	0.0193	0.0156	0.0163
	2	-	-	-	0.0196	0.0154	0.0166
	3	0.0053	0.0046	0.0050	0.0068	0.0060	0.0064
	4	-	-	-	0.0143	0.0115	0.0128
8	1	0.0071	0.0059	0.0062	0.0060	0.0050	0.0053
	2	0.0074	0.0054	0.0060	0.0065	0.0048	0.0053
	3	0.0041	0.0034	0.0037	0.0040	0.0036	0.0038
	4	0.0033	0.0027	0.0031	0.0041	0.0033	0.0038
9	1	0.0071	0.0059	0.0062	0.0079	0.0065	0.0068
	2	0.0070	0.0056	0.0059	0.0081	0.0064	0.0068
	3	-	-	-	-	-	-
	4	-	-	-	0.0189	0.0182	0.0185

*Low value, probably inaccurate
- Data not obtained

$$k = \sqrt[3]{\frac{P^4}{64EI_y z^4}}$$

where k = track modulus, lb/in./in.

P = wheel load, lb

E = modulus of elasticity of rail steel, psi

I_y = moment of inertia of rail cross section with respect to neutral axis, in.^{4}

z = rail deflection at load location, in.

To calculate track modulus and to correct for a suspended track condition, deflection due to light wheel loads was subtracted from that measured under heavy wheel loads. Deflection for a locomotive wheel was calculated by multiplying the average values shown in Table 15 by 33 kip. Deflection under light loads was recorded for a caboose wheel whose load was assumed at 7.6 kips. These values and the corresponding calculated track modulus are shown in Table 16.

The simplified equation used for calculating track modulus is based on the application of a single wheel load. However, deflection values substituted in the equation include the effect of adjacent wheels. Therefore, track modulus values shown in Table 16 could contain a maximum error of approximately 5%.

Table 16 indicates that except for the January 1975 data, Test 3, track modulus remained practically unchanged in Sections 2, 6, 8, and 9, and slightly changed in Section 3. Large increases in track modulus were recorded in January 1975, Test 3. This increase might be attributed to ballast and subgrade freezing at that time. For Test 4, the largest track modulus was calculated for Section 2 with concrete ties spaced at 27 in. The smallest calculated track modulus for the same test was for Section 6, timber ties on stabilized ballast.

Distribution of Wheel Load to Ties

Load sensors under rail seat plates of the load cell tie at the main array indicated that a measurable force is transferred to the tie from the rail. This force, expressed as a percentage

TABLE 16 - RAIL DEFLECTIONS AND TRACK MODULUS

Test Section	Test	Measured Average Deflection, in.			Calculated Track Modulus lb./in./in.
		33 kip Wheel	7.6 kip Caboose Wheel	Net	
1	1	-	-	-	-
	2	-	-	-	-
	3	0.076*	0.064	0.012	47960**
	4	0.168	0.100	0.068	4760
2	1	0.158	0.111	0.047	7780
	2	0.162	0.116	0.046	8010
	3	-	-	-	-
	4	0.119	0.076	0.043	8760
3	1	0.132	0.074	0.058	5880
	2	0.129	0.082	0.047	7780
	3	0.073*	0.043	0.030	14150**
	4	0.228	0.134	0.094	3090
6	1	0.538	0.308	0.230	940
	2	0.548	0.316	0.232	930
	3	0.211	0.151	0.060	5620
	4	0.422	0.216	0.206	1090
8	1	0.175	0.106	0.069	4670
	2	0.175	0.106	0.069	4670
	3	0.125	0.076	0.041	9340
	4	0.125	0.057	0.068	4760
9	1	0.224	0.125	0.099	2890
	2	0.224	0.125	0.099	2890
	3	-	-	-	-
	4	0.611	0.520	0.091	3230

*Low value, probably inaccurate
**High value, probably inaccurate
- Data not obtained

of the corresponding wheel load, is shown in Table 17. Average values for data obtained at 30 mph are plotted in Figure 23.

A comparison of rail seat load data for Sections 2 and 8 indicate that in general ballast depth had only a slight effect on rail seat load.

Effect of concrete tie spacing on rail seat load is shown in Figure 24. These data indicate that rail seat load generally increased as tie spacing increased. The increase of tie spacing from 24 in. to 30 in. increased rail seat load by up to 45%.

Also, Figures 23 and 24 show rail seat force values calculated from average track modulus data. Rail seat force was calculated from the following formula:(9)

$$R = \frac{Pa}{2} \sqrt[4]{\frac{k}{4EI_y}}$$

where
- R = rail seat force, lb
- P = wheel load, lb
- a = tie spacing, in.
- k = calculated track modulus, lb/in./in.
- E = modulus of elasticity of steel, psi
- I_y = moment of inertia of rail cross section, $in.^4$

As Figure 24 indicates, reasonable agreement exists between data from rail seat measurements and those calculated from track modulus. Thus, the equations used in this analysis seem applicable for prediction of rail seat load.

Distribution of Tie Load to Ballast

In the main array, sensors at the bottom of the load cell tie measured the force between tie and ballast at 10 locations along the tie length. Average force, expressed as a percentage of the corresponding axle load, is presented in Table 18. Load distribution over the length of the tie is shown in Figure 25. Unknown data were replaced by data from the symmetrical base pad, while others were replaced by calculated values so that the sum of the base-pad forces equals the sum of the rail-seat forces.

TABLE 17 - RAIL SEAT FORCE AT TIE

Test Section	Test-Run	Tie-Rail Seat Force, Percent of Wheel Load					
		North Rail Seat			South Rail Seat		
		Max.	Min.	Ave.	Max.	Min.	Ave.
1	1-1	54	41	44	58	41	49
	2-1	65	44	49	-	-	-
	3-1	63	53	57	76	61	69
	4-1	94	46	62	65	48	56
	4-2	70	44	59	65	44	55
2	1-1	26	19	21	58	42	49
	2-1	30	19	21	42	32	36
	3-1	34	24	29	57	43	51
	3-2	31	21	27	59	46	52
	4-1	56	37	45	57	39	49
	4-2	55	44	47	56	45	52
3	1-1	46	36	39	39	29	31
	2-1	57	40	45	40	29	32
	3-1	42	39	41	48	43	45
	3-2	46	37	42	52	43	48
	4-1	62	41	53	45	35	40
	4-2	56	39	48	45	38	42
6	1-1	31	20	22	-	-	-
	2-1	39	19	25	-	-	-
	3-1	41	35	38	40	34	37
	3-2	51	36	40	42	34	38
	4-1	-	-	-	35	17	27
8	1-1	35	26	28	25	14	16
	2-1	45	32	35	-	-	-
	3-1	28	25	27	43	38	41
	3-2	29	20	24	49	35	43
	4-1	53	38	45	56	38	46
	4-2	54	46	49	51	36	42
9	1-1	36	25	28	35	24	26
	2-1	43	25	29	37	23	26
	3-1	44	30	36	36	26	32
	3-2	43	29	36	50	30	40
	4-1	48	31	37	45	30	37

- Data not obtained

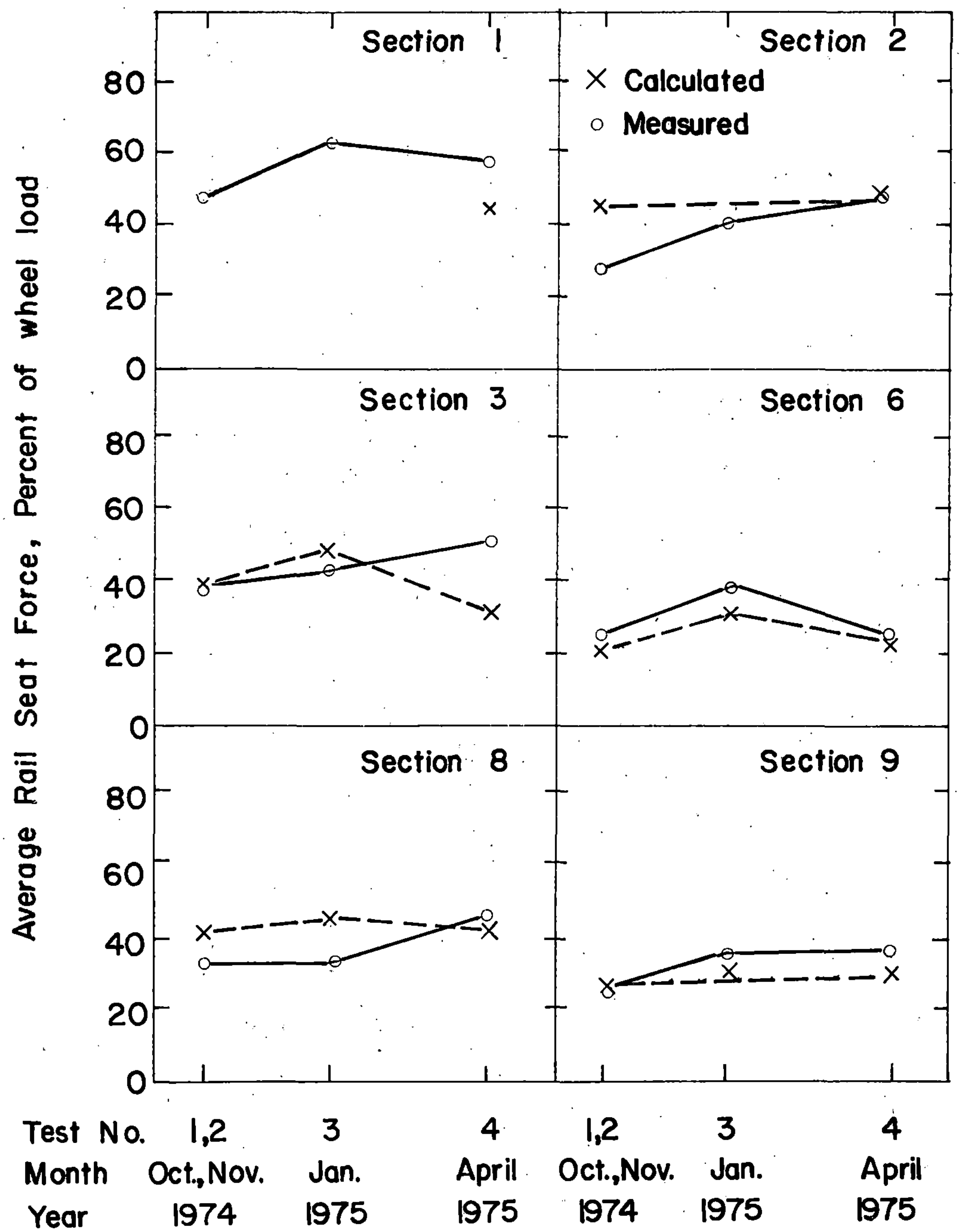


FIGURE 23 - AVERAGE FORCE INTO TIE FROM RAIL

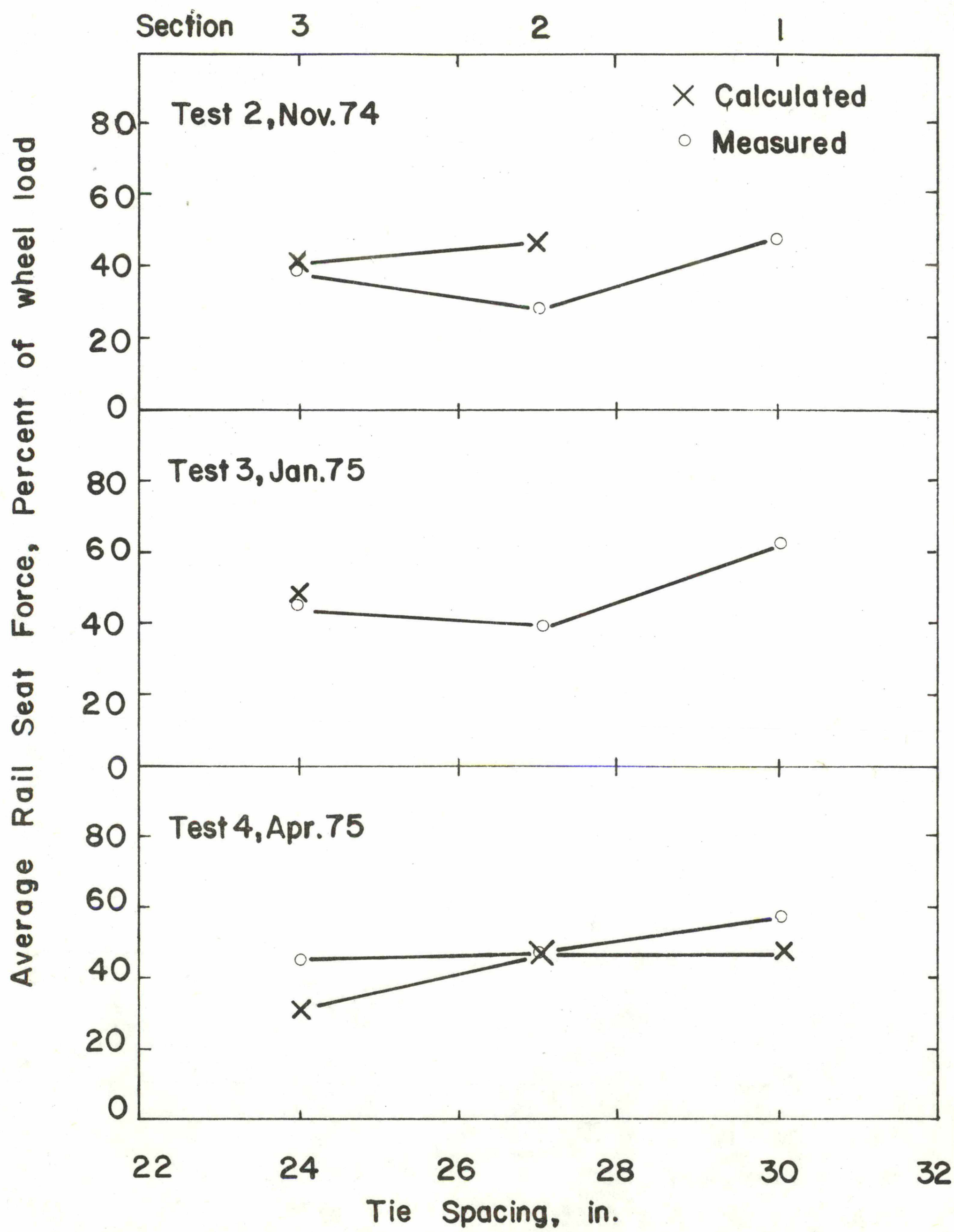


FIGURE 24 - AVERAGE RAIL SEAT FORCE VERSUS TIE SPACING

TABLE 18 - TIE BASE FORCE

Test Section	Test	% of Axle Load at Base Pad of Load Cell Tie									
		Pad 1	Pad 2	Pad 3	Pad 4	Pad 5	Pad 6	Pad 7	Pad 8	Pad 9	Pad 10
1	1	-	4.09	-	-	3.00	-	6.75	-	4.48	-
	2	-	4.15	-	-	3.09	-	4.73	-	4.65	-
	3	6.20	7.41	-	4.71	2.11	2.58	5.36	5.72	8.25	5.17
	4	-	5.33	4.68	6.04	-	2.35	7.49	5.62	-	-
2	1	3.35	3.93	-	1.89	-	-	4.24	4.43	8.92	-
	2	3.24	3.86	-	1.88	-	-	4.56	4.63	9.07	-
	3	2.50	4.87	-	4.86	1.19	1.62	4.39	4.11	5.80	6.53
	4	4.42	4.01	4.54	4.00	2.65	3.85	5.67	5.61	-	-
3	1	5.99	-	5.43	3.02	-	1.74	3.08	2.76	2.42	-
	2	4.31	-	3.74	3.31	-	1.85	3.33	2.98	2.58	-
	3	3.76	3.71	-	4.15	1.97	2.67	4.22	3.92	5.30	5.23
	4	4.80	4.53	5.66	6.12	3.24	4.27	5.04	3.28	-	3.93
6	1	-	4.10	-	0.92	0.75	0.28	4.35	2.40	2.30	-
	2	-	4.13	-	1.33	0.85	-	-	2.61	2.43	-
	3	1.54	6.80	-	2.88	0.31	1.64	7.88	4.25	3.65	2.47
	4	-	7.31	-	-	-	-	-	-	-	-
8	1	-	2.12	-	1.50	1.13	1.01	2.09	2.79	2.85	-
	2	-	5.91	-	1.65	1.44	1.20	2.52	3.07	3.11	-
	3	-	6.12	-	5.31	0.68	1.83	5.15	4.19	4.23	5.86
	4	1.28	4.67	-	5.61	-	3.18	4.95	4.49	4.26	4.54
9	1	3.19	3.94	4.45	1.49	0.12	2.50	2.95	3.83	3.57	-
	2	3.30	4.13	4.74	1.61	-	2.54	2.95	3.74	3.33	-
	3	4.04	3.02	-	3.31	1.50	1.92	3.01	4.83	3.40	2.69
	4	3.59	2.49	-	3.83	2.54	2.98	4.82	6.40	3.30	3.34

- Data not obtained

Change in load distribution with time can be seen in Figure 25. It indicates that the load tended to be more uniformly distributed along tie length as time progressed. This was particularly evident in Sections 2, 3, and 8.

Pressure between the tie and ballast was calculated using the tie width, pad length, and typical axle load. Tie width for concrete ties is 11 in. and for timber ties is 8 in. Pad length is 10.8 in. and typical axle load is 66 kips. The largest pad load for a concrete tie was 9.07% of axle load, measured in Section 2. The largest pad load for a wood tie was 7.88% of axle load, measured in Section 6. These values correspond to a ballast pressure of 50.4 psi below concrete ties, and 60.2 psi below wood ties.

Rail Stresses

Rail strains were measured using a pattern of strain gages at two cross sections, one directly above a tie and the other midway between ties. Gages were located directly under the rail head, at mid-depth, and on the top surface of the base. Curvature, calculated from rail strains, was converted to stress at the top fiber. For this calculation, a linear strain distribution throughout the rail depth was assumed. Calculated stress at the top of rail head are shown in Tables 19 and 20.

Stresses at the top of rail head are shown in Figures 26 and 27 for cross sections directly above a tie and midway between ties, respectively. Average stresses are plotted for the north and south rail and a line drawn to show the average of the two rails. The data show no definite trend.

Maximum stresses at both locations occurred in wood tie Section 6 during Test 2. The maximum stress at a cross section above a tie was 503 psi/kip of wheel load or 16,600 psi for a 33-kip locomotive wheel. The maximum stress at a cross section midway between ties was less and amounted to 374 psi/kip of wheel load or 12,300 psi for a 33-kip locomotive wheel.

Data obtained from the main array and from instrumentation 100 feet away during Tests 1, 2, and 3 were generally in good

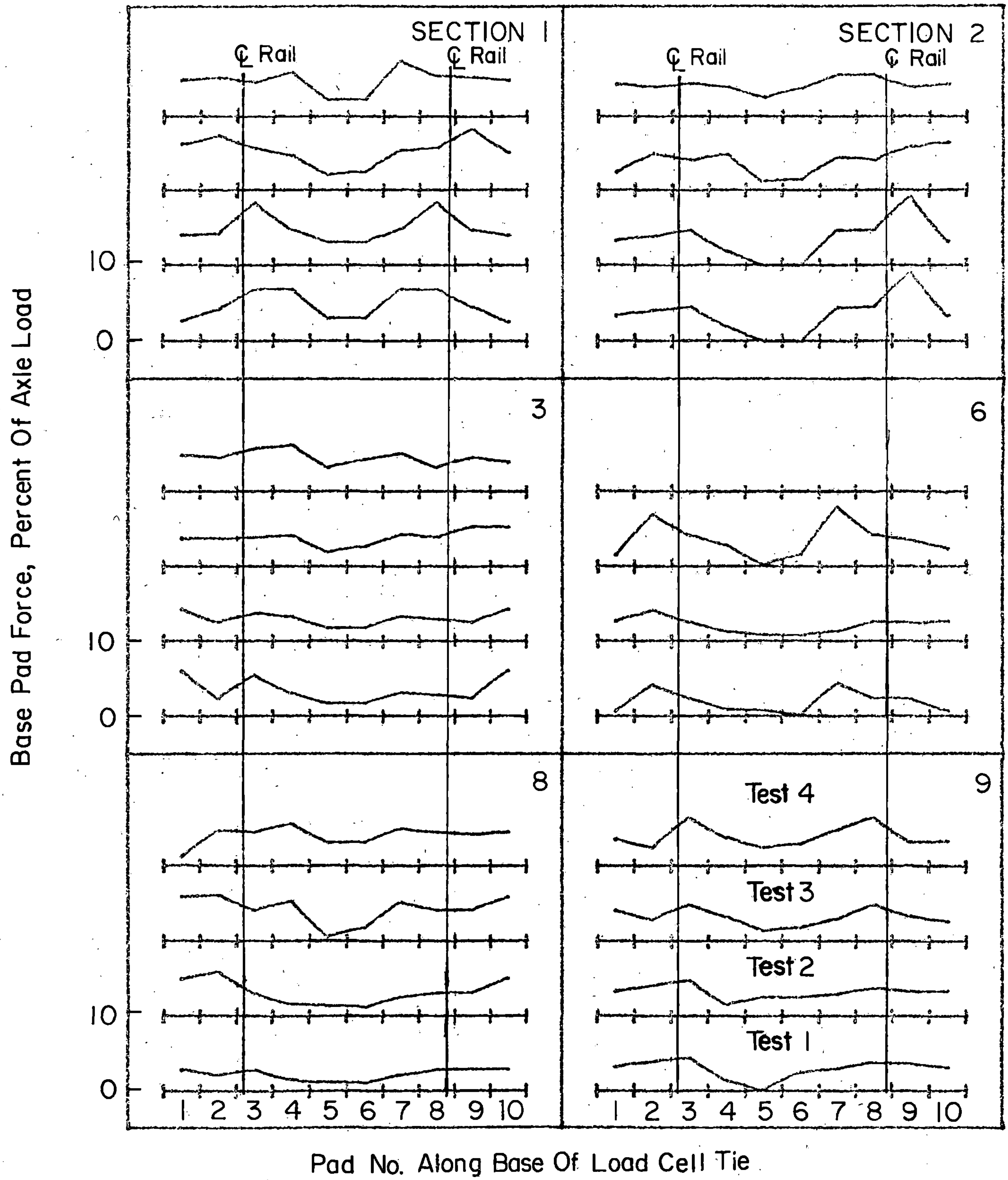


FIGURE 25 - DISTRIBUTION OF TIE LOAD TO BALLAST

TABLE 19 - RAIL STRESSES AT TIES

Section	Test	Rail Stress, psi/kip of Wheel Load					
		North Rail			South Rail		
		Max.	Min.	Ave.	Max.	Min.	Ave.
1	1	202	151	175	41	22	36
	2	188	152	173	69	36	52
	3	276	220	245	248	210	227
	4	230	187	213	243	195	219
2	1	-	-	-	79	54	70
	2	-	-	-	93	69	78
	3	233	168	204	309	205	243
	4	183	27	84	165	92	133
3	1	-	-	-	187	103	123
	2	-	-	-	141	82	121
	3	227	183	202	212	166	186
	4	257	167	225	185	93	151
6	1	409	279	374	430	341	390
	2	503	372	449	462	325	397
	3	208	146	184	200	120	167
	4	175	59	118	228	99	167
8	1	371	199	249	386	327	343
	2	306	165	242	429	277	338
	3	244	217	231	317	260	289
	4	209	121	162	143	89	116
9	1	267	129	218	224	167	204
	2	-	-	-	186	88	163
	3	335	278	308	317	256	283
	4	167	139	151	149	111	126

- Data not obtained

TABLE 20 - RAIL STRESSES BETWEEN TIES

Section	Test	Rail Stress, psi/kip of Wheel Load					
		North Rail			South Rail		
		Max.	Min.	Ave.	Max.	Min.	Ave.
1	1	195	159	177	176	59	142
	2	190	162	179	164	50	131
	3	263	224	238	209	152	179
	4	321	218	280	97	67	83
2	1	164	135	148	-	-	-
	2	168	142	152	-	-	-
	3	235	198	216	309	180	218
	4	214	113	167	237	153	207
3	1	-	-	-	250	186	230
	2	-	-	-	245	153	206
	3	226	178	197	205	138	170
	4	286	164	228	149	76	119
6	1	371	242	339	-	-	-
	2	374	229	343	-	-	-
	3	226	139	175	-	-	-
	4	241	149	205	-	-	-
8	1	294	229	280	272	208	248
	2	303	238	280	261	175	229
	3	269	223	242	182	148	172
	4	213	133	171	-	-	-
9	1	297	177	267	358	214	308
	2	284	201	245	289	163	250
	3	353	297	325	245	190	221
	4	238	189	206	291	209	253

- Data not obtained

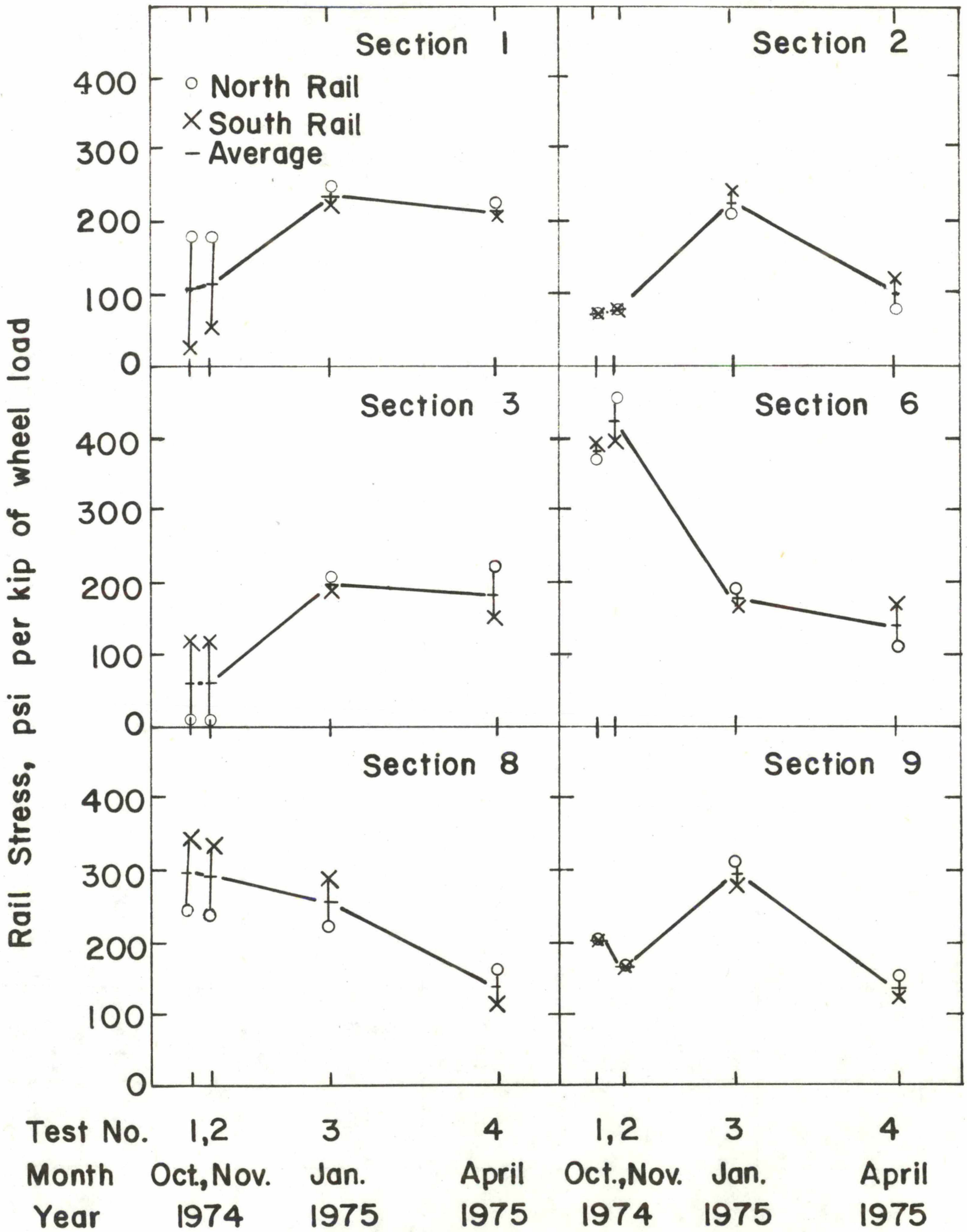


FIGURE 26 - AVERAGE RAIL STRESS AT TOP OF RAIL HEAD AT A FASTENER

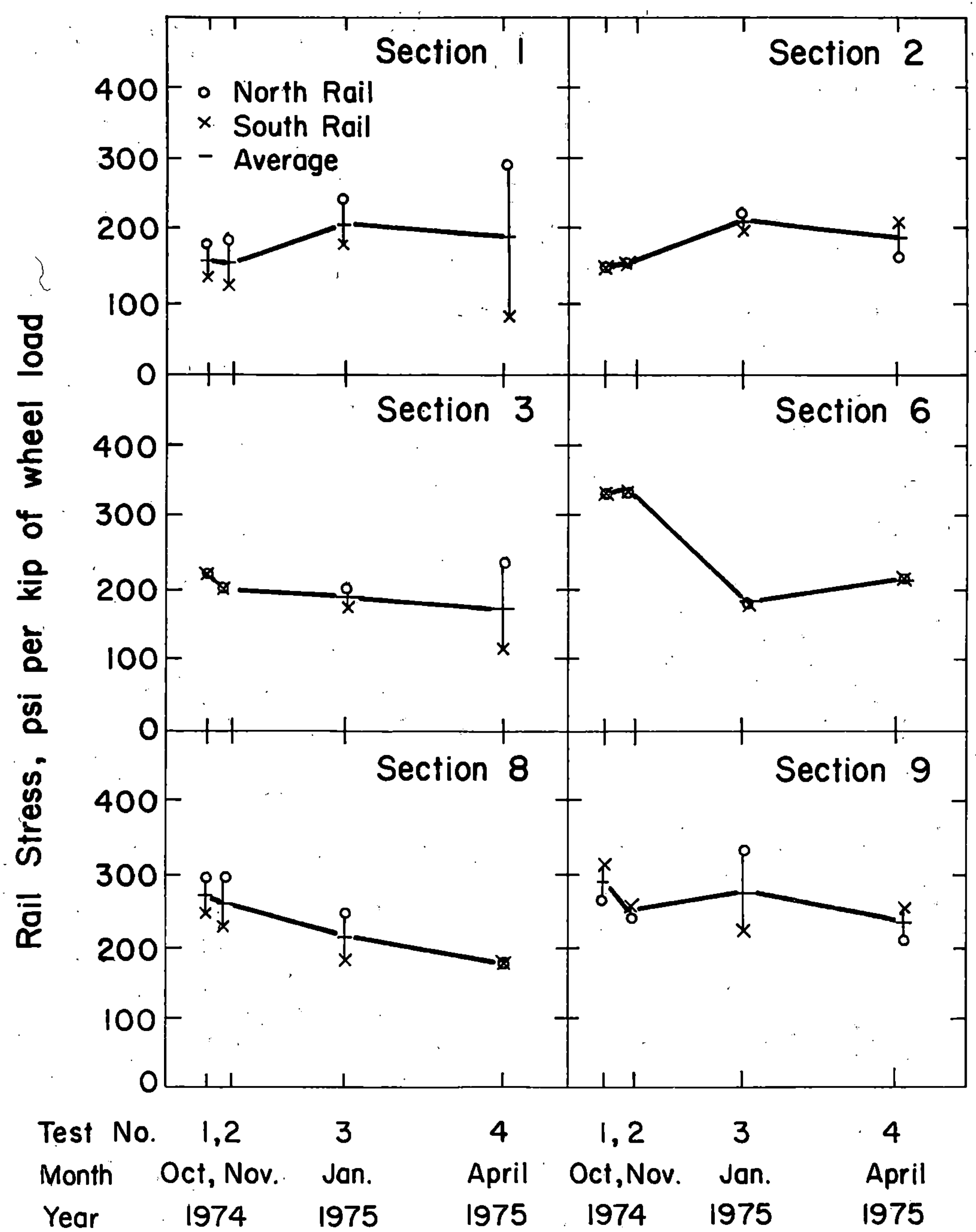


FIGURE 27 - AVERAGE RAIL STRESS AT TOP OF RAIL HEAD BETWEEN FASTENERS

agreement. However, this changed considerably for Sections 1 and 3 during Test 4 in April 1975. The 100-ft location showed lower stresses than those obtained from the main array.

Tie Bending Moments

Strains on concrete and wood ties were measured using a pattern of gages at two cross sections, one directly under the rail and the other midway between rails. Gages were located on tie sides near the top and bottom surfaces. Also, a gage was located at midheight on the cross section below the rail. Curvature, calculated from tie strains, was converted to bending moment. This was accomplished using the calculated moment of inertia of the section and the measured modulus of elasticity of the concrete or timber, as outlined in Appendix A. These results appear in Tables 21 and 22.

Average bending moments at the rail seat are shown in Figure 28. Values at the main array and a 100 feet away differed greatly.

As laboratory tests indicated, initial cracking of a concrete tie at the rail seat required an average bending moment of 260 in.-kips. For a 33-kip wheel load, this corresponds to 7,880 in.-lb/kip of wheel load. A maximum value of 5,250 in.-lb/kip of wheel load was recorded in Section 1 during Test 4 in April 1975.

As the maximum measured moment was less than the experimental cracking moment, it might appear that rail seat cracking had not occurred. However, as explained earlier, a visual examination of the ties showed bottom tensile cracking at most rail seats. It is assumed that the moments that caused cracking occurred at times when data were not being recorded.

Average bending moments at midlength are plotted in Figure 29. In some test sections, these values tended to increase with time. As laboratory tests indicated, initial cracking of a concrete tie at midlength requires an average bending moment of 189 in.-kip. For a 33-kip wheel load this corresponds to 5,730 in.-lb/kip of wheel load. A maximum value of 1,440

TABLE 21 - TIE BENDING MOMENT AT RAIL SEAT

Section	Test	Moment, in.-lbs/kip of Wheel Load					
		Main Array			100 ft East		
		Max.	Min.	Ave.	Max.	Min.	Ave.
1	1	286*	201*	225*	2410	1520	1900
	2	349*	160*	221*	2670	1350	1850
	3	1110	742	939	2410	1530	1920
	4	5250	2890	3750	1840	1360	1600
2	1	557*	410*	443*	1370	680	825
	2	693*	414*	487*	1440	917	1050
	3	1380	928	1090	2330	1950	2070
	4	2420	1160	1970	2000	1480	1720
3	1	440*	265*	317*	2160	1040	1270
	2	842*	292*	442*	1707	968	1180
	3	951	812	890	2310	1780	1970
	4	2080	1460	1790	2490	1890	2180
6	1	964	668	736	-	-	-
	2	1070	666	780	-	-	-
	3	1360	1090	1200	-	-	-
	4	-	-	-	-	-	-
8	1	-	-	-	2120	1120	1420
	2	-	-	-	1800	1090	1280
	3	2290	1740	2090	2130	1830	2000
	4	3420	2340	2900	1940	1250	1540
9	1	829	564	646	134	96	110
	2	944	601	725	79	39	67
	3	607	456	522	1331	1110	1220
	4	680	485	572	310	202	256

*Questionable value
- Data not obtained

TABLE 22 - TIE BENDING MOMENT AT MID LENGTH

Section	Test	Moment, in.-lbs/kip of Wheel Load					
		Main Array			100 ft East		
		Max.	Min.	Ave.	Max.	Min.	Ave.
1	1	350	285	308	295	151	183
	2	451	288	324	229	145	188
	3	748	647	693	613	537	575
	4	1440	1140	1280	-	-	-
2	1	167	74*	139	363	299	324
	2	224	100	134	417	335	360
	3	458	392	422	360	263	307
	4	447	207	304	-	-	-
3	1	75*	57*	64*	832	689	742
	2	88*	66*	75*	713	581	621
	3	450	368	413	334	314	321
	4	-	-	-	908	771	853
6	1	213	132	171	-	-	-
	2	282	164	205	-	-	-
	3	525	409	455	-	-	-
	4	-	-	-	-	-	-
8	1	-	-	-	501	363	482
	2	-	-	-	528	456	495
	3	-	-	-	-	-	-
	4	945	748	837	826	711	755
9	1	715	561	612	732	564	604
	2	890	666	736	955	732	856
	3	359	287	332	-	-	-
	4	689	598	640	524	377	447

*Questionable value
- Data not obtained

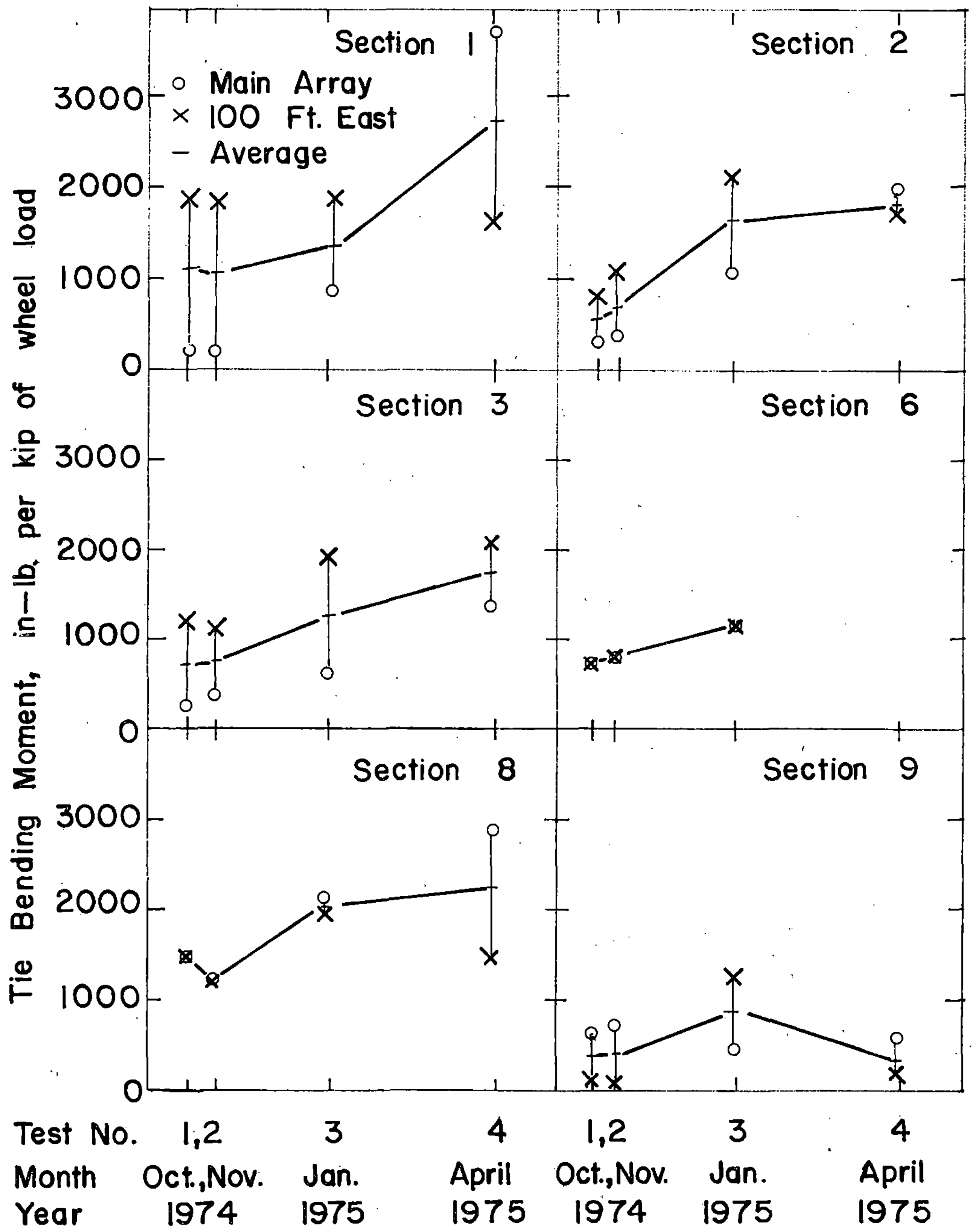


FIGURE 28 - TIE BENDING AT RAIL SEAT

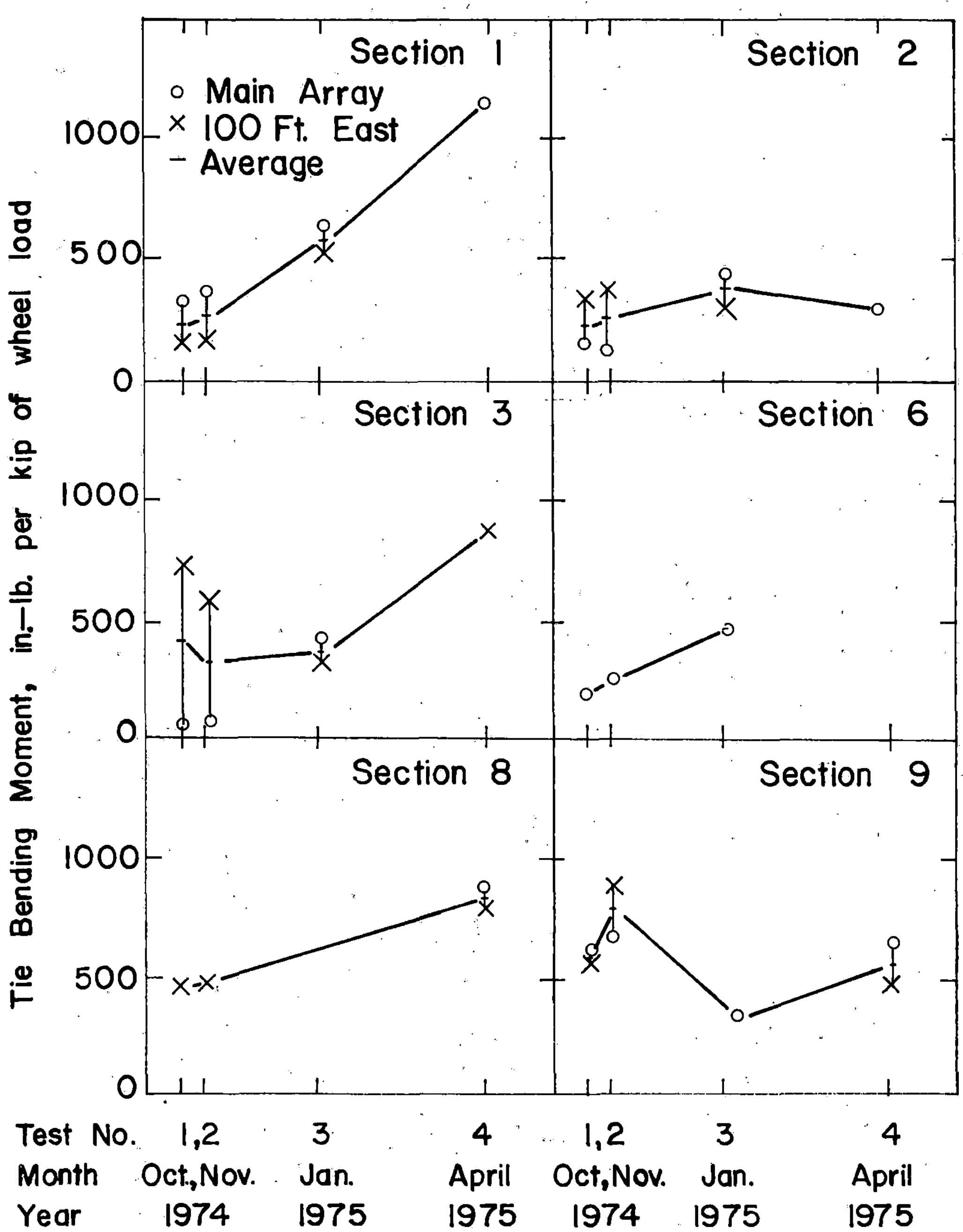


FIGURE 29 - TIE BENDING AT MID LENGTH

in.-lb/kip of wheel load was recorded in Section 1 during Test 4 in April 1975.

Pressure at Ballast - Embankment Interface

Pressure was measured using pressure cells in the embankment just below the ballast. Cells were placed below the load cell tie, below the adjacent instrumented tie and between ties at the main array. At each location the cells were below each rail and below the track centerline.

Maximum pressure at ballast embankment interface, expressed as psi/kip of wheel load, is shown in Table 23. Data from pressure cells located below the instrumented ties are plotted for a 33-kip wheel load in Figure 30. Also shown in Figure 30 are data from base pads of the load cell ties. These data are converted to pressure under the tie for comparison with interface pressure cell data.

Peak values of interface pressures and peak values from the load cell ties occurred simultaneously. In some instances pressure cells indicated lower values. When these low values occurred, they were probably inaccurate due to displacement of the pressure cells in the soft subgrade. However, pressure values obtained from pressure cells generally showed the same trend as those obtained from tie base pad pressures.

Effect of Train Speed

Impact factor due to speed effect is a measure of increased loading due to increased speed. Impact factor is zero for a standing or slowly moving train, and increases as speed increases.

Impact factor was calculated from the increase in rail seat loads due to 30 mph speed as compared to that at creep speed in the first data acquisition. Using average values of the rail seat force for the north rail seat shown in Table 17, impact factors were 0.11 for Section 1, zero for Section 2, 0.15 for Section 3, 0.14 for Section 6, 0.25 for Section 8, and 0.04 for Section 9. There was no obvious effect of track stiffness on

TABLE 23 - MAXIMUM PRESSURE AT BALLAST-EMBANKMENT INTERFACE

Section	Test	Pressure Below Instrumented Tie, psi/kip			Pressure Between Ties, psi/kip			Pressure Below Load Cell Tie, psi/kip		
		North	Center	South	North	Center	South	North	Center	South
1	1	1.40	0.62	0.99	0.06	–	0.25	–	0.50	0.94
	2	1.66	0.57	0.37	0.02	0.08	0.23	–	0.51	0.78
	3	0.62	0.50	0.48	0.28	0.22	0.29	0.74	0.28	0.40
	4	0.44	–	0.26	0.18	0.22	0.21	0.18	0.39	0.48
2	1	–	–	–	–	0.12	0.36	–	–	0.65
	2	–	0.42	–	–	0.10	0.36	–	–	0.64
	3	0.78	0.48	0.24	–	0.22	0.50	0.07	0.23	0.47
	4	0.68	0.81	–	–	0.37	–	0.47	0.57	0.71
3	1	–	–	–	–	0.33	0.29	–	0.48	0.93
	2	–	–	–	–	0.45	0.29	–	0.49	0.98
	3	0.09	0.40	0.25	0.35	0.29	0.47	0.65	0.42	0.48
	4	0.86	0.37	0.21	0.36	0.58	–	0.99	0.92	0.25
6	1	–	–	0.15	0.15	0.07	–	–	–	0.28
	2	–	–	0.11	0.16	–	–	–	–	0.27
	3	1.06	0.35	0.47	0.69	0.19	0.60	0.26	0.48	0.81
	4	0.56	–	0.74	0.83	0.06	0.57	0.39	0.03	0.53
8	1	–	–	–	–	–	0.03	–	–	0.17
	2	–	–	–	–	–	0.02	–	–	0.13
	3	0.36	0.36	0.56	0.51	0.26	0.53	0.68	0.50	0.15
	4	0.28	0.56	0.35	0.44	0.61	0.47	0.28	0.41	0.53
9	1	–	–	–	–	–	–	–	–	–
	2	–	–	–	–	–	–	–	–	–
	3	0.40	0.49	0.79	0.49	–	0.41	0.64	0.59	0.83
	4	–	–	0.75	0.11	–	0.31	0.19	0.45	0.79

– Data not obtained

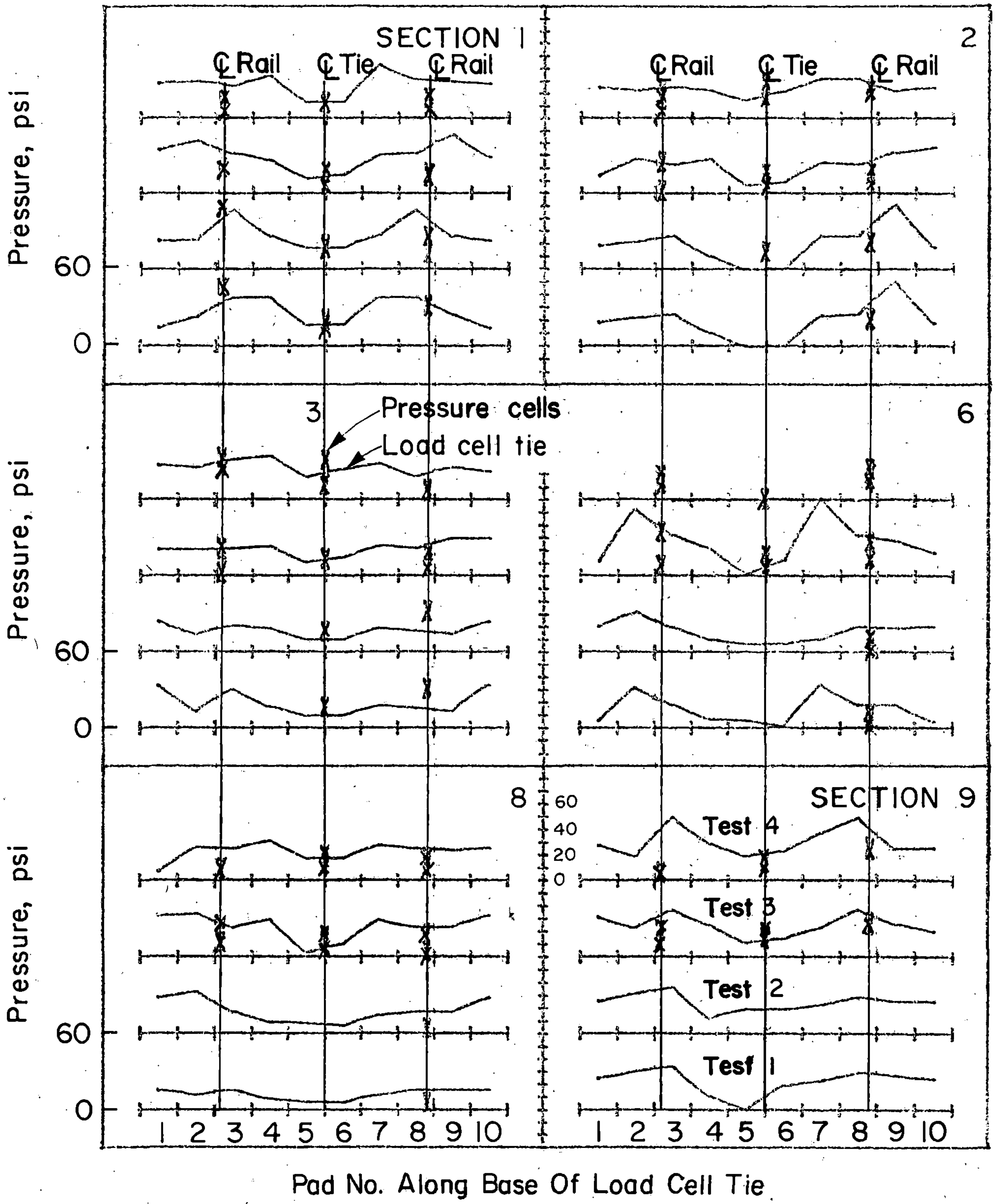


FIGURE 30 - PRESSURE AT BALLAST-SUBGRADE INTERFACE

impact factor. The lowest impact factor was calculated for Section 2 that had the highest track modulus at the time data were taken, while the lowest impact factor was calculated for Section 6 that had an intermediate track modulus value.

Effect of Flat Wheels

Thirteen "flat-wheel" effects were recorded as 24 trains passed during Tests 3 and 4. Peak effects as measured on load cell tie rail seats are shown in Figure 31. Impact factor due to flat wheel was calculated from the increase in rail seat load due to a wheel flat as compared to that due to a round wheel on the same car. It is interesting to note that flat wheels of lightly loaded cars developed higher impact factors than those of heavily loaded cars. However, the magnitude of imposed loads were less than those imposed by heavily loaded cars.

Traces of rail seat force for two different cars with wheel flats are shown in Figure 32. Impacts of flat wheels caused fluctuations in response immediately preceeding and following the impact of the flat spot on the rail.

Longitudinal Rail Movement

Longitudinal rail displacement was measured at eight locations in each test section. Linear Variable Differential Transformers were connected to each rail at the main array, at the 100-ft east location, and at two intermediate locations. A maximum amplitude of displacement for each case was obtained by taking the sum of the maximum east and west displacements for a complete train passage. The greatest maximum amplitude in each test section is shown in Figure 33 for each trip. The difference in position of the rail before and after a train passage is termed residual displacement. Maximum residual displacement in each test section is also shown in Figure 33.

The least longitudinal rail movement was measured in Section 8, with concrete ties spaced at 27 in. on 15-in. ballast.

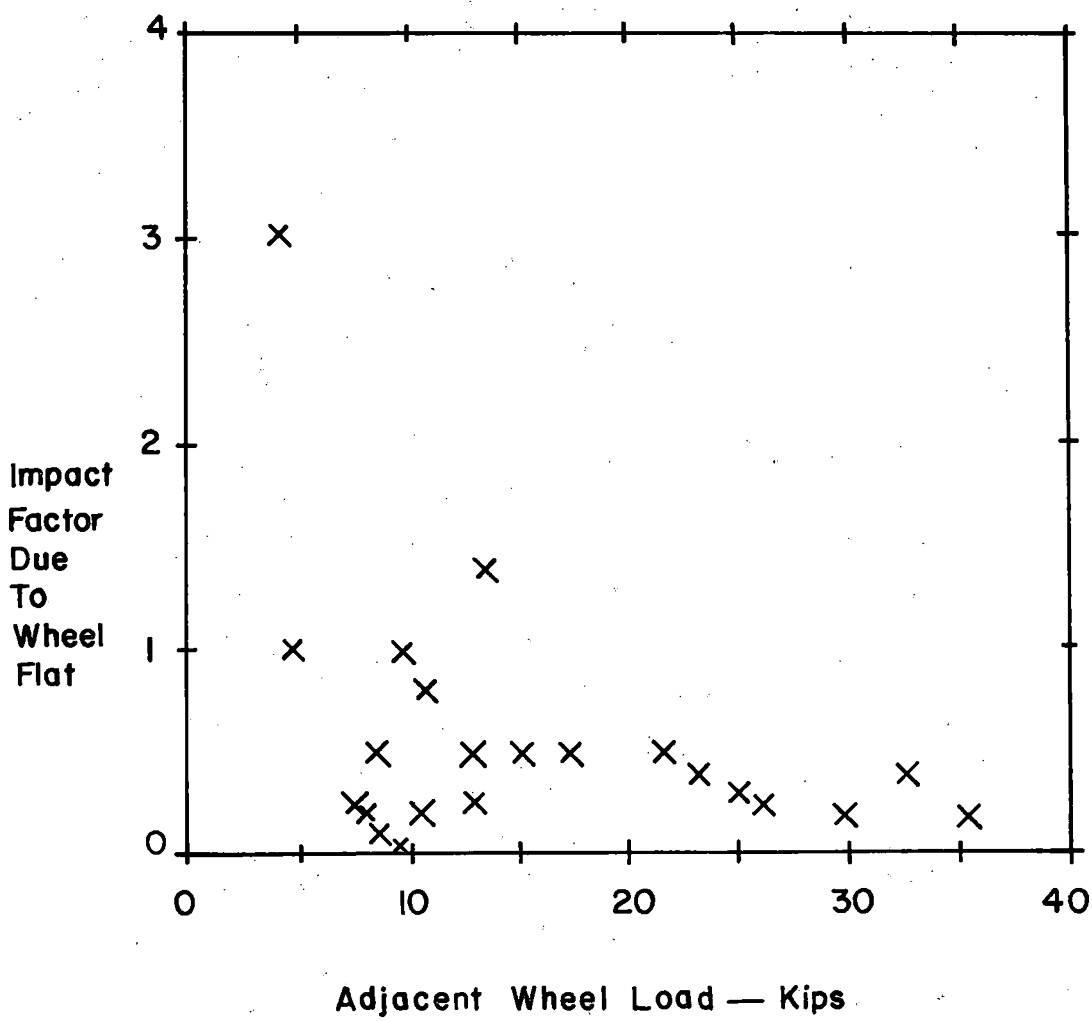


FIGURE 31 - IMPACT FACTOR DUE TO WHEEL FLATS

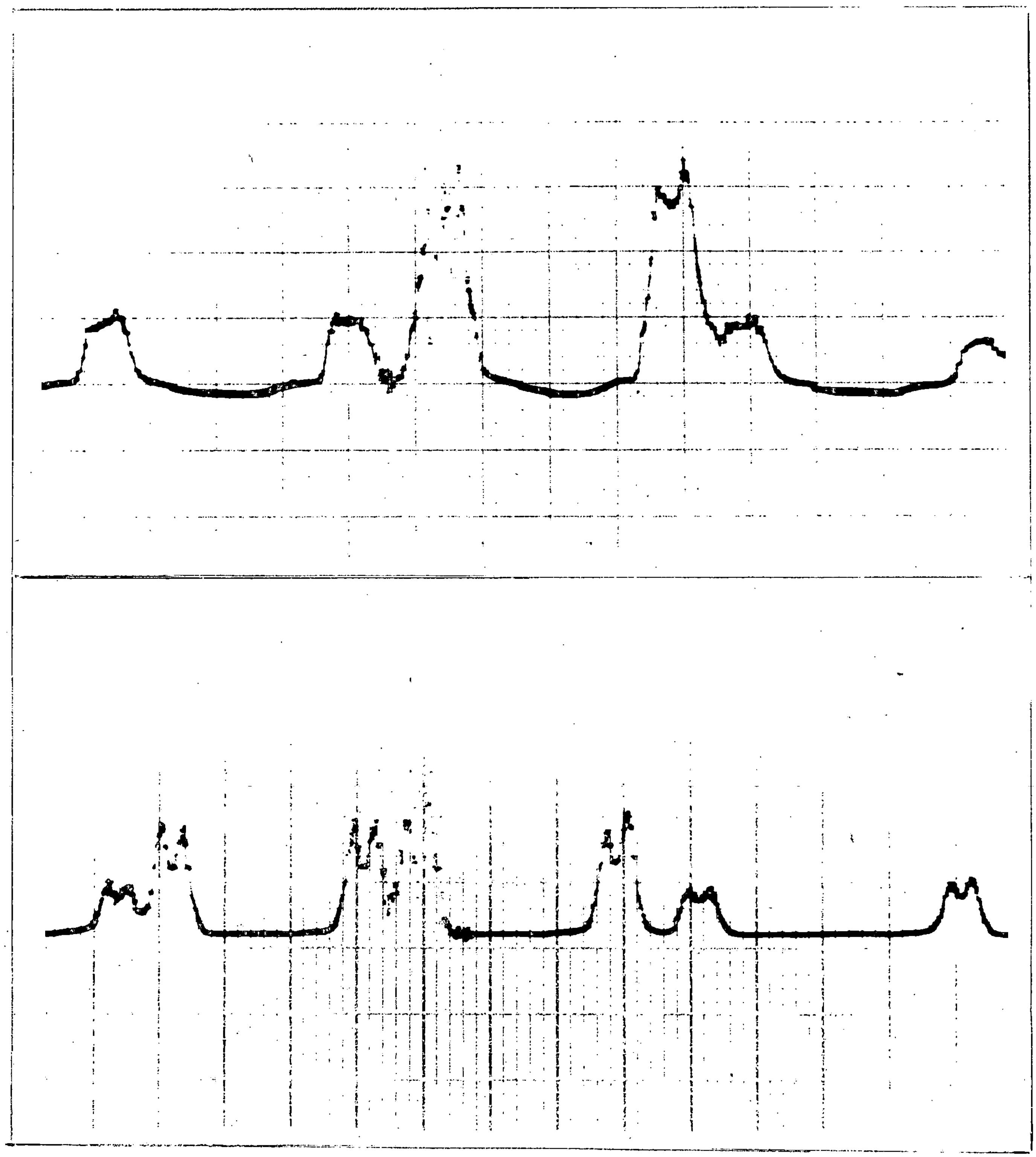


FIGURE 32 - DATA TRACE OF WHEEL FLAT EFFECT

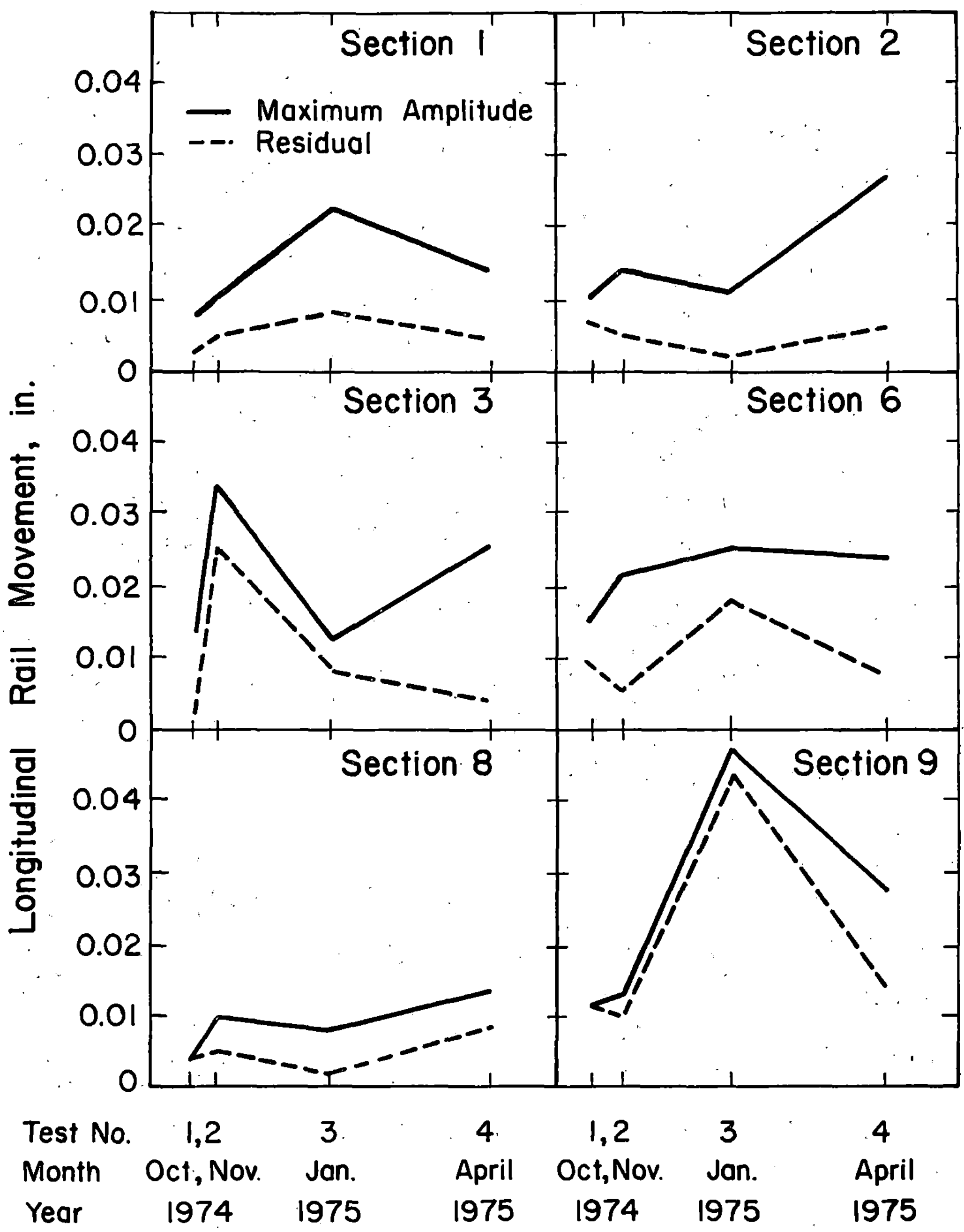


FIGURE 33 - LONGITUDINAL RAIL MOVEMENT

The largest rail movement was measured in the standard wood tie track, Section 9.

Rail and Tie Accelerations

Rail and tie accelerations were measured using nine accelerometers, two per rail at the instrumented tie at the main array and five at the tie itself. Several accelerometer locations are shown in Figure 34. Electrical output from the nine accelerometers was recorded during the passage of one train as each tie track section was visited in Tests 3 and 4. Analog outputs of the accelerometers were frequency-modulated for recording purposes.

Data reduction as described in Appendix E produced a Power Spectral Density (PSD) plot for each recording. These plots are presented in Appendix C. The acceleration amplitude versus frequency is plotted from zero to maximums of 5, 10 or 20 kHz so as to include all significant data. The initial 3.2 seconds of the full train record are included in averaging used to generate the PSD plots. As shown in Appendix E, the initial, final, and intermediate portions of the record produced similar PSD plots.

Peak values of rail acceleration are shown in Table 24 for frequencies ranging from zero to 2 kHz. These values were calculated from PSD plots. The sum of accelerations in each of the eight subgroups represents a possible peak value and will be used for comparison.

Peak acceleration at the rail is related to impact load on the track. For Test 3 in January 1975, rail acceleration decreased as concrete tie spacing was reduced. Rail acceleration for the wood-tie track was 2.5 times that of the concrete tie track. This difference was considerably less for Test 4 in April 1975. At this time, wood-tie track values were less and the concrete tie track values were higher than those measured in Test 3.

Peak values of tie acceleration are shown in Table 25. These values were calculated from PSD plots of vertical accel-

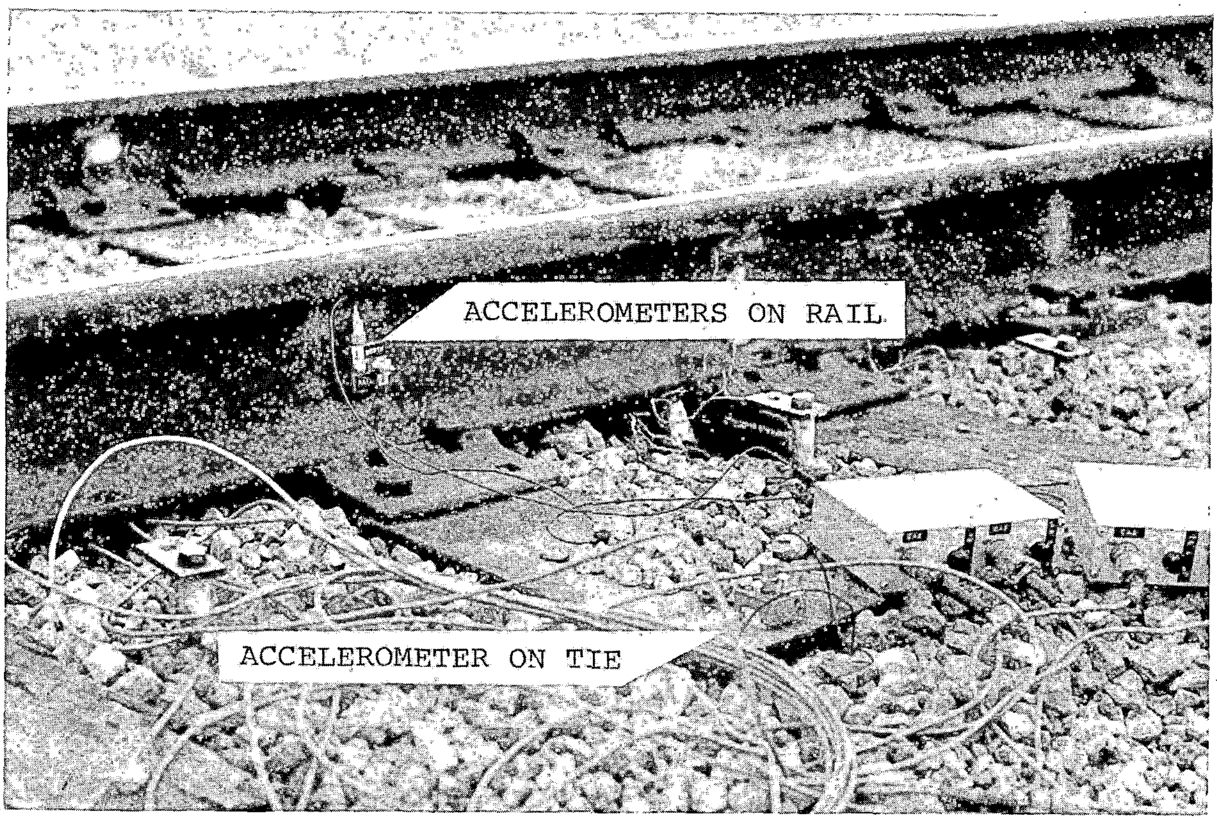


FIGURE 34 - VIEW OF ACCELEROMETERS ON RAIL AND TIE

TABLE 24 - RAIL VERTICAL ACCELERATIONS

Test	Section	Rail Accelerations* Over Range of Frequencies								
		Frequency Range, Hz								
		0-250	250-500	500-750	750-1000	1000-1250	1250-1500	1500-1750	1750-2000	Total
3	1	0.5	0.8	1.2	1.9	8.0	0.3	1.8	2.8	17.3
4		5.9	0.9	2.4	3.2	12.3	3.3	1.0	1.3	30.4
3	2	1.5	0.3	1.7	1.4	3.0	5.2	0.8	1.2	15.1
4		2.0	1.4	2.1	2.8	6.9	10.0	2.5	4.5	32.1
3	3	2.0	1.2	0.9	1.1	1.0	2.0	0.6	3.0	11.8
4		6.3	1.1	2.0	2.0	8.7	2.0	5.0	2.1	29.2
3	6	0.8	4.6	7.8	10.1	21.0	3.4	2.8	2.6	53.1
4		7.5	4.0	8.5	13.8	1.6	2.8	1.6	1.9	41.6
3	8	0.7	0.8	1.0	0.6	0.7	2.8	1.6	0.8	8.9
4		2.0	0.6	1.5	1.3	8.7	4.9	4.2	7.1	30.4
3	9	0.6	4.0	10.3	21.2	2.5	2.1	2.6	3.6	46.9
4		8.0	3.9	7.3	12.0	1.7	2.3	1.5	2.2	38.8

*Calculated from Power Spectral Density Plots, as a multiple of gravity ($g = 32.2$ ft/sec^2)

TABLE 25 - TIE VERTICAL ACCELERATIONS

Test	Section	Tie Accelerations* Over Range of Frequencies								
		Frequency Range, Hz								
		0-250	250-500	500-750	750-1000	1000-1250	1250-1500	1500-1750	1750-2000	Total
3	1	0.9	1.9	1.6	1.3	0.6	0.8	0.5	1.0	8.6
4		4.7	6.3	6.9	10.6	8.7	9.1	5.0	3.8	55.1*
3	2	0.2	0.5	0.5	0.7	0.3	0.3	0.4	0.4	3.3
4		1.6	3.4	3.1	3.2	3.0	2.4	2.8	4.5	24.0
3	3	0.4	0.4	0.5	0.4	0.6	0.8	0.5	0.5	4.1
4		0.8	0.6	0.8	0.9	1.1	1.4	0.8	0.9	7.3
3	6	0.7	2.8	3.6	3.9	4.3	4.1	1.5	1.6	22.5
4		0.4	1.8	7.9	8.9	2.1	3.0	4.0	3.7	31.8
3	8	0.4	0.7	0.9	0.5	0.5	0.3	0.3	0.2	3.8
4		0.8	1.2	2.2	2.1	3.0	2.2	1.9	1.7	15.1
3	9	0.9	3.8	5.9	7.3	10.3	12.8	5.6	1.5	48.1
4		0.6	2.4	3.5	3.3	6.9	5.7	1.5	0.4	24.3

*Calculated from Power Spectral Density Plots, as a multiple of gravity ($g = 32.2$ ft/sec^2)

erations measured at tie rail seats. Data obtained in Test 4 show that for concrete tie track tie acceleration decreased as tie spacing decreased. Tie acceleration measured in Test 3 for wood tie sections was at least 2.6 times that measured for any concrete tie section. At Test 4, the relationship reversed and acceleration measured for concrete ties in Section 1 was 2.3 times that measured for timber ties in Section 9. This might indicate that the timber ties were well seated in the ballast whereas the concrete ties in Section 1 were losing support.

Comparison of values of rail and tie acceleration generally indicates considerably more vibration in the rail. In Test 3, the average rail acceleration for concrete tie sections was 2.95 times the average tie acceleration. This attenuation of vibration to the tie changed in Section 1 during Test 4 after the support condition had deteriorated.

TEST RESULTS - BEAM AND SLAB SECTIONS

Data on the behavior of cast-in-place beam Section 4, cast-in-place slab Section 5, and precast beam Section 7 were obtained on four separate occasions. An initial test was made in April 1973 prior to opening track to mainline traffic. The purpose of this test was to determine track modulus at the main arrays of the three sections. Only limited deflection instrumentation was monitored.

A second test was made in late October 1974, after the new fastener anchorage system was installed and before reopening the track to mainline traffic. Only instrumentation at the main array was monitored.

The third and fourth tests were made in December 1974 and April 1975. In these tests all instrumentation at main and secondary arrays was monitored. All data reported is referenced to section number. Main arrays are designated Sections 4, 5, or 7. Secondary arrays are designated 4-1 and 5.

Beam and slab data are reported for engine wheel loads unless otherwise specified. Engine wheel loads were used for analysis because, in most cases, they produced the largest readings. Also, their nearly equal wheel loads provided practically the same load condition for each train passage.

For each train monitored, data were reduced for the engine axle that produced the largest readings in all instrumentation. The selection was not critical since the effect of all engine axles was about the same.

A comparison of the 26 different engine wheel loads obtained from ATSF showed an average wheel load of 32,800 lb with a coefficient of variation of less than one percent. Computed wheel loads assume that engine weight was distributed evenly between all wheels. Designation and weight of engines monitored are given in Appendix D.

Beam and Slab Deflections

Deflections of concrete beams and slabs in Sections 4, 5, and 7 were measured on one side at a joint edge and at middistance between joints at the edge.

Edge deflections between joints for the test train passing the sections in April 1973 are presented in Figure 35. This figure indicates that deflections under ballast car wheels were up to 63% higher than those under engine wheels. This difference, however, decreased in later data-acquisition trips. In the December 1974 trip, maximum deflection under ballast car wheels was only 23% higher than that under engine wheels. During the April 1975, test regular traffic was monitored. In this case, maximum deflection occurred under the engine wheels.

Average deflections between joints and at joints for tests made in 1974 and 1975 are shown in Figures 36 and 37. These data were obtained for engines passing the test section at 30 mph in October and December 1974, and 50 mph in April 1975. The lowest deflections were measured in slab Section 5-1 and the largest deflections were measured in precast beam Section 7. Figures 36 and 36 also indicate that deflections increased significantly with time in all sections except Section 5. Deflection of Section 5 increased between October and December 1974, and then decreased slightly between December 1974 and April 1975.

Increases in deflection can be attributed primarily to a change in support condition at the main arrays. Such change may have been caused by improper consolidation of ballast and subgrade or lack of suitable drainage. The presence of numerous subgrade instrumentation at the main arrays made it difficult to properly consolidate subgrade materials around the instrumentation. Lack of suitable drainage was observed on several occasions as water was seen pouring from horizontal extensometer tubes. Also, during the last data-acquisition trip, mud pumping was observed through joints and around the beams if Section 7 as shown in Figure 38.

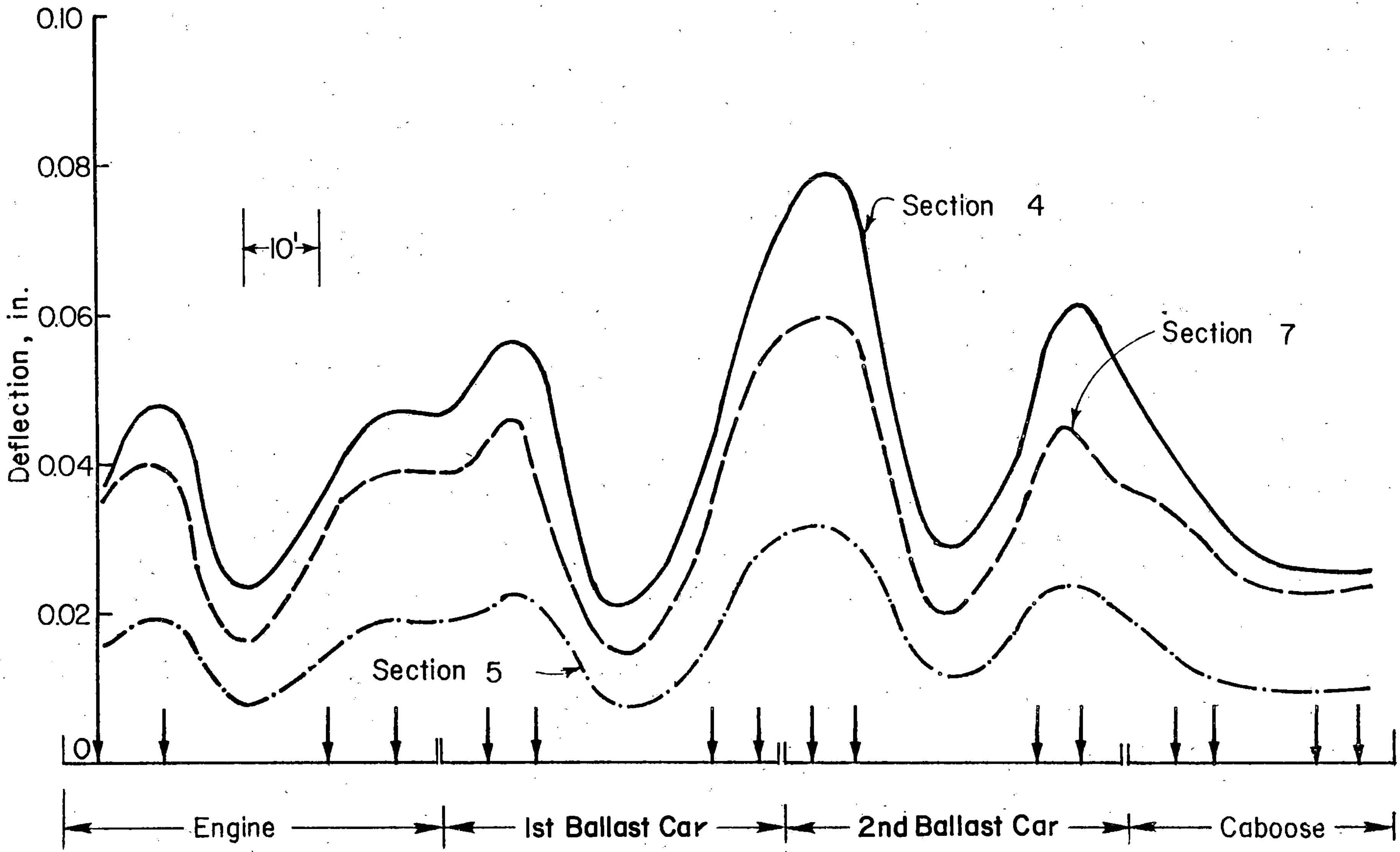


FIGURE 35 - BEAM AND SLAB DEFLECTIONS AT MIDDISTANCE BETWEEN JOINTS

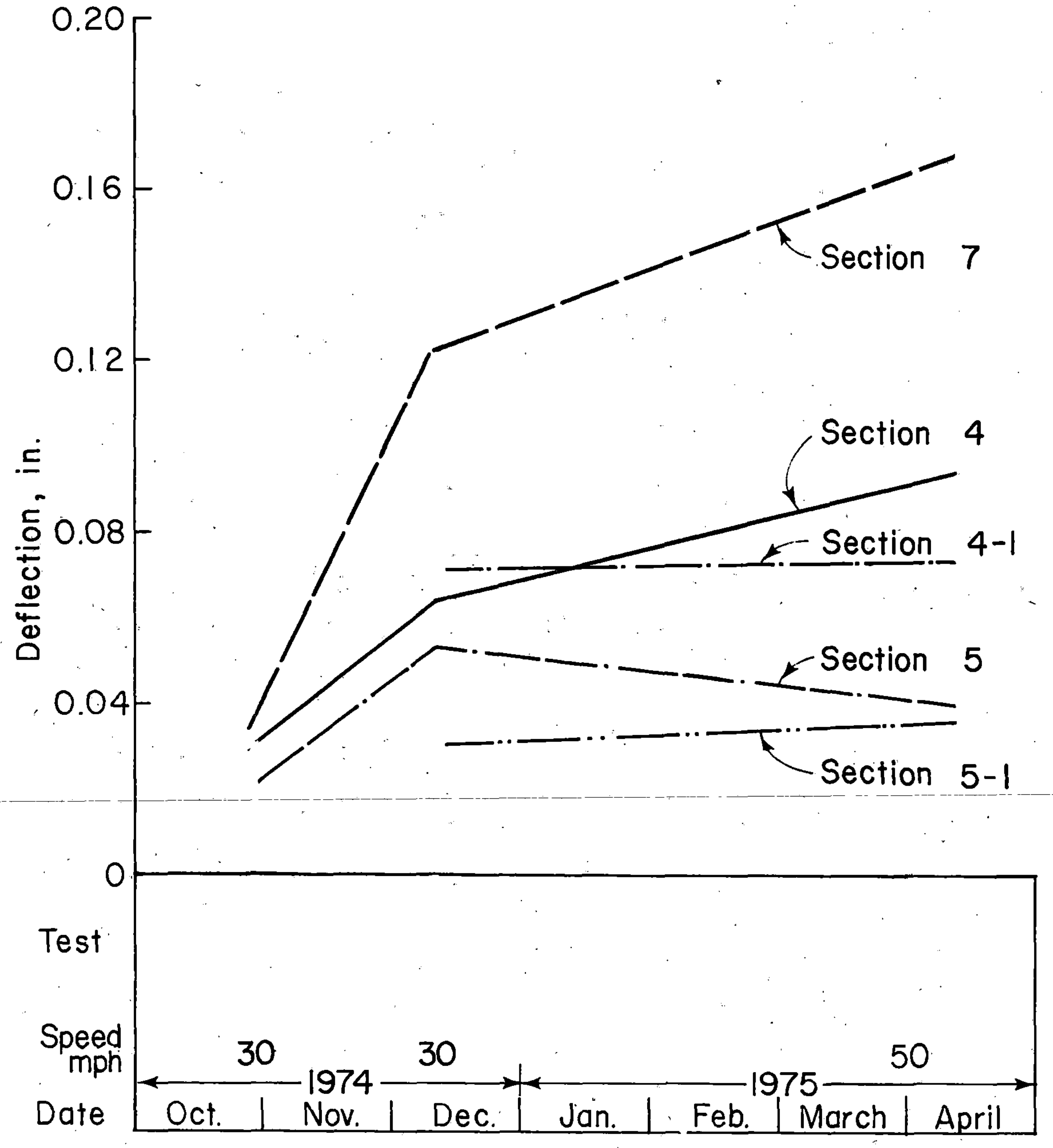


FIGURE 36 - AVERAGE BEAM AND SLAB DEFLECTIONS BETWEEN JOINTS

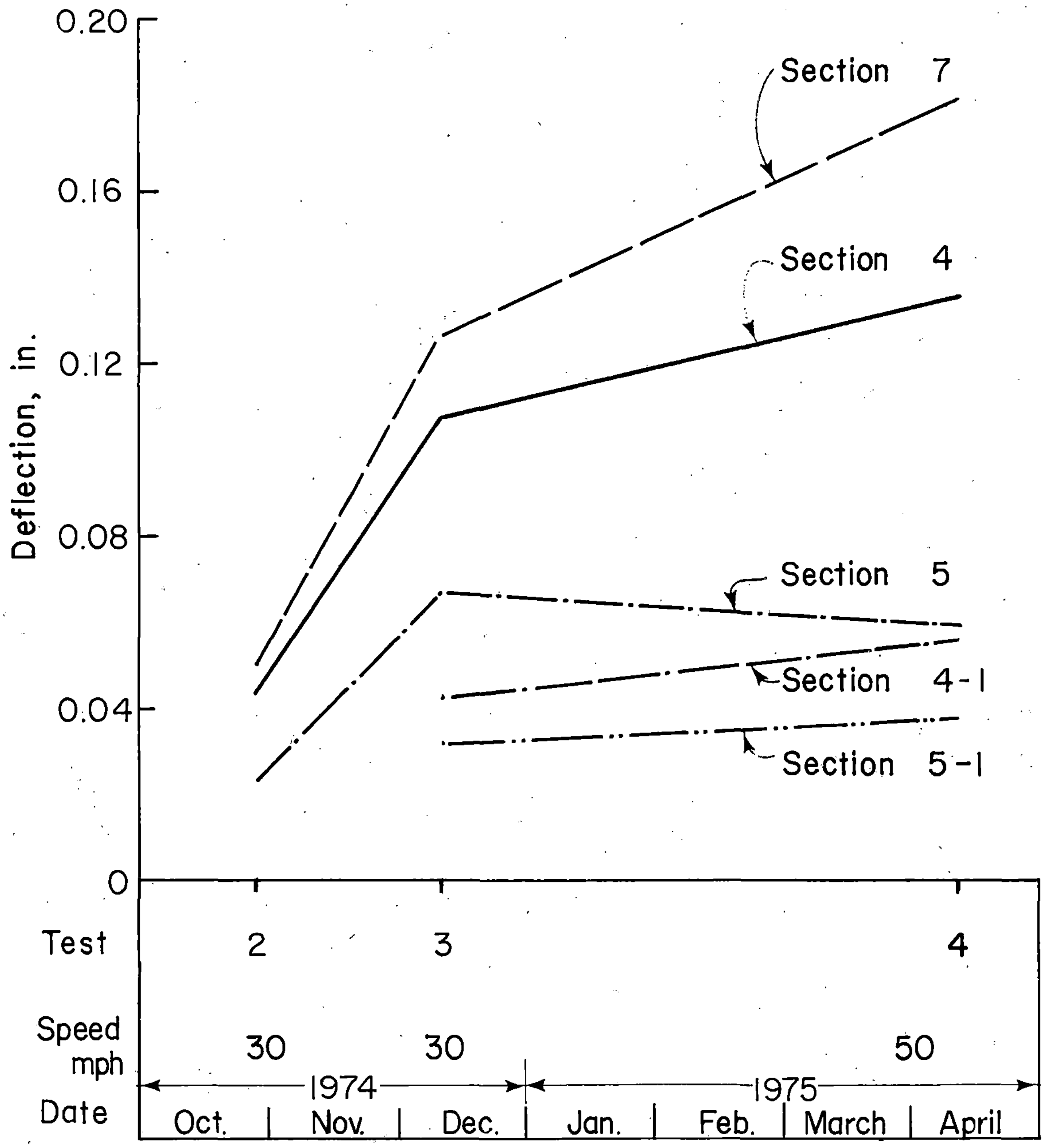


FIGURE 37 - AVERAGE BEAM AND SLAB DEFLECTION AT JOINT

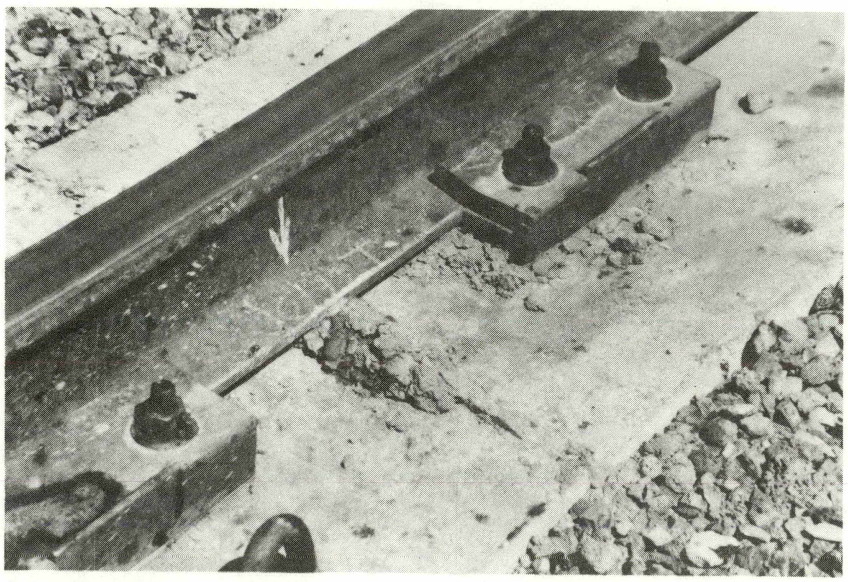


FIGURE 38 - MUD-PUMPING IN SECTION 7

Rail Deflection

Vertical rail movement relative to the concrete panels was measured between joints halfway between two fasteners. Average rail movements are listed in Figure 39.

Rail deflection was calculated by adding rail movement relative to the concrete to the deflection of the beams or slabs. Rail deflection under a moving train is shown in Figure 40 for slab and beam sections. These data indicate that rail deflection under ballast car wheels was larger than under engine wheels. This may be attributed to closer axle spacing and unrecovered rail movement caused by leading axles.

Track Modulus

Track modulus for beam and slab sections were computed using the equation:[6]

$$k = \sqrt[3]{\frac{P^4}{64EI_y z^4}}$$

where:
- k = track modulus, lb/in./in.
- P = wheel load, lb.
- E = modulus of elasticity of the rail steel (29×10^6 psi)
- I_y = moment of inertia of the rail (94.9 in.^{4} for 136 lb rail)
- z = rail deflection, in.

Track modulus for the different sections and different data-acquisition trips is presented in Table 26. These values were calculated for engine wheel load that averaged 32,800 lb. As expected, the highest value was calculated for slab Section 5, and the lowest value for precast beam Section 7. These data also show that the track modulus of all sections generally decreased with time. Modulus values in April 1975 were about 36% of initial values in April 1973 for Sections 4 and 5, and 17% for Section 7.

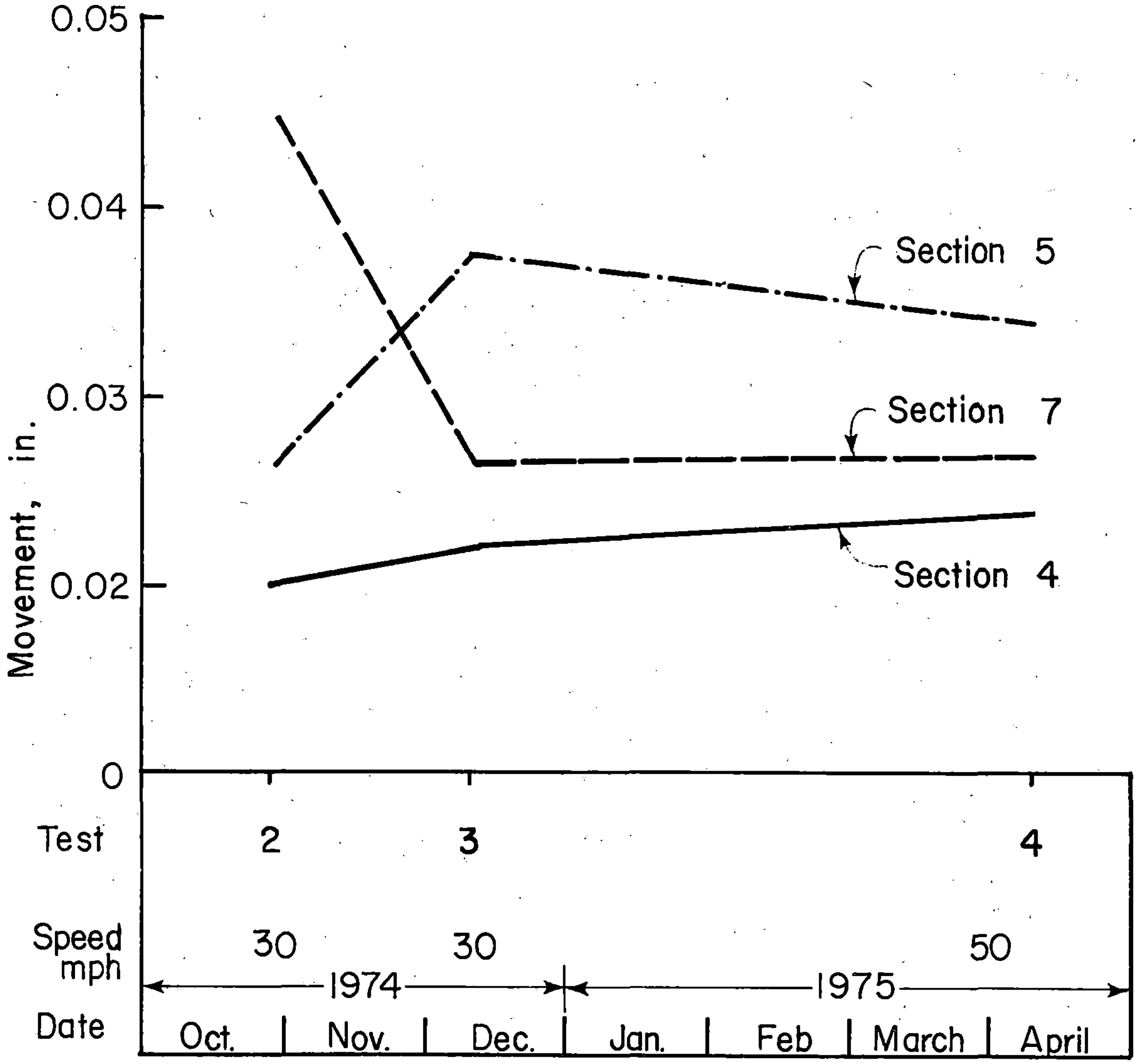


FIGURE 39 - VERTICAL RAIL MOVEMENT BETWEEN FASTENERS

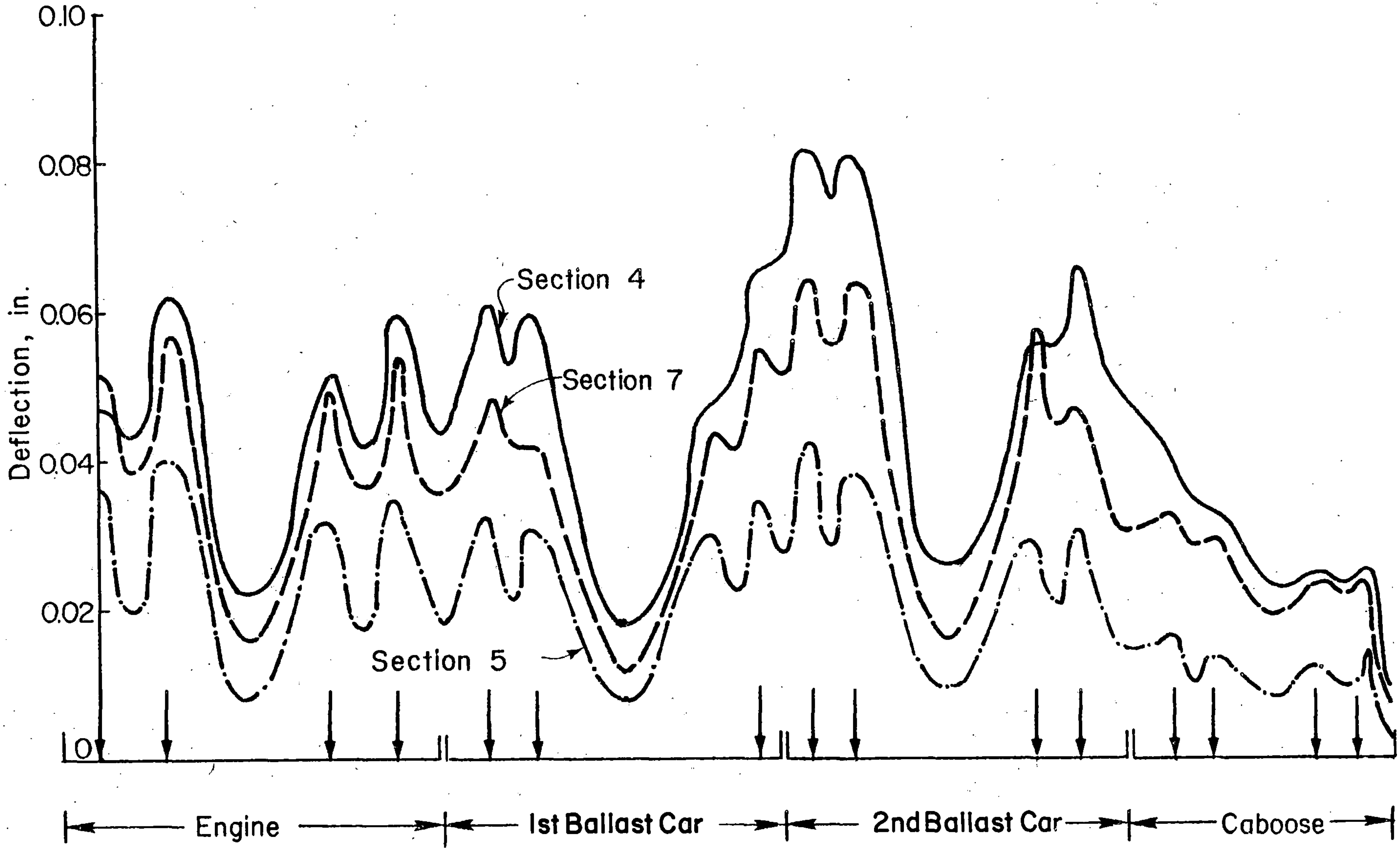


FIGURE 40 - RAIL DEFLECTION UNDER TRAIN MOVING AT CREEP SPEED IN APRIL 1973

TABLE 26 - TRACK MODULUS FOR SLAB AND BEAM SECTIONS

Section	Track Modulus, lb/in./in.			
	4/23/73	11/28/74	12/10/74	4/11/75
4	9,347	10,540	4,902	3,229
5	16,149	11,466	4,617	6,083
7	9,612	5,769	2,414	1,683

Rail-Fastener Loads

Since rail-fastener loads were not scheduled for measurement during the first two trips and because of the sensor malfunction during the December 1974 trip, loads were obtained only for the final test in April 1975. Details of sensors and the difficulties associated with their field use are presented in Appendices A and F. Rail-fastener loads were recorded by one sensor on the south rail at all arrays, two sensors on the north rail at the main arrays, and one sensor on the north rail at the secondary arrays.

The distribution of wheel load to adjacent fasteners was determined from static tests conducted in track. In these tests, a static load was applied to the rail and the response of the fastener directly beneath the load and the one on the west side were monitored. Load transmitted to the adjacent east fastener was assumed equal to that measured for the adjacent west fastener.

Distribution of a single static load to the fastener directly beneath the load and an adjacent one, expressed as percentage of the applied load, is presented in Table 27. These data indicate that the portion of a wheel load transmitted to a fastener varied for the different sections. The largest load transmitted to a fastener was calculated for Section 7 and was 79.4% of the wheel load.

Variations in load distribution, shown in Table 27, were due primarily to nonuniformity in rail support at fasteners. Fastener inspection during sensor installation revealed that the fasteners were not shimmed uniformly. This resulted in height differences between fasteners and influenced load distribution.

Maximum loads transmitted to a fastener under the moving train are presented in Table 28 for the different test sections. These data indicate that largest fastening loads measured were 27, 31 and 34 kips for Sections 4, 7, and 5, respectively. These loads were 82%, 96% and 103% of the average locomotive wheel load.

TABLE 27 - DISTRIBUTION OF WHEEL LOAD TO FASTENERS

Test Section	Percent Wheel Load Transmitted to Fastener			
	Load at Fastener 1		Load at Fastener 2	
	Fastener 1	Fastener 2	Fastener 1	Fastener 2
4	72	14	20	60
5	78	11	30	40
7	40	30	10	80

TABLE 28 - MAXIMUM RAIL-FASTENER LOADS

Test Section	Run No.	Load on Fastener 1		Load on Fastener 2	
		Load, kip	% of Wheel Load*	Load, kip	% of Wheel Load*
4	1	27	82	26	80
	2	23	70	19	60
5	1	33	101	16	47
	2	34	103	17	50
7	1	14	43	31	96
	2	15	46	31	93

*Based on 32,800 lb average locomotive wheel load.

Interface Soil Pressure

Interface soil pressures were measured at joints and midway between joints of a 10-ft panel. Pressure cells were located directly below the rails at the subgrade-ballast interface.

No particular trends were observed in soil pressure data. In general, there was random variation in data between rails, sections, trains, and tests. However, the highest soil pressures measured in the three sections varied slightly. These were 35, 32, and 33 psi for Sections 4, 5, and 7, respectively. Individual pressures obtained for each section and each test are presented in Appendix D.

An inspection of the soil pressure cells after closure of track in June 1975 indicated some reasons for the erratic nature of the data. For example, the bottom of concrete panels were specified to be flat and to be supported on a 4- to 6-in. thick layer of ballast. However, as shown in Figure 41, the bottom was not flat and ballast adhered to the concrete during casting. Uneven pressures created by this condition plus the direct bearing of concrete on load cells, in some cases, resulted in wide differences in pressure readings. In addition, as shown in Figure 42, some pressure cells became displaced laterally and tilted during the project. As cells were initially installed level, tilting could have been caused by ballast leveling, concrete casting, weakening of the subgrade, or direct bearing of the concrete panels. Other examples of construction factors that affected pressure cell readings are shown in Figures 43 and 44. In Figure 43, one cell in Section 7 was separated from the concrete by a wooden wedge. Another cell, shown in Figure 44, had steel banding straps and concrete bearing directly on the cell. In general, construction procedures and mud pumping that developed during the project made it virtually impossible to obtain meaningful interface soil pressure data on the beam and slab sections.

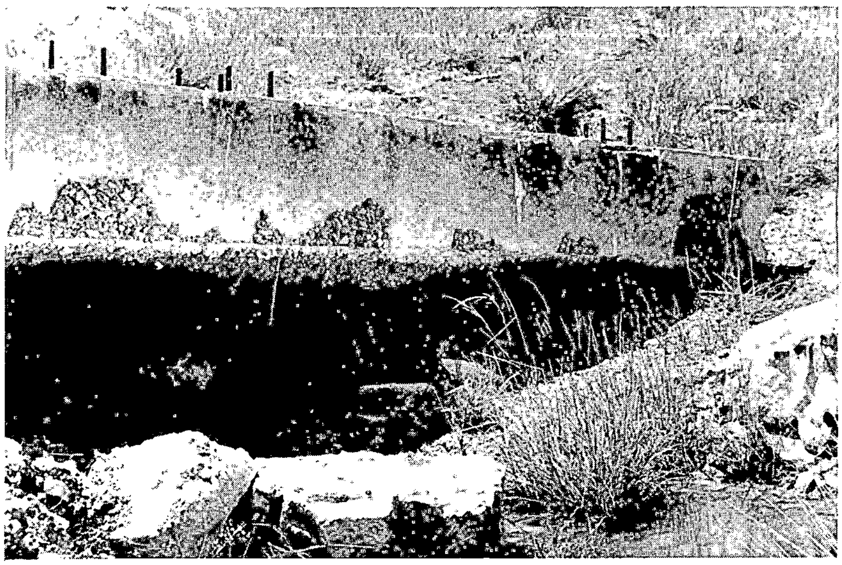


FIGURE 41 - VIEW OF CONCRETE PANEL REMOVED FROM TRACK

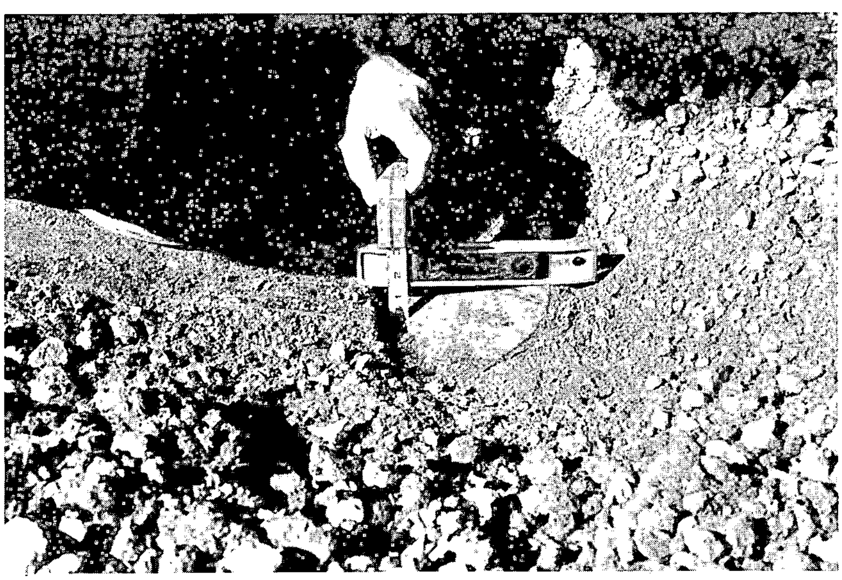
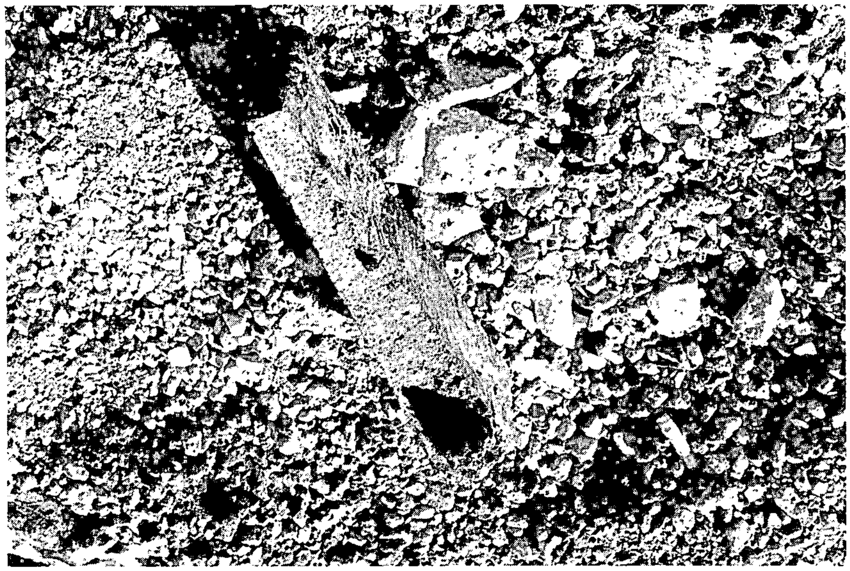
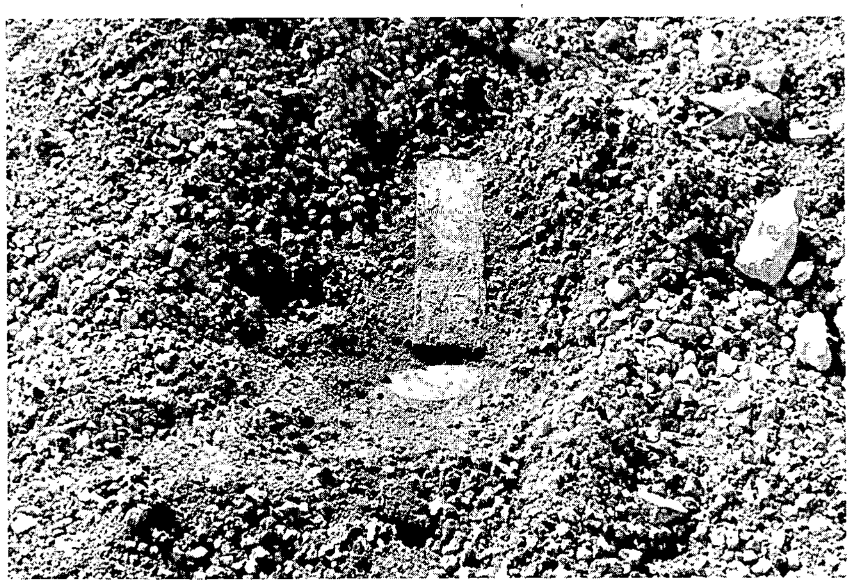


FIGURE 42 - VIEW OF SOIL PRESSURE CELL AFTER PROJECT TERMINATION



(a) Before wedge removal



(b) After wedge removal

FIGURE 43 - SEPARATION OF CONCRETE FROM LOAD CELL BY A WOODEN WEDGE

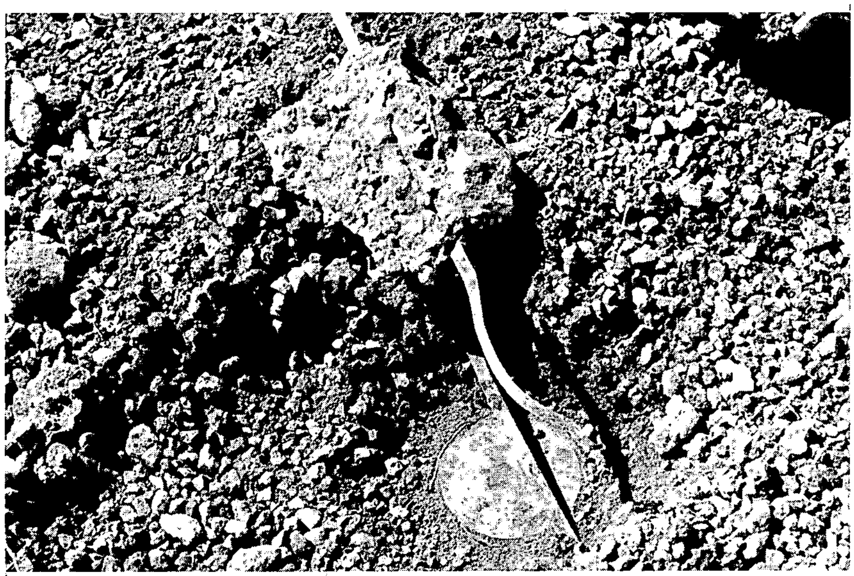


FIGURE 44 - BANDING STRAPS AND DIRECT BEARING OF CONCRETE ON LOAD CELL

Rail Stresses

Rail strains were measured at cross sections directly over the fastener and midway between two fasteners. Strain gage locations are shown in Figure 45. Stresses at the top and bottom surfaces of the rail were computed from strain gage readings on both sides of the rail. Stress computations were based on a linear strain relationship over the rail height.

Average values of the maximum stress for both rails, on the top of rail head and bottom of rail base are presented in Table 29 for a cross section at a fastener and in Table 30 for a cross section midway between two fasteners. These values were calculated from the strain average obtained from gages located each side of the rail for both rails. Individual stress readings at strain gage locations are presented in Appendix D. Maximum stresses at and between fasteners, for any side of the rail are shown in Table 31 for the different test sections.

In general, rail stresses for all three sections were about the same for each trip. Maximum stresses were 15 to 25% greater than average values.

Reinforcing Bar Stresses

Stresses in the top and bottom reinforcing steel bars were measured at the joint and midway between joints of the 10-ft panels. At each location two top and two bottom longitudinal reinforcing bars located under each rail were monitored.

Average of the maximum stresses in the top and bottom reinforcing bars, at the joint, are presented in Figures 46 and 47, respectively. Stresses in the top reinforcing bars at the main arrays generally increased with time. Stresses in the top reinforcing bars at the secondary array 5-1 changed slightly. No particular trend was observed in the bottom reinforcing bar stresses.

Average of the maximum stresses in the top and bottom reinforcing bars, midway between joints of the panel are presented in Figures 48 and 49, respectively. Top stresses increased with time in beam Sections 4 and 7 and remained unchanged in

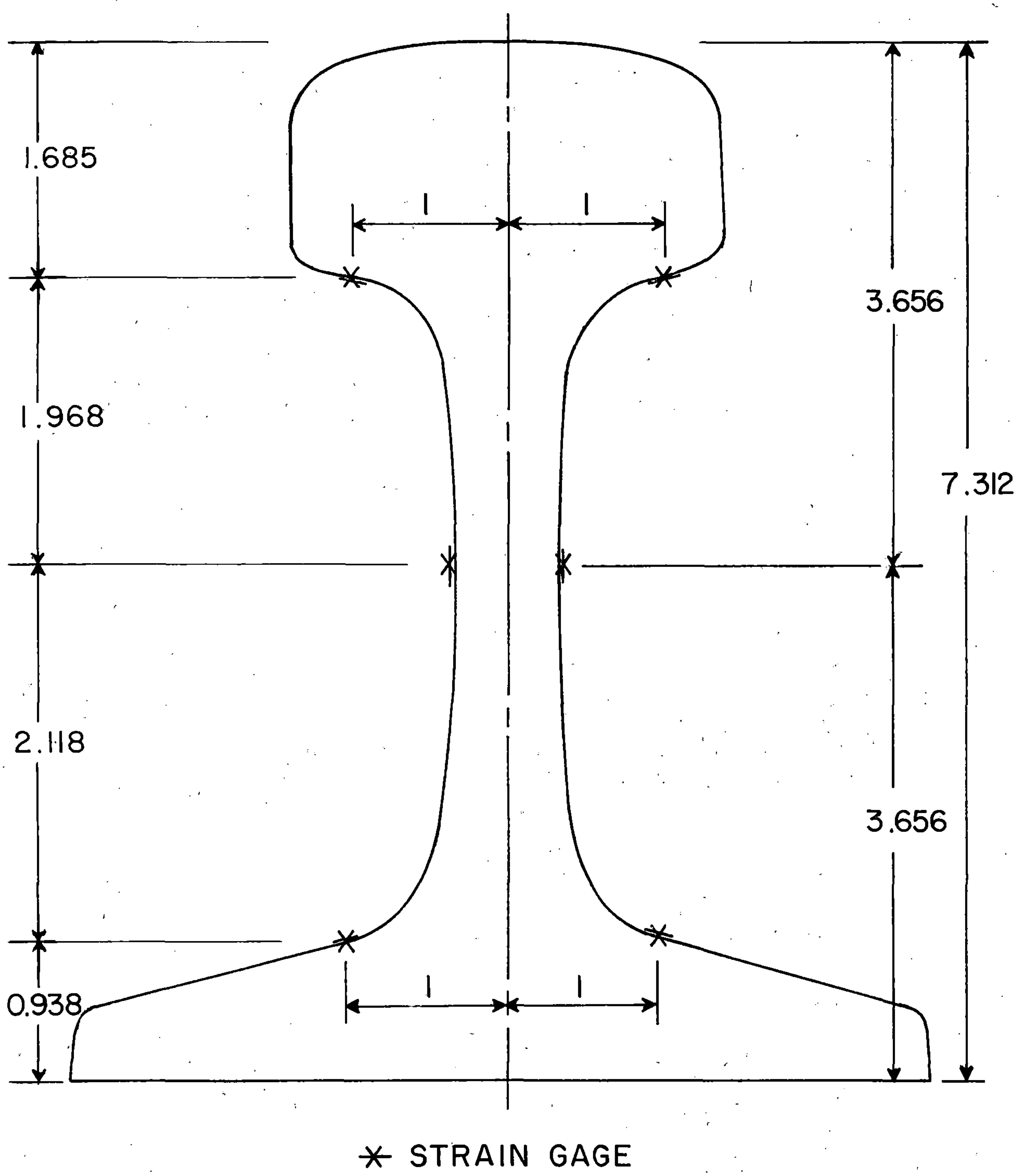


FIGURE 45 - LOCATION OF STRAIN GAGES ON RAIL CROSS SECTION
(All dimensions are in inches)

TABLE 29 - RAIL STRESSES AT FASTENERS*

Test Section	Stress on Top of Rail Head,(-) psi				Stress on Bottom of Rail Base,(+) psi			
	10/28/74		12/10/74	4/11/75	10/28/74		12/10/74	4/11/75
	Creep	30 mph	30 mph	50 mph	Creep	30 mph	30 mph	50 mph
4-2	9,500	9,900	12,400	5,900	5,900	7,500	9,600	3,400
5-1	14,100	13,600	8,000	8,200	10,200	10,400	5,500	6,500
5-2	-	-	9,200	5,800	-	-	7,400	5,100
7	14,600	14,200	9,800	8,100	10,700	11,200	7,000	6,900

*Values represent the average of maximum values for both rails.

(-) Values are compressive
(+) Values are tension

TABLE 30 - RAIL STRESSES BETWEEN FASTENERS*

Test Section	Stress on Top of Rail Head,(-) psi				Stress on Bottom of Rail Base,(+) psi			
	10/28/74		12/10/74	4/11/75	10/28/74		12/10/74	4/11/75
	Creep	30 mph	30 mph	50 mph	Creep	30 mph	30 mph	50 mph
4-2	11,100	9,300	9,800	6,400	7,900	6,700	8,100	5,500
5-1	11,000	10,000	8,400	6,300	7,400	7,000	6,200	5,300
5-2	-	-	9,500	6,900	-	-	7,600	5,900
7	11,500	10,600	10,300	6,100	8,300	7,600	7,800	4,200

*Values represent the average of maximum values for both rails.

(-) Values are compressive
(+) Values are tension

TABLE 31 - MAXIMUM RAIL STRESSES

Test Section	Rail Stress at Fastener, psi		Rail Stress between Fastener, psi	
	Top (-)	Bottom (+)	Top (-)	Bottom (+)
4	14,200	10,400	13,300	9,400
5	16,000	10,500	12,600	8,400
7	18,400	13,400	14,200	9,600

(-) All values are compression
(+) All values are tension

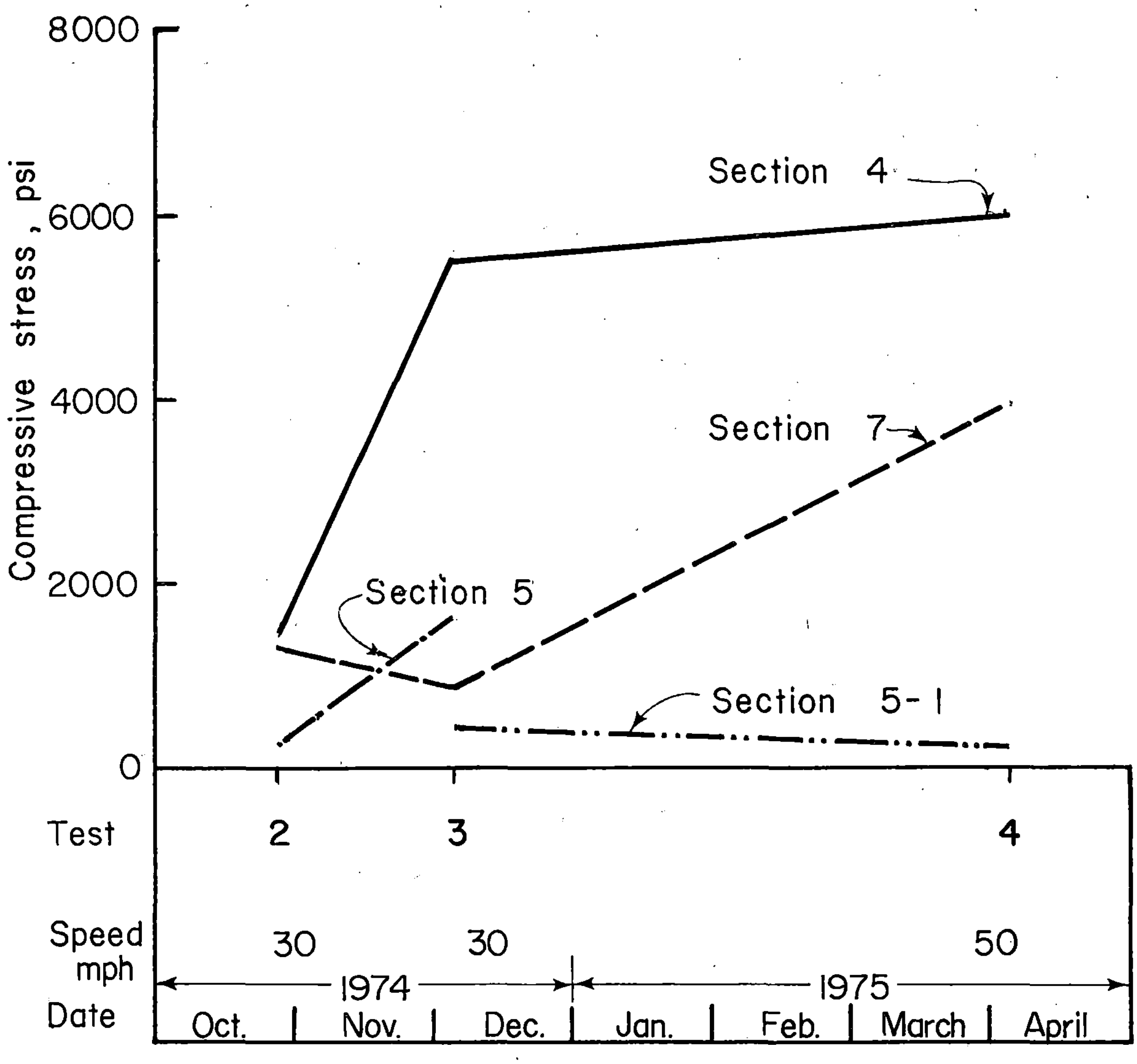


FIGURE 46 - STRESSES IN A TOP REINFORCING BAR AT A JOINT

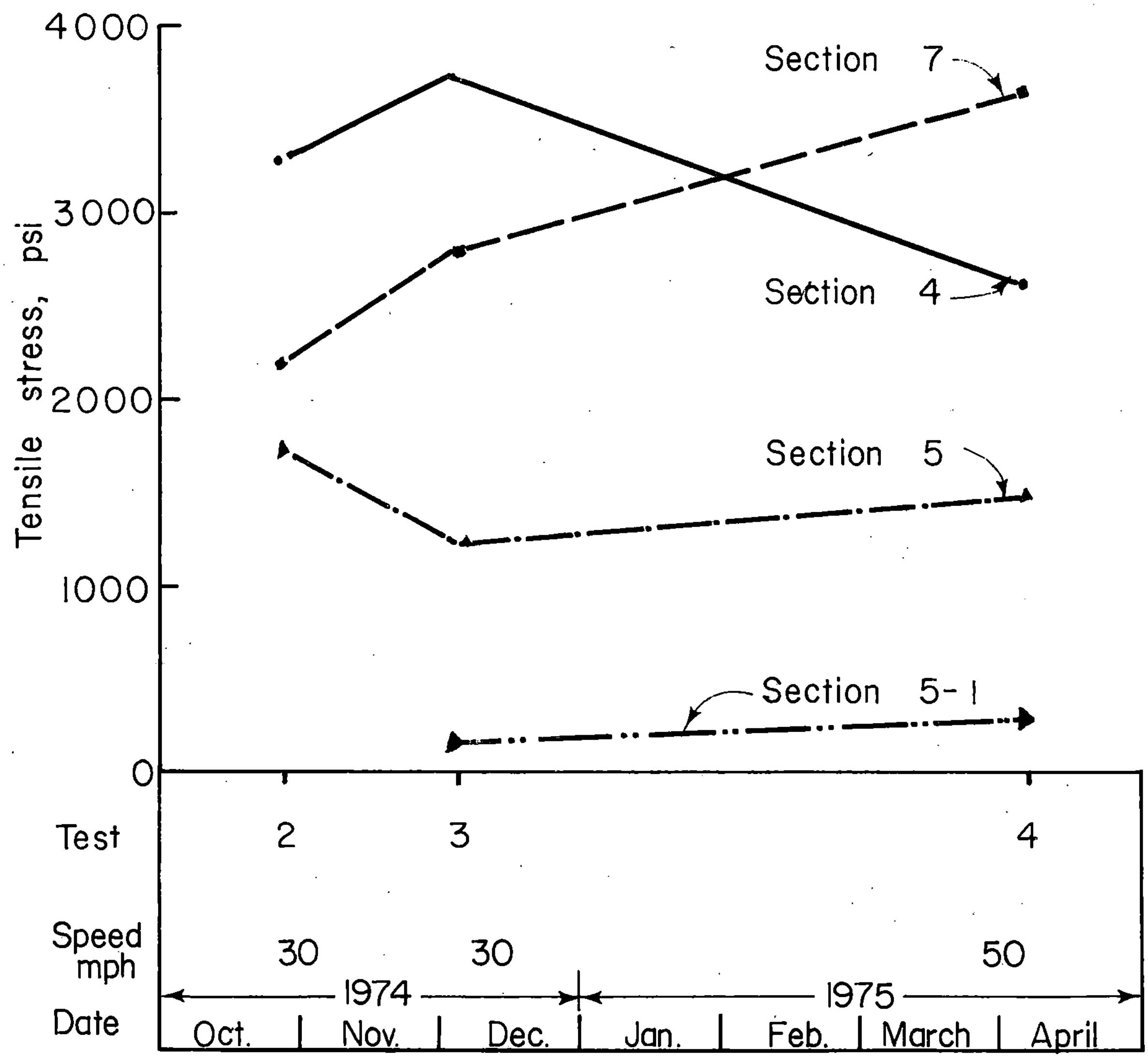


FIGURE 47 - STRESSES IN BOTTOM REINFORCING BAR AT A JOINT

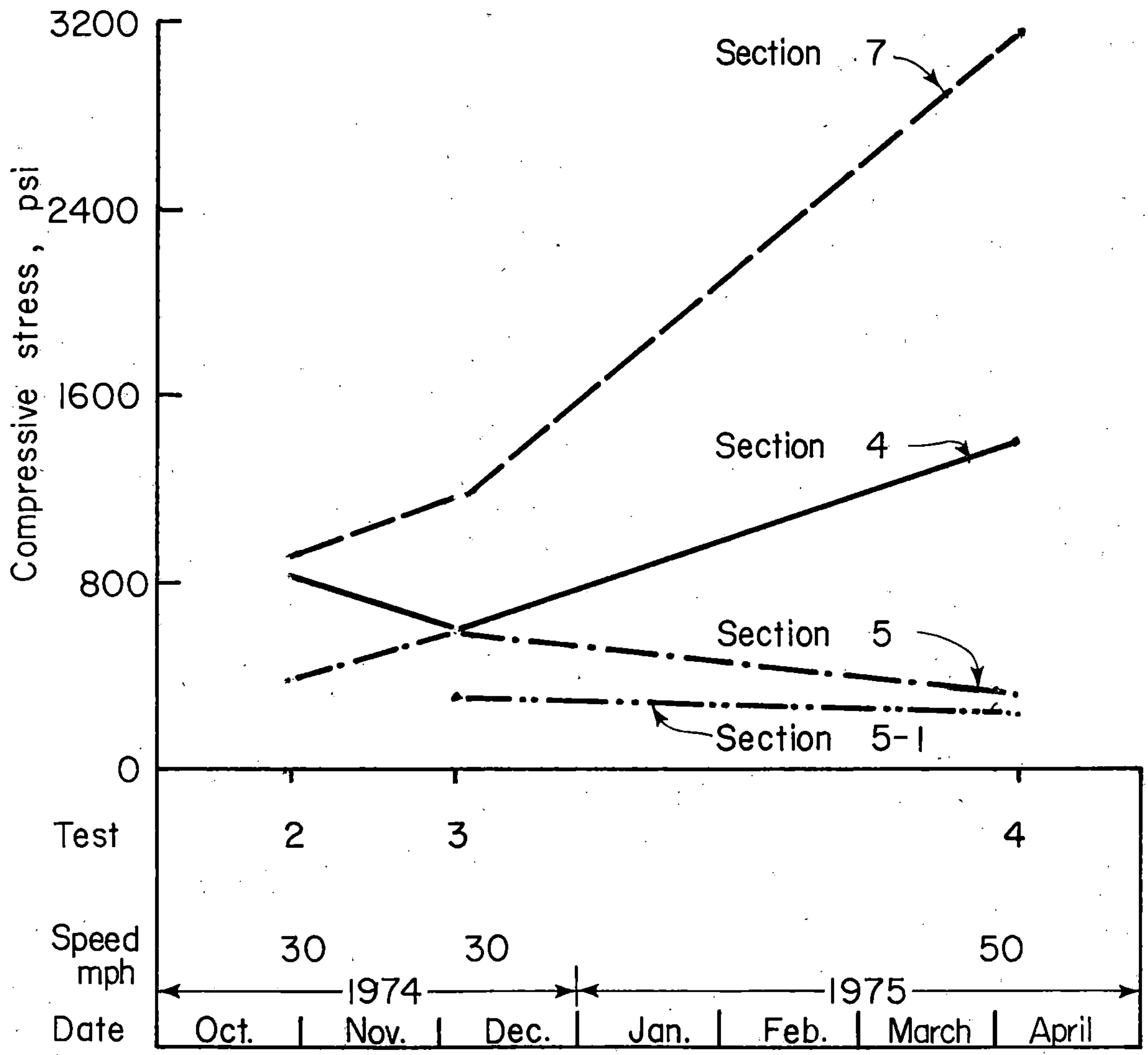


FIGURE 48 - STRESSES IN A TOP REINFORCING BAR AT MIDDISTANCE BETWEEN JOINTS

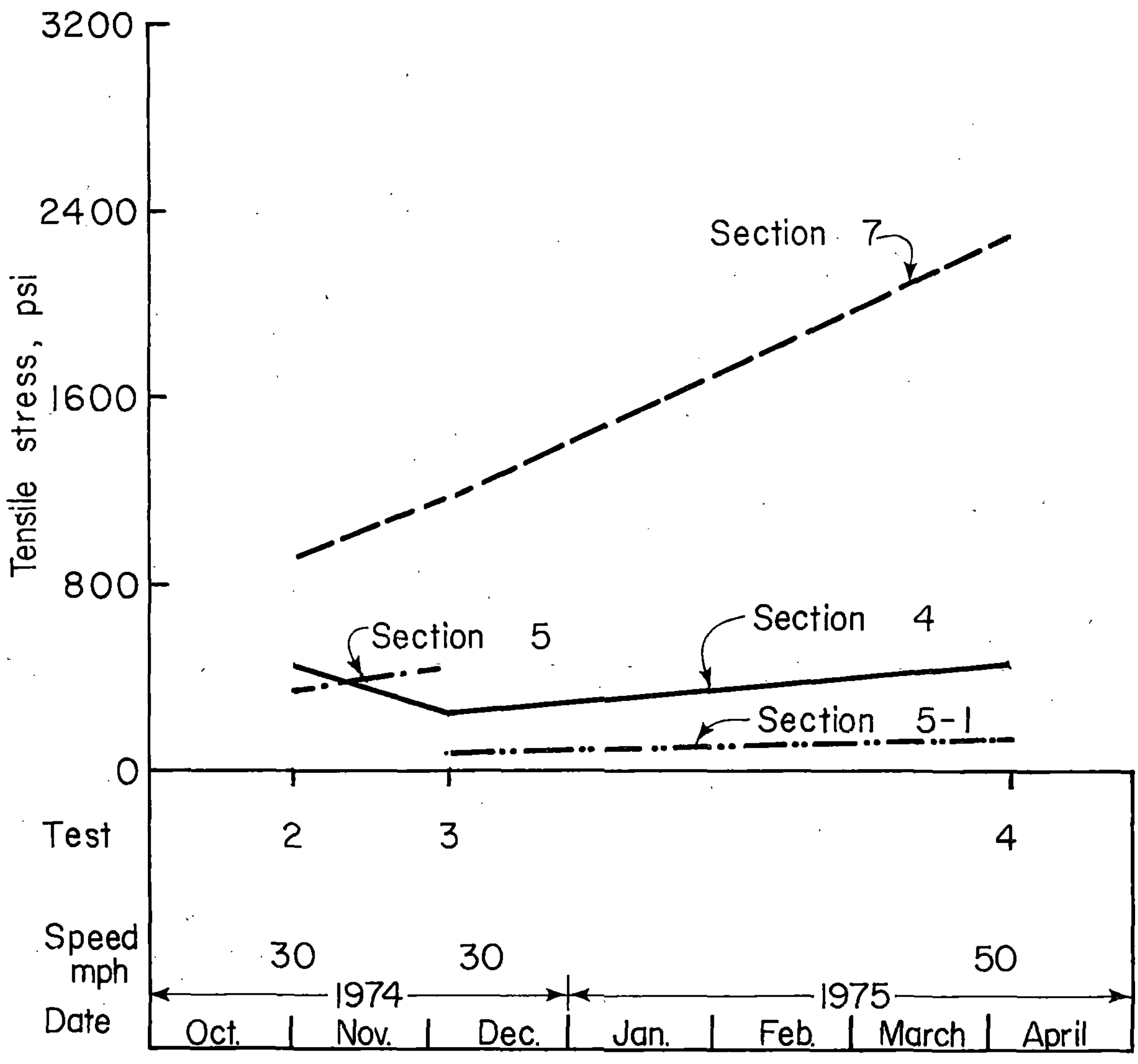


FIGURE 49 - STRESSES IN A BOTTOM REINFORCING BAR AT MIDDISTANCE BETWEEN JOINTS

slab Sections 5 and 5-1. Stresses in bottom reinforcing bars increased with time in Section 7, and remained relatively unchanged in all other sections.

In general, reinforcing bar stresses in beam sections were considerably higher than those in slab sections. Stresses in the secondary array Section 5-1 were consistently the lowest throughout the monitoring period. Individual stresses recorded for each rebar are presented in Appendix D.

Steel Stirrup Stresses

A limited number of strain gages were placed on the steel stirrup in Section 4-1. These gages were monitored during the December 1974 and April 1975 tests. These data, presented in Table 32, indicate that stresses increased between December 1974 and April 1975. All recorded stresses were very low.

Concrete Stresses

Stresses in the concrete were measured at the joint and midway between joints of the instrumented panels in Sections 4, 5, 5-1, and 7. Stresses midway between joints were monitored with strain gages located longitudinally at the top, middle, and bottom of the concrete directly below each rail. Stresses at joints were monitored with strain gages located transversely at the top and bottom of the concrete directly below each rail. Middle and bottom gages were embedded in the concrete during construction in 1972. Top gages were installed in 1973 and were replaced periodically during the project.

Concrete stresses were generally low except in a few cases. Highest compressive stresses were 140, 80, 56, and 289 psi in Sections 4, 5, 5-1, and 7, respectively. Highest tensile stresses were 404, 77, 56, and 257 psi in Sections 4, 5, 5-1, and 7, respectively. These stresses were calculated assuming a concrete modulus of elasticity of 4 million psi. Individual stresses recorded at each strain gage location are presented in Appendix D.

TABLE 32 - STEEL STIRRUP STRESSES

Beam	Beam Side	Stress at Top, psi (+)		Stress at Bottom, psi (-)	
		12/10/74	4/11/75	12/10/74	4/11/75
North	North	145	-	131	-
	South	167	273	116	228
South	North	167	505	123	327
	South	123	610	116	352
Average		151	463	122	302

(+) All values are tension
(-) All values are compression

A computer analysis of beam and slab sections[10] determined that maximum concrete stress would occur at each end of Section 4 in the transition between tie track sections and the beam section. For this reason strain gages were placed on the two beams adjoining Section 3, midway between joints. These beams are designated north and south beams. Gages were placed longitudinally near the top and bottom, and at a 45 degree angle at mid-depth on both sides of the beams. Monitoring of gages was scheduled for December 1974 and April 1975 tests. During the December 1974 test it was determined that most gages were damaged probably during ballast retamping. Because of weather conditions and testing schedule gages were not replaced until the April 1975 test.

Concrete stresses obtained in April 1975 for Section 4 are presented in Table 33. Although compressive stresses in Section 4 were considerably higher than those obtained at the main array, 4, tensile stresses varied only slightly. Maximum tensile and compressive stresses were 428 and 289 psi, respectively. These values occurred on the south side (field side) of the south panel.

As part of this study, beam deflections were also measured. Deflection obtained in December 1974 and April 1975 were of similar magnitude to those measured at the main array. Deflections measured at Section 4-1 in December 1974 and May 1975 were, respectively, 25% higher and 17% lower than those measured at the same time for main array Section 4.

Gage Bar Stresses

Stress in threaded gage bars that connected the north and south concrete beams in Section 4-1 were monitored at two locations during the December 1974 and April 1975 tests. Stress in one bar could not be obtained in December 1974 because the lead wires were damaged during ballast retamping.

Tensile stress in the other gage bar in December 1974 was 892 psi. Tensile stresses in the two bars in April 1975 were 1,251 and 7,194 psi.

TABLE 33 - CONCRETE STRESSES AT SECTION 4

Beam	Beam Side	Concrete Stress, psi		
		Top	Middle	Bottom
North	North	-120	0	+108
	South	-130	-77	+103
South	North	-208	+51	+230
	South	-289	-	+428
Average		-187	-13	+217

(-) Indicates compressive stress
(+) Indicates tensile stress

Joint Opening

Joint width was measured during October and December 1974 and April 1975 tests. Reference readings were taken on April 23, 1973.

Changes in joint width obtained during each trip are presented in Table 34. The maximum recorded change in joint width from April 1973 to April 1975 was 0.0277 in.

Beam and Slab Settlement

Beam and slab elevation readings were taken during October and December 1974 and April 1975. Reference values were taken on April 23, 1973.

Elevation changes recorded between trips are presented in Table 35. These data indicate that instrumented panels in Sections 4, 5, and 5-1, settled approximately 2 in., 1 in. and 0.50 in., respectively, between October 1974 and April 1975. However, the instrumented panel in Section 7 settled 0.5 to 1 in. between October and December 1974 and then rose nearly 2 in. between December 1974 and April 1975.

Effect of Train Speed

During the October 1974 trip, data were obtained for creep and 30 mph speeds. In most cases the difference in speed had a minor effect on the deflection response of beam and slab sections. Beam and slab deflections at creep speeds were approximately 20% higher than those at 30 mph. Corresponding fastener deflections at creep speed were 10 to 20% higher than those at 30 mph.

TABLE 34 - CHANGE IN JOINT OPENING

Test Section	Reading Location	Change in Joint Opening, in.*		
		10/28/74	12/10/74	4/11/75
4	North	*	**	**
	South	*	**	**
4-1	North	+0.0022	+0.0066	+0.0150
	South	*	**	**
5	North	+0.0108	+0.0128	+0.0009
	South	+0.0106	+0.0112	+0.0005
5-1	North	+0.0092	+0.0202	+0.0267
	South	+0.0090	+0.0195	+0.0277
7	North	+0.0004	-0.0027	-0.0050
	South	+0.0006	-0.0045	-0.0038

*All values are relative to joint opening on April 23, 1973, (+) indicates increase in joint opening, (-) indicates decrease in joint opening.

**Reference points were destroyed during construction and maintenance.

TABLE 35 - CHANGE IN ELEVATION

Test Section	Reading Location	Change in Elevation, in.*		
		10/28/74	12/10/74	4/11/75
4	North	**	**	-0.270***
	South	**	**	-0.111***
4-1	North	+0.155	+0.676	+2.255
	South	*	**	**
5	North	+0.189	+0.037	+1.061
	South	+0.217	+0.096	+0.401
5-1	North	+0.254	+0.116	+0.362
	South	+0.213	+0.181	+0.136
7	North	+0.355	-1.341	-0.637
	South	+0.407	-0.951	-0.928

*All values are relative to joint opening on April 23, 1973, (+) indicates increase in joint opening, (-) indicates decrease in joint opening.

**Reference points were destroyed during construction and maintenance.

SUMMARY AND FINDINGS

In 1971-72 a 1.8 mile test track containing concrete tie, slab, beam, and wooden tie test sections was constructed on the Atchison, Topeka, and Santa Fe Railway Company mainline near Aikman, Kansas. Test sections were instrumented with load sensors, soil pressure cells, deflection meters, strain gages, and accelerometers.

Part of the experimental program was designed to monitor changes in track response so that the effect of time and traffic on the performance of track structure components would be determined. A time schedule of observations was established to correspond to an expected gradual deterioration. Data were scheduled to be gathered on a quarterly basis for the first year of traffic and once each year thereafter for three years. However, the track was closed and testing terminated after approximately six months of service. This action was taken due to subgrade failure that resulted in excessive deflections and mud pumping throughout most of the test sections.

The high rate of subgrade deterioration prevented establishment of the initial steady base of data intended in the first year. However, the data gathered provide some information on relative performance.

Tie Track Sections

Rail stress was not dependent on tie spacing. Maximum rail stress measured on any of the concrete tie sections was within 20% of the value for timber tie track. No definite trends were evident.

Concrete tie track, even with 30-in. tie spacing, was stiffer than conventional timber tie track. The track modulus of track with concrete ties at 27-in. spacing was substantially greater than that of standard ASTF track with timber ties at about 20-in. spacing. The track modulus of stabilized timber tie track was less than that of standard timber tie track.

Longitudinal rail movements measured during passage of traffic indicated that concrete ties were better than timber ties at restraining rail movement and creep.

Rail and tie acceleration measurements generally indicated that rail acceleration was greater than tie acceleration. Data trends are masked by variable conditions of track support between the two measurement times.

Load on the rail seat of concrete ties increased as tie spacing increased. Reasonable agreement existed between data from rail seat measurements and calculated values based on track modulus. Measured rail seat force was approximately 33% of passing wheel load for the standard timber tie track. The average value was 40% for the concrete ties at 27-in. spacing on 10-in. deep ballast, and only slightly less for 15-in. ballast depth.

Bending moments measured on concrete ties were never more than 2/3 of the established cracking moment. However, visual examination of ties showed bottom tensile cracks at most rail seats at the end of traffic.

Ballast pressure under ties was initially highest directly under the rail seats, but gradually tended toward uniform distribution along the length of tie as time progressed. Load-cell tie data indicated the largest recorded pressures were 60.2 psi for the wooden tie section and 50.4 psi for the concrete tie sections.

Impact factor due to speed was not dependent on track modulus.

Laboratory tie tests verified that the ties met: (a) 1971 AREA specifications in effect when the ties were produced and installed in track, and (b) 1975 AREA specifications except positive rail seat strength. Both unused ties and cracked ties from Sections 1 and 8 met 1971 AREA repeated load and bond development requirements. Two concrete ties withstood 8.25 million cycles each without any visible sign of damage. In the wooden tie test, the top rail seat area became spongy, spikes loosened, and one steel plate cracked.

Beam and Slab Sections

Maximum concrete tensile stress recorded in the slab section was 77 psi compared to a maximum value of 404 psi in the beam sections. All recorded concrete compressive stresses were less than 300 psi. Reinforcing steel stresses were small. A maximum reinforcing steel stress of 8,200 psi was recorded during the last test period. Prior to subgrade softening the maximum reinforcing steel stress was only 3,500 psi.

Track modulus for all three sections generally decreased with time. Track modulus for the slab section decreased from approximately 16,100 to 4,600 lb/in./in. while modulus of cast-in-place and precast beam sections decreased from 9,300 to 3,200 lb/ in./in. and 16,000 to 1,700 lb/in./in., respectively.

Deflections of slab and beam test sections at main instrumentation locations increased significantly from track opening to the final test period. Maximum slab deflections increased from 0.023 in. to 0.068 in. while deflections of cast-in-place and precast beams increased from 0.046 in. to 0.138 in. and 0.049 in. to 0.181 in., respectively.

In contrast, slab and beam deflections at secondary test locations away from the heavily instrumented main test locations remained comparatively small and did not increase more than 30% during the recording period. Maximum deflections at secondary test locations in the slab and cast-in-place beam sections were only 0.038 in. and 0.074 in., respectively. A comparison of deflections at main and secondary test locations indicates that the use of numerous subgrade instrumentation at one location and subgrade softening weakened the support at the main test locations significantly.

Maximum vertical rail movement relative to the slab and beam surfaces varied between 0.020 and 0.045 in. Variations in rail movement between sections and test periods was due primarily to adjustments in fastener vertical alignment.

Soil pressure readings varied considerably during the project. No particular trends were observed. Highest soil pressures were approximately the same for all three sections. Maximum recorded pressure was 35 psi.

Rail stresses both at and between fasteners were approximately the same for slab and beam sections. Average rail stress at the fastener was 9,550 psi with a 30% coefficient of variation for the maximum stress recorded for all sections. Similarly, the maximum stress between fasteners was 8,510 psi with a 21% coefficient of variation. Maximum recorded rail stress was 18,400 psi.

The beam and slab replacement anchoring system performed perfectly. Pullout capacities for each of the new anchor bolts exceeded 30,000 lb compared to 4,800 lb for the original anchor bolts. No stud pullouts were observed with the new replacement anchor bolt system.

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