NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT 21

DETECTING VARIATIONS IN LOAD-CARRYING CAPACITY OF FLEXIBLE PAVEMENTS

HIGHWAY RESEARCH BOARD NATIONAL ACADEMY OF SCIENCES - NATIONAL RESEARCH COUNCIL

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DETECTING VARIATIONS IN LOAD-CARRYING CAPACITY OF FLEXIBLE PAVEMENTS

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
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WITH THE BUREAU OF PUBLIC ROADS

SUBJECT CLASSIFICATION:
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, United States Department of Commerce.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

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DYNAMIC RESPONSE OF PAVEMENT DUE TO IMPULSIVE LOAD (STEEL STRIKER PLATE)

The dynamic displacement transducer (DDT) and the fixed displacement transducer (FDT) were fastened to the pavement. Oscilloscope and oscillograph records were obtained for a 2-ft drop height using a 12-in. diameter by 1-in. thick steel striker plate.

Displacement

The oscilloscope and oscillograph displacement responses of the pavement from the DDT and FDT are shown in Figure B-2. The shape of the responses shows good agreement.

Impulse Duration

The trace in Figure B-2 appears to indicate intermittent contact between the bar and the striker plate. Because the trace does not show a clean impulse duration, the circuit was improved by using a faster galvanometer and by replacing banana-type contacts with screw-type contacts. The striker plate was also replaced by a 15-in. diameter by 1-in. thick aluminum plate. The result of the change is discussed in the following.

DYNAMIC RESPONSE OF THE PAVEMENT DUE TO IMPULSIVE LOAD (ALUMINUM STRIKER PLATE)

The displacement response of the pavement using a 15-in. diameter by 1-in. thick aluminum striker plate and a 1-ft drop height as monitored by the FDT and DDT were obtained with the transducers at 18 in., 34 in., and 51 in. from the center of the striker plate.

Displacement

The displacement responses using the aluminum striker plate are shown in Figure B-3. The response at 18 in. from the center shows a maximum deflection of about 0.04 in. and a frequency of about 30 cycles per second for a 1-ft drop height. On the other hand, the responses at 34 in. and 51 in. from the center of impact show a larger first minimum. The possible explanations for this phenomenon are being studied.

In comparing the response of the DDT to the response of the FDT (Fig. B-3) it is found that the response of the DDT is close to that of the FDT up to the first peak and diverges after that due to the very soft suspension of the DDT. At a point far (51 in.) removed (Fig. B-3c) the shapes of the two responses are very close to each other.

From the foregoing results the following preliminary conclusions are obtained:

1. The displacement parameter shows good promise, partly because of the observed 0.04-in. maximum deflection, as a criterion for determining the seasonal variation of the load-carrying capacity of flexible pavements.

2. The dynamic displacement transducer (DDT) is a valid sensing device, within the range previously discussed, in the measure of dynamic displacements.

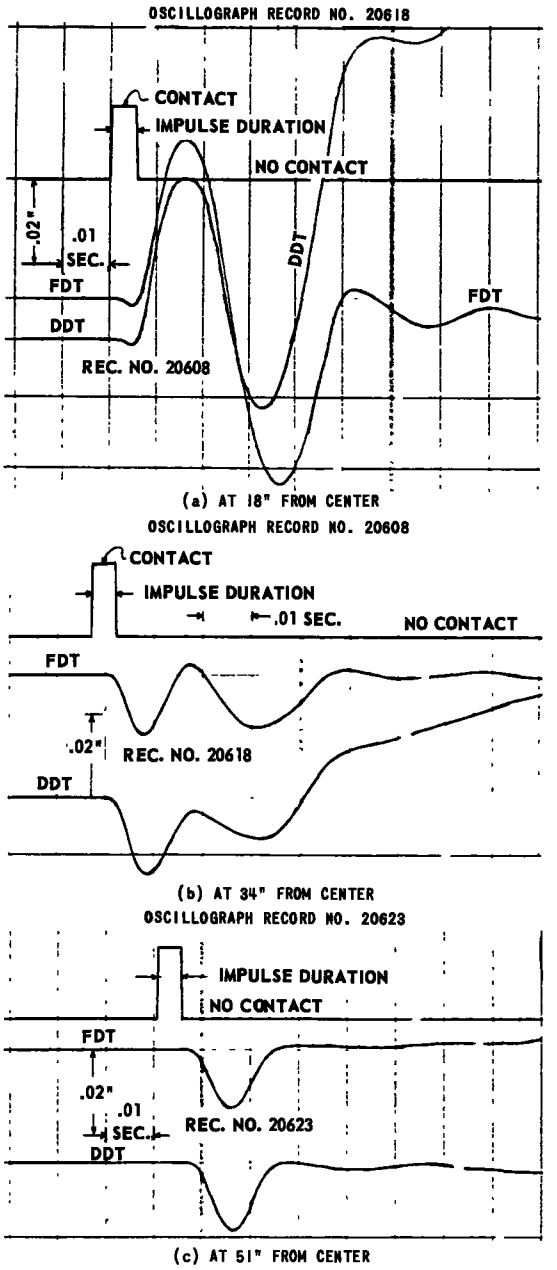

Figure B-3. Dynamic response of pavement due to impulsive load; aluminum striker plate, drop height 1 ft.

Impulse Duration

The response of the circuit for impulse duration shows a much cleaner contact and loss of contact (bounce) as shown in Figure B-3. Similar clean traces (not shown) are also observed for the second and third impacts. These clean traces are due to an improvement in circuitry and change of striker plate.

EQUIPMENT

The dynamic displacement transducer and the road impulse generator were constructed on a developmental basis;

i.e., by the use of rough sketches, fabrication, testing, and modifications. The following materials were used:

1. Dynamic displacement transducer (see Fig. 8):

   Housing: Bored from a solid 5-in. diameter aluminum rod.
   Suspended rod: ¼-In. diameter brass.
   Attached mass: Brass.
   Suspension spring: Steel, giving a natural frequency of 1.7 cps.
   LVDT coil: Schaevitz ± 0.06 in.
   Ball bushings: ¼-In. diameter.
   Battery: 9v.

2. CAL experimental gravity-type road impulse generator (see Fig. 6):

   Tripod legs and braces: 3-In. steel channels.
   Housing: 5-In. steel pipe.
   Guide bearings: Machined from steel stock.
   Pulley support: 2×2-In. angles.
   Winch: 1,000-lb capacity.
   Cable: 1,500-lb capacity, stainless steel.
   Trailer: Kohler model A-264325.

3. Commercial devices:

   Recorder: CEC recording oscillograph type 5-114.
   Oscilloscope: Tektronix type 502A.
   Galvanometers: CEC types 315 and 317.
   Camera: Polaroid Land.
   Batteries: 12 v.
   Velocity pickups: CEC 4-102A.

APPENDIX C

DATA FROM FIELD TESTS

A breakdown of the response data (each value is the average of three drops) from the field tests is given in Tables C-1 through C-4. Tables C-1 and C-2 are for the poor road, Tables C-3 and C-4 for the good road. It should be noted that on 11/12/64 some data were taken at Stations 1 and 2A but later were discarded because they were found to be faulty. The dispersion of variability of the data for the three drops showed a range of approximately ± 10%, indicating good repeatability.

TABLE C-1
FIELD TEST DATA FOR TEST STATION 1A OF POOR ROAD

Response \ Date		10/15	10/30	11/6	11/12	11/25	12/14	3/8	3/31	4/8	4/20	4/26	5/13	6/3	6/16
DDT First Peak Deflection	c	.0303	.0321	.0298		.0345		.0411	.0505	.0491	.0540	.0530	.0463	.0398	.0424
	m	.0360	.0326	.0309		.0329		.0426	.0478	.0493	.0542	.0502	.0479	.0412	.0392
	e	.0486	.0503	.0455		.0474		.0513	.0617	.0622	.0705	.0634	.0550	.0428	.0474
	Σ	.1149	.1150	.1062		.1148		.1350	.1600	.1606	.1787	.1666	.1492	.1238	.1290
	Av.	.0383	.0383	.0354		.0383		.0450	.0533	.0535	.0596	.0555	.0497	.0413	.0430
Corrected DDT Deflection	c	.0294	.0250	.0226		.0279		.0344	.0386	.0416	.0401	.0402	.0389	.0317	.0368
	m	.0292	.0292	.0260		.0220		.0390	.0419	.0414	.0408	.0367	.0323	.0317	.0312
	e	.0371	.0373	.0386		.0323		.0420	.0438	.0474	.0628	.0513	.0435	.0310	.0404
	Σ	.0957	.0915	.0872		.0822		.1154	.1243	.1304	.1437	.1282	.1147	.0944	.1084
	Av	.0319	.0305	.0291	No Data Taken	.0274	No Data Taken	.0385	.0414	.0435	.0479	.0427	.0382	.0315	.0361
Rebound Time	c	.164	.165	.152		.164		.143	.145	.157	.165	.168	.178	.180	.179
	m	.132	.154	.139		.166		.129	.153	.165	.171	.175	.177	.176	.188
	e	.144	.162	.162		.161		.150	.144	.171	.169	.178	.166	.169	.179
	Σ	.440	.481	.453		.491		.422	.442	.493	.505	.521	.521	.525	.546
	Av	.147	.160	.151		.164		.141	.147	.164	.168	.174	.174	.175	.182
Impulse Duration	c	.0045	.0041	.0044		.0044		.0047	.0048	.0051	.0051	.0049	.0037	.0047	.0046
	m	.0038	.0038	.0044		.0049		.0048	.0045	.0050	.0048	.0045	.0046	.0047	.0046
	e	.0042	.0047	.0047		.0046		.0049	.0053	.0057	.0052	.0050	.0051	.0050	.0049
	Σ	.0125	.0126	.0135		.0139		.0144	.0146	.0158	.0151	.0144	.0134	.0144	.0141
	Av.	.0042	.0042	.0045		.0046		.0048	.0049	.0053	.0050	.0048	.0045	.0048	.0047
Wave Propagation Time	c										.0029	.0057	.0051	.0046	.0027
	m										.0029	.0020	—	.0043	.0033
	e					1964 ←	→		1965		.0060	.0023	.0043	.0049	.0043
	Σ										.0118	.0100	—	.0138	.0103
	Av.										.0039	.0033	—	.0046	.0034

TABLE C-2
FIELD TEST DATA FOR TEST STATION 1 OF POOR ROAD

Response \ Date		10/15	10/30	11/6	11/12	11/25	12/14	3/8	3/31	4/8	4/20	4/26	5/13	6/3	6/16
DDT First Peak Deflection	c	.0306	.0392	.0398	Wrong Electrical Connections Made Incomplete Data Discarded	.0453	No Data Taken	.0440	.0561	.0617	.0615	.0607	.0512	.0489	.0430
	m	.0332	.0368	.0327		.0413		.0466	.0592	.0518	.0634	.0520	.0464	.0448	.0410
	e	.0395	.0427	.0399		.0456		.0501	.0617	.0651	.0742	.0587	.0527	.0573	.0414
	Σ	.1033	.1187	.1124		.1322		.1407	.1770	.1786	.1991	.1714	.1503	.1510	.1254
	Av.	.0344	.0396	.0375		.0441		.0469	.0590	.0595	.0664	.0571	.0501	.0503	.0418
Corrected DDT Deflection	c	.0233	.0278	.0253		.0303		.0337	.0483	.0459	.0601	.0553	.0366	.0417	.0356
	m	.0310	.0323	.0232		.0244		.0384	.0375	.0414	.0484	.0343	.0323	.0304	.0300
	e	.0371	.0306	.0298		.0283		.0456	.0471	.0494	.0658	.0578	.0465	.0513	.0392
	Σ	.0914	.0907	.0783		.0830		.1177	.1329	.1367	.1743	.1474	.1154	.1234	.1048
	Av	.0305	.0302	.0261		.0277		.0392	.0443	.0456	.0581	.0491	.0385	.0411	.0349
Rebound Time	c	.169	.142	.140		.152		.103	.119	.127	.124	.127	.138	.149	.157
	m	.124	.125	.121		.143		.111	.111	.130	.139	.148	.156	.163	.180
	e	.167	.171	.164		.166		.136	.121	.143	—	.152	.155	.157	.189
	Σ	.460	.438	.425		.461		.350	.351	.400	—	.427	.449	.469	.526
	Av.	.153	.146	.142		.154		.117	.117	.133	—	.142	.150	.156	.175
Impulse Duration	c	.0046	.0050	.0046		.0045		.0052	.0049	.0043	.0048	.0044	.0045	.0048	.0046
	m	.0046	.0052	.0051		.0049		.0054	.0054	.0063	.0063	.0052	.0052	.0054	.0052
	e	.0046	.0048	.0051		.0043		.0050	.0048	.0057	—	.0047	.0050	.0052	.0049
	Σ	.0138	.0150			.0137		.0156	.0151	.0163	—	.0143	.0147	.0154	.0147
	Av	.0046	.0050	.0049		.0046		.0052	.0050	.0054	—	.0048	.0049	.0051	.0049
Wave Propagation Time	c										.0039	.0052	.0048	—	.0023
	m										.0040	—	.0057	.0052	.0036
	e					1964 ←	→		1965		.0045	.0037	.0045	.0038	.0039
	Σ										.0124	—	.0150	—	.0098
	Av										.0041	—	.0030	—	.0033

TABLE C-3
FIELD TEST DATA FOR TEST STATION 2A OF GOOD ROAD

Response \ Date		10/15	10/30	11/6	11/12	11/25	12/14	3/8	3/31	4/8	4/20	4/26	5/13	6/3	6/16
DDT First Peak Deflection	c	No Data Taken	.0137	.0116	Wrong Electrical Connections Made Incomplete Data Discarded	No Data Taken	.0133	.0139	.0147	.0171	.0146	.0166	.0201	.0197	.0181
	m		.0146	.0100			.0127	.0154	.0174	.0180	.0146	.0156	.0232	.0218	.0191
	e		.0172	.0145			.0166	.0273	.0232	.0250	.0195	.0244	.0358	.0310	.0277
	Σ		.0455	.0361			.0426	.0566	.0553	.0601	.0487	.0566	.0791	.0725	.0649
	Av		.0152	.0120			.0142	.0189	.0184	.0200	.0162	.0189	.0264	.0242	.0216
Corrected DDT Deflection	c		.0114	.0106			.0116	.0123	.0121	.0145	.0138	.0150	.0188	.0172	.0176
	m		.0122	.0092			.0112	.0121	.0148	.0147	.0132	.0148	.0217	.0198	.0180
	e		.0140	.0126			.0141	.0217	.0202	.0206	.0194	.0205	.0347	.0252	.0248
	Σ		.0376	.0324			.0369	.0461	.0471	.0498	.0464	.0503	.0752	.0622	.0604
	Av		.0125	.0108			.0123	.0154	.0157	.0166	.0155	.0168	.0251	.0207	.0201
Rebound Time	c		.171	.190			.210	.160	.198	.201	.203	.209	.195	.215	.222
	m		.171	.162			.182	.154	.159	.171	.178	.173	.183	.183	.190
	e		.168	.156			.182	.121	.142	.153	.164	.161	.161	.163	.172
	Σ		.510	.508			.574	.435	.499	.525	.545	.543	.539	.561	.584
	Av		.170	.169			.191	.145	.166	.175	.182	.181	.180	.187	.195
Impulse Duration	c		.0031	.0044			?	.0035	.0037	.0041	.0035	.0037	.0040	.0040	.0040
	m		.0031	.0027			.0029	.0028	.0031	.0033	.0032	.0034	.0038	.0037	.0040
	e		.0021	.0027			.0024	.0029	.0028	.0030	—	.0030	.0038	.0035	.0037
	Σ		.0083	.0098				.0092	.0096	.0104	—	.0101	.0116	.0112	.0117
	Av		.0028	.0033				.0031	.0032	.0035	—	.0034	.0039	.0037	.0039
Wave Propagation Time	c										.0019	.0020	—	.0030	.0021
	m										.0028	.0019	.0025	.0045	.0016
	e					1964 ←	→		1965		.0012	.0015	.0019	.0017	.0016
	Σ										.0059	.0054	—	.0092	.0053
	Av										.0020	.0018	—	.0031	.0018

TABLE C-4
FIELD TEST DATA FOR TEST STATION 2 OF GOOD ROAD

Response \ Date		10/15	10/30	11/6	11/12	11/25	12/14	3/8	3/31	4/8	4/20	4/26	5/13	6/3	6/16
DDT First Peak Deflection	c		.0119	.0111			.0119	.0160	.0146	.0158	.0128	.0135	.0177	.0159	.0140
	m		.0139	.0109			.0113	.0156	.0136	.0145	.0118	.0131	.0184	.0186	.0161
	e		.0160	.0141			.0131	.0252	.0204	.0209	.0192	.0208	.0349	.0294	.0243
	Σ		.0418	.0361			.0363	.0568	.0486	.0512	.0438	.0474	.0710	.0639	.0544
	Av		.0139	.0120			.0121	.0189	.0162	.0171	.0146	.0158	.0237	.0213	.0181
Corrected DDT Deflection	c		.0107	.0083			.0105	.0144	.0128	.0129	.0115	.0112	.0163	.0124	.0123
	m		.0128	.0103			.0100	.0139	.0121	.0120	.0106	.0123	.0173	.0160	.0141
	e		.0138	.0137			.0114	.0190	.0165	.0176	.0181	.0177	.0307	.0244	.0233
	Σ		.0373	.0323			.0319	.0473	.0414	.0425	.0402	.0412	.0643	.0528	.0497
	Av		.0124	.0108			.0106	.0158	.0138	.0142	.0134	.0137	.0214	.0176	.0166
Rebound Time	c	No Data Taken	.194	.188	No Data Taken	No Data Taken	.208	.162	.192	.208	.211	.218	.229	.237	.247
	m		.164	.151			.176	.152	.161	.170	.169	.172	.204	.206	.203
	e		.158	.147			.179	.138	.137	.151	.154	.149	.167	.174	.170
	Σ		.516	.486			.563	.452	.490	.529	.534	.539	.600	.617	.620
	Av		.172	.162			.188	.151	.163	.176	.178	.180	.200	.206	.207
Impulse Duration	c		.0033	.0038				.0037	.0040	.0042	.0035	.0041	.0047	.0045	.0044
	m		.0029	.0031				.0034	.0034	.0037	.0031	.0035	.0039	.0042	.0040
	e		.0028	.0029				.0034	.0037	.0041	.0035	.0037	.0044	.0039	.0043
	Σ		.0090	.0098				.0105	.0111	.0120	.0101	.0113	.0130	.0126	.0127
	Av		.0030	.0033				.0035	.0037	.0040	.0034	.0038	.0043	.0042	.0042
Wave Propagation Time	c										.0011	.0022	.0016	.0016	.0020
	m										.0014	.0012	.0071	.0041	.0018
	e					1964 ←		→	1965		.0015	.0007	.0029	.0025	.0023
	Σ										.0040	.0041	.0116	.0082	.0061
	Av										.0013	.0014	.0039	.0027	.0020

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* Highway Research Board Special Report 80

This figure shows trends that are quite similar to the results indicated by the DDT first peak deflection (Fig. 16). The curve for the poor road shows a minimum of about 0.028 in. in the fall series of tests and a maximum of about 0.053 in. in the spring series, indicating a variation of about 89%. This variation is slightly higher than the variation of the DDT first peak deflection. Again, the poor road shows about 2½ times as much displacement as the good road.

Breakdown of the data as a function of transverse position is not plotted. Examination of the data in Appendix C shows trends similar to those found with the DDT first peak deflection data.

Based on the foregoing, it can be concluded that the corrected DDT deflection results are roughly equivalent to the peak deflection results. However, because the corrected DDT deflection data are obtained by processing the original data, the technique is not as simple as the peak deflection technique.

REBOUND TIME

The averages of the rebound time (the bounce time between first and second impacts) are plotted in Figure 21, which shows some seasonal variation and differences between the poor and the good roads. These variations are not as marked, however, as they are with the deflection parameters.

IMPULSE DURATION

The averages of the impulse duration (i.e., the time that the bar remains in contact with the striker plate) are shown in Figure 22, which exhibits some seasonal variations and differences between the poor and the good roads. They are not as marked, however, as the differences in the deflection parameters.

It should be noted that the maximum impulse duration for the poor road occurs in early April, close to the minimum occurrence in Motl's curve (Fig. 1) and to the corrected DDT deflection curves (Fig. 20).

WAVE PROPAGATION TIME

The averages of the surface wave propagation time are shown in Figure 23. Because of the priority given to the other more promising response parameters previously discussed, these data were obtained only towards the end of the test program. Also, because of the relatively small number of test points the results are considered to be inconclusive.

GENERAL CONCLUSION

As a general conclusion, it is evident that the impulsive loading technique is a feasible method of detecting seasonal

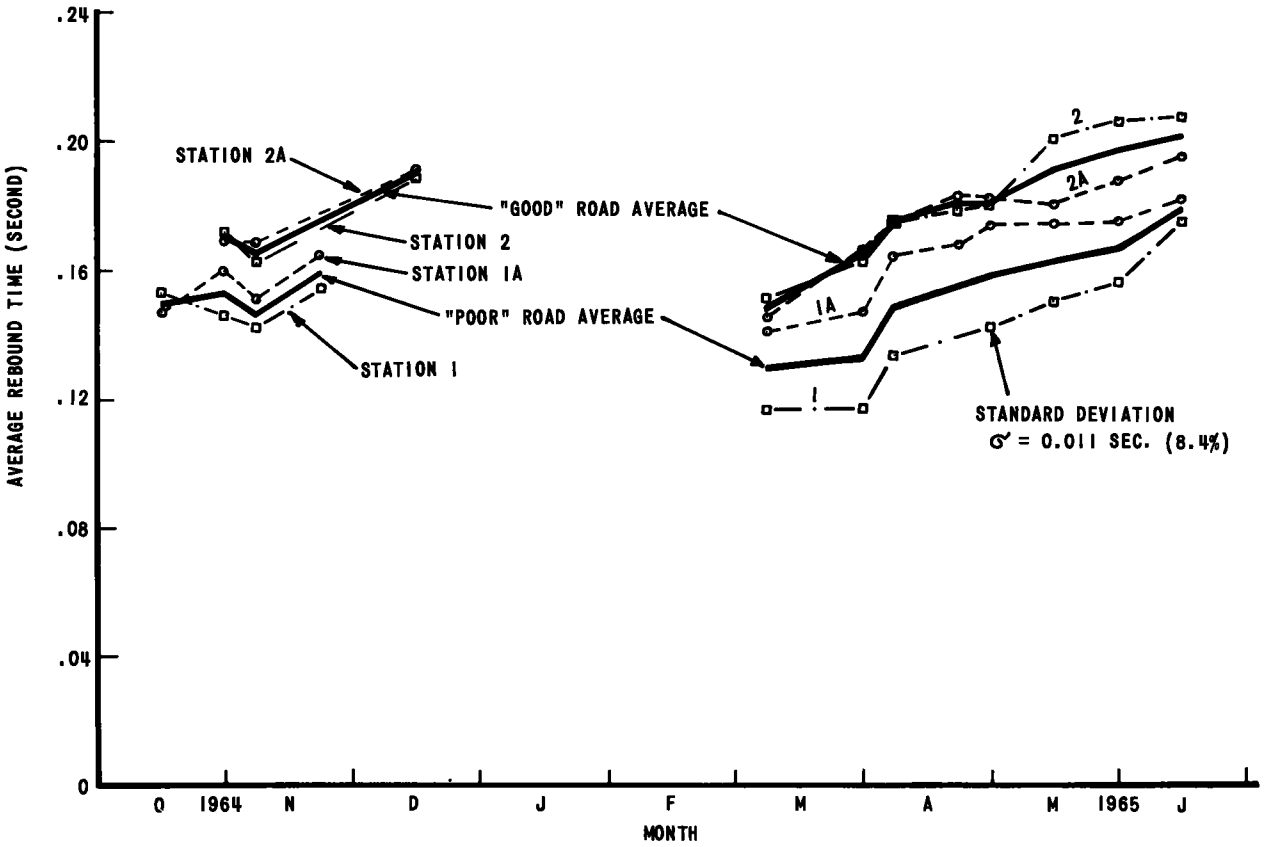


Figure 21. Seasonal variation of average rebound time.

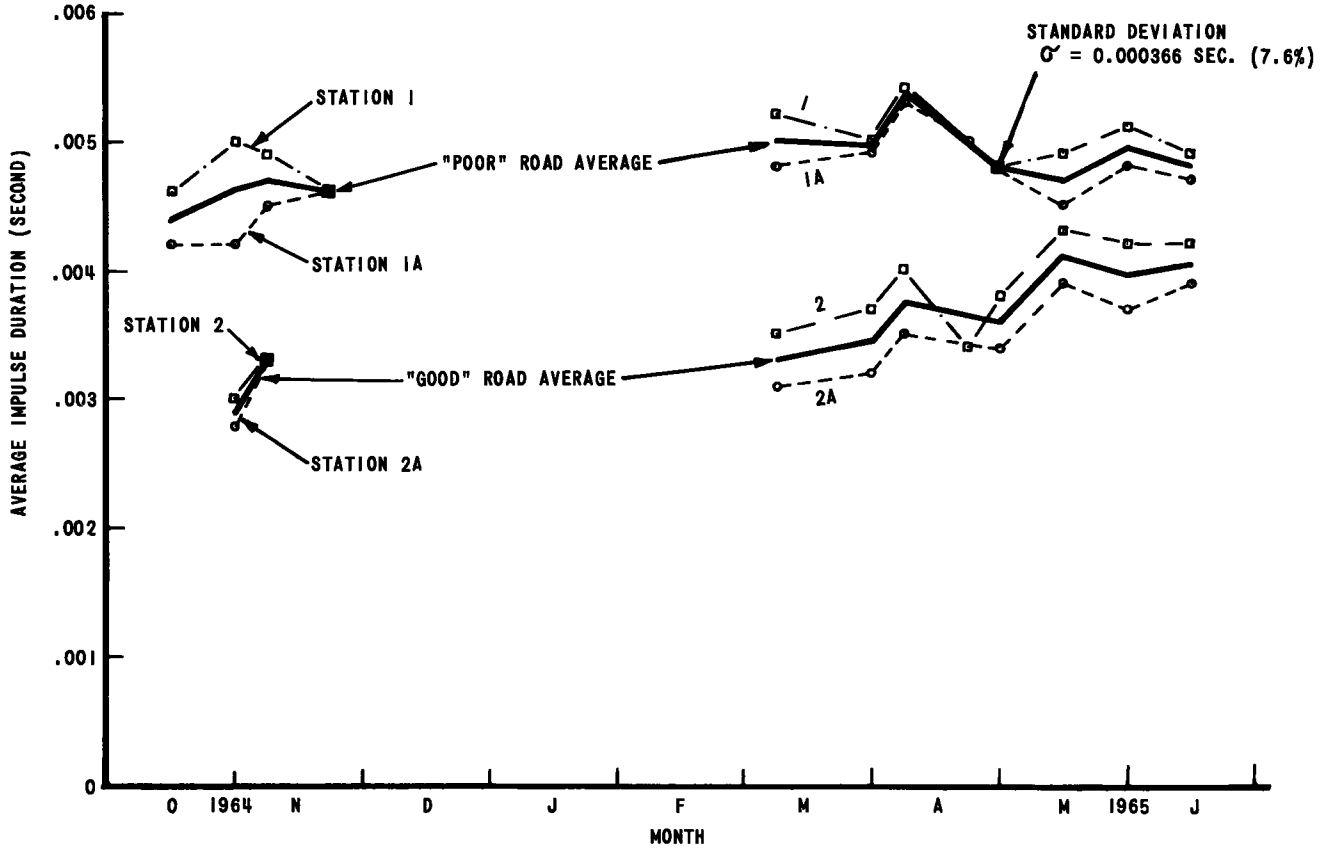


Figure 22. Seasonal variation of average impulse duration.

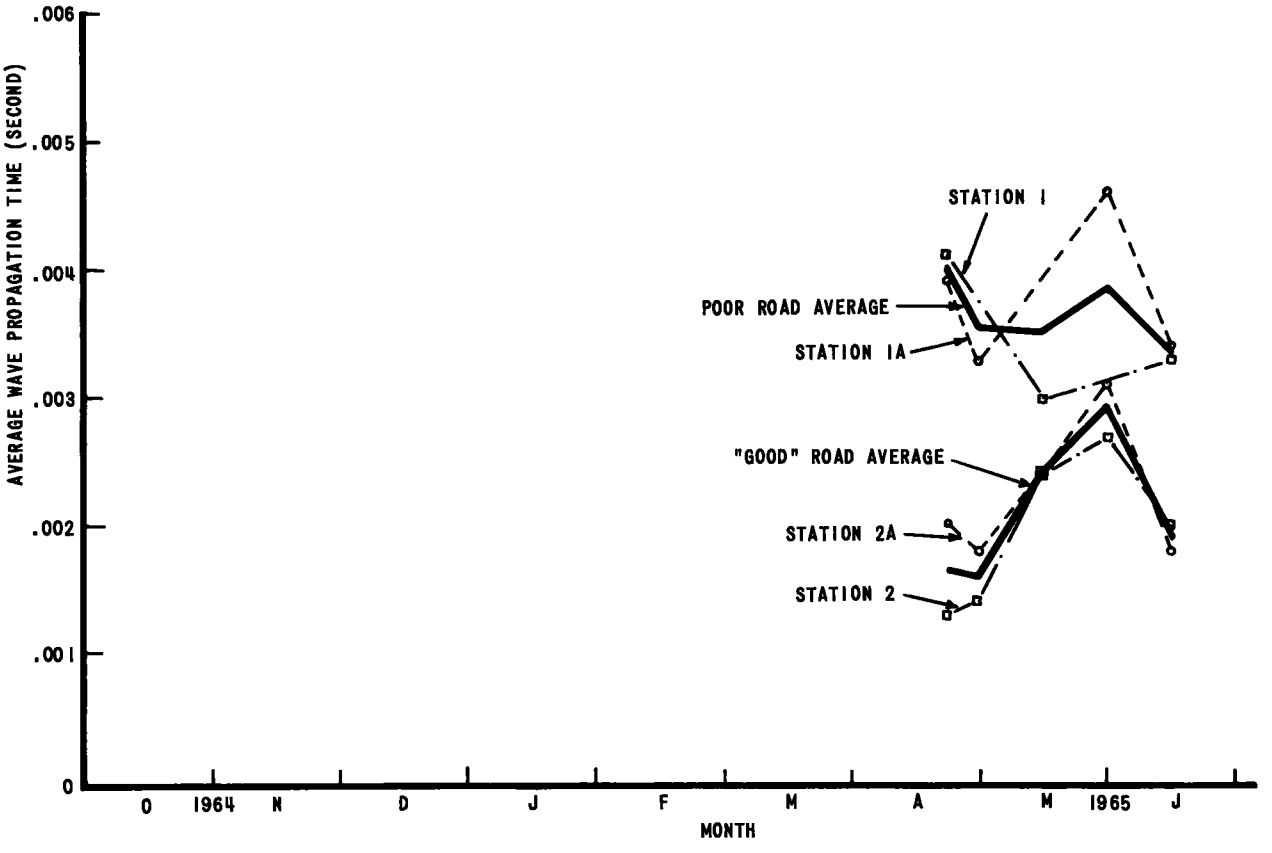


Figure 23. Seasonal variation of average wave propagation time.

variations in the load-carrying capacity of flexible pavements, the DDT first peak deflection or the corrected DDT deflection being the most promising variables to be measured.

SUGGESTED RESEARCH

In view of the promising results obtained during the course of this program it is recommended that the following tasks be undertaken:

1. Design equipment for applying an impulsive load to a road that is better suited for large-scale field testing. The equipment should be compact, mobile, and simple to operate. The device may, for example, combine a pneumatically powered hammer method with the present gravity method. In conjunction with the development of a field operational impulse generator, investigate methods of detecting peak dynamic deflection by means that are simpler than the present DDT recorder method. The seismic mass principle appears to be quite adequate, but a direct readout of deflection is desirable.

2. Using the field equipment previously described, conduct a large-scale field test program covering a wider geographic area and a wider variety of road geometries (e.g., thickness) than has been covered in the present program. Perform these tests simultaneously with static tests in order to provide a basis for a more precise correlation between static and dynamic measures of road condition. Although there appears to be no reason to suspect the results obtained in the present program, a field test program based on a larger statistical sample will increase confidence in the technique.

3. If the results of the large-scale field test program follow the general trends of the present program, establish standards that relate peak deflection readings to road condition (i.e., load-carrying capacity) as a function of road construction, geographic area, time of the year, etc.

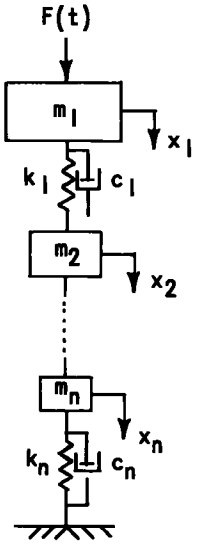
REFERENCES

1. *A Prospectus—Detecting Variations in Load-Carrying Capacity of Flexible Pavements.* Cornell Aeronautical Laboratory. Proposal No. 257 (Aug. 2, 1963).
2. Monismith, C. L., Finn, F. N., et al., "Symposium on Flexible Pavement Behavior as Related to Deflection." *Proc. Assn. Asph. Pav. Tech.*, Vol. 31 (1962).
3. Yoder, E. J., *Principles of Pavement Design*, pp. 506-517, 135-145. Wiley (1959).
4. Clark, D. C., *The Analysis and Design of a Road Impulse Generator.* Cornell Aeronautical Laboratory Report No. VJ-1620-V-2 (June 1962).
5. Hansen, H., and Chenea, P., *Mechanics of Vibration*, pp. 296-298. Wiley (1956).
6. Heukelom, W., "Analysis of Dynamic Deflections of Soils and Pavements." *Geotechnique*, Vol. 11, No. 3 (Sept. 1961).
7. Timoshenko, S., and Goodier, J., *Theory of Elasticity*, pp. 454-459. McGraw-Hill (1961).
8. Lamb, H., "On the Propagation of Tremors over the Surface of an Elastic Solid." *Philosophical Trans. Roy. Soc. Lond.*, Vol. 203 (1904).
9. Finn, F. N., "Symposium on Flexible Pavement Behavior as Related to Reflection. Part I. Methods of Measurement." *Proc. Assn. Asph. Pav. Tech.*, Vol. 31, pp. 211-214 (1962).
10. Jones, R., and Whiffin, A. C., "A Survey of Dynamic Methods of Testing Roads and Runways." *HRB Bull. 277*, pp. 1-7 (1960).
11. Swift, G., *Lane-Wells Dynamic Determination System.* Lane-Wells Co. Tech. Memo. (Feb. 7, 1964).
12. Jones, R., "Measurement and Interpretation of Surface Vibrations on Soil and Roads." *HRB Bull. 277*, pp. 8-29 (1960).
13. Heukelom, W., "Dynamic Stiffness of Soils and Pavements." *Proc. Symp. on Vibration Testing of Roads and Runways*, pp. VI 1-22. Koninklijke/Shell-Laboratorium, Amsterdam (Apr. 20-24, 1959).
14. Maxwell, A. A., "Non-Destructive Testing of Pavements." *HRB Bull. 277*, pp. 30-36 (1960).
15. Jeuffroy, G., and Bachelez, J., "A Contribution to the Study of Deformations and Strains in Roads." *Proc. Symp. on Vibration Testing of Roads and Runways*, pp. IX 1-13. Koninklijke/Shell-Laboratorium, Amsterdam (Apr. 20-24, 1959).
16. Heukelom, W., and Foster, C., "Dynamic Testing of Pavements." *ASCE J. of Soil Mech. and Found.*, Vol. 86, No. SMI (Feb. 1960).
17. Press, F., Oliver, J., and Ewing, M., "Seismic Model Study of Refractions from a Layer of Finite Thickness." *Geophysics*, Vol. 19, No. 3, pp. 388-401 (July 1954).

APPENDIX A

MULTI-DEGREE OF FREEDOM SYSTEM

It has been proposed (*6*) to represent a flexible pavement by a system of masses, dampers, and springs, as shown in the sketch, in which $F(t)$ is an impulsive load, m is a mass, k is a spring, c is a damper, and x is an absolute displacement.



The equations of motion for this systems are

$$m_1 \ddot{x}_1 = - c_1 (\dot{x}_1 - \dot{x}_2) - k_1 (x_1 - x_2) + F(t) \qquad \text{(A.1}a\text{)}$$

$$m_2 \ddot{x}_2 = + c_1 (\dot{x}_1 - \dot{x}_2) + k_1 (x_1 - x_2) - c_2 (\dot{x}_2 - \dot{x}_3) + k_2 (x_2 - x_3) \qquad \text{(A.1}b\text{)}$$

$$\vdots$$

$$m_n \ddot{x}_n = + c_{n-1} (\dot{x}_{n-1} - \dot{x}_n) + k_n (x_{n-1} - x_n) - c_n \dot{x}_n + k_n x_n \qquad \text{(A.1}n\text{)}$$

which may be rearranged to form the matrix equation

$$[M] \{\ddot{x}\} + [C] \{\dot{x}\} + [K] \{x\} = \{f(t)\} \qquad \text{(A.2)}$$

in which

$[M]$ = inertia matrix (diagonal square matrix);
$\{\ddot{x}\}$ = column matrix of acceleration;
$[C]$ = damping matrix (square matrix);
$\{\dot{x}\}$ = column matrix of velocity;
$[K]$ = elastic matrix (square matrix);
$\{x\}$ = column matrix of displacement; and
$\{f(t)\}$ = column matrix of excitation (zero elements except for the impulsive load applied at m_1).

The solutions of Eqs. 1 and 2 may be obtained analytically, by the use of an analog computer, or by the use of a digital computer.

APPENDIX B

PRELIMINARY ROAD TESTS DATA

DYNAMIC RESPONSE OF 3-INCH STANDARD PIPE DUE TO IMPULSIVE LOAD

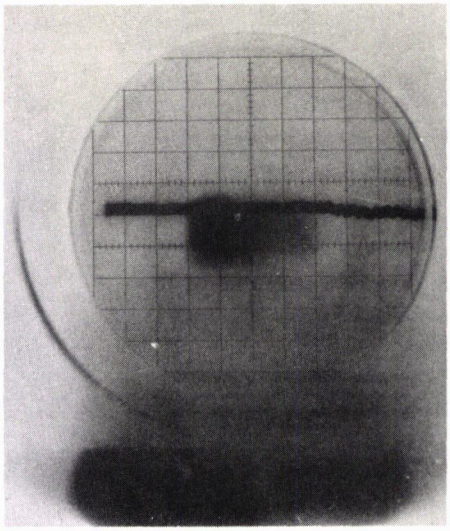
The dynamic displacement transducer (DDT) and the CEC velocity pickup were mounted on the pipe. Oscilloscope and oscillograph records of the DDT and oscillograph record of the CEC were obtained for a 2-ft drop height using a 12-in. diameter by 1-in. thick steel striker plate.

Displacement

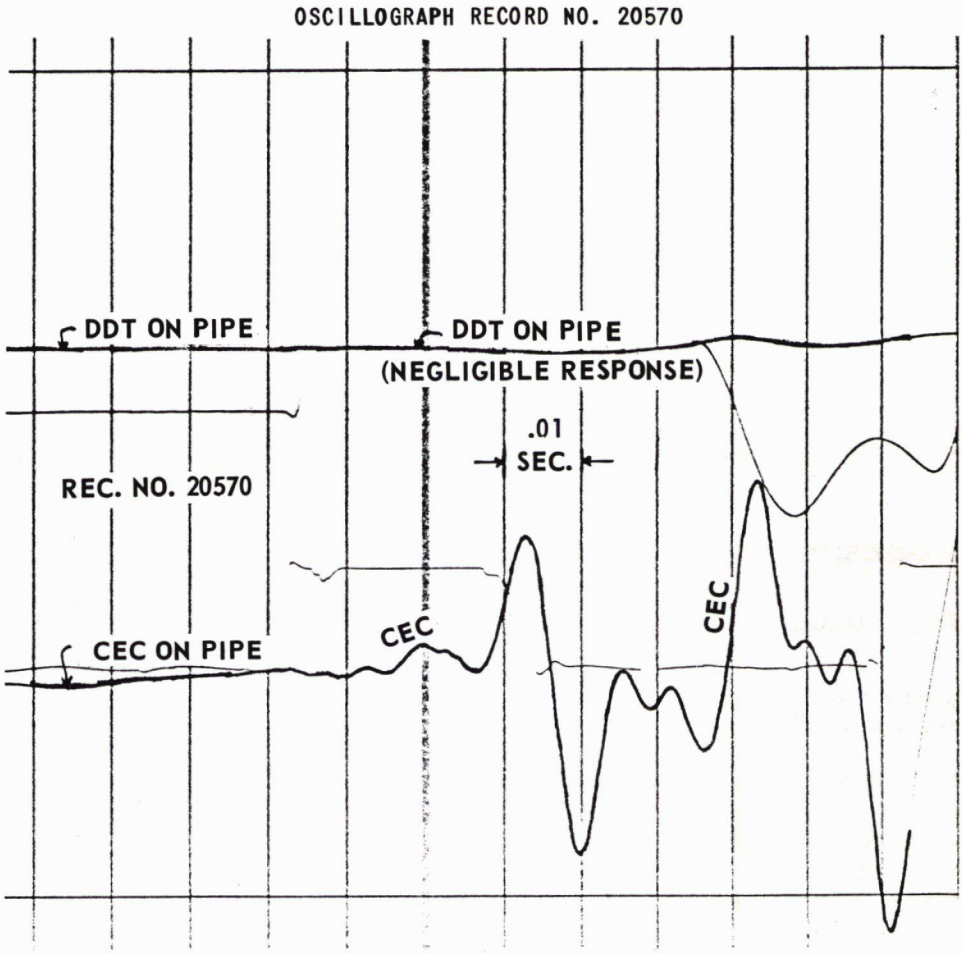
The displacement response of the pipe as monitored by the DDT (Fig. B-1) is negligible compared to the displacement response of the pavement (Fig. B-2), which goes off the paper. It is therefore concluded that the pipe provides a rigid mechanical ground and that the displacement response of the FDT (fixed displacement transducer) is the true displacement response and can be used as a basis for determining the validity of the theory and the fidelity of the DDT.

Velocity

The velocity response of the pipe, as monitored by the CEC and shown in the lower trace of Figure B-1, contains numerous higher harmonics. It was decided that the usefulness of velocity data was quite limited, so the velocity response parameter was abandoned.

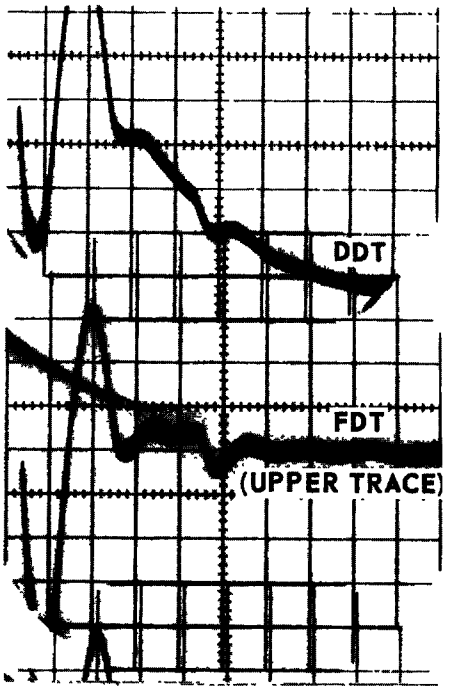


(a) DISPLACEMENT RESPONSE FROM DDT
SWEEP: 0.10 SEC. PER BOX
GAIN: 2 VOLTS PER BOX



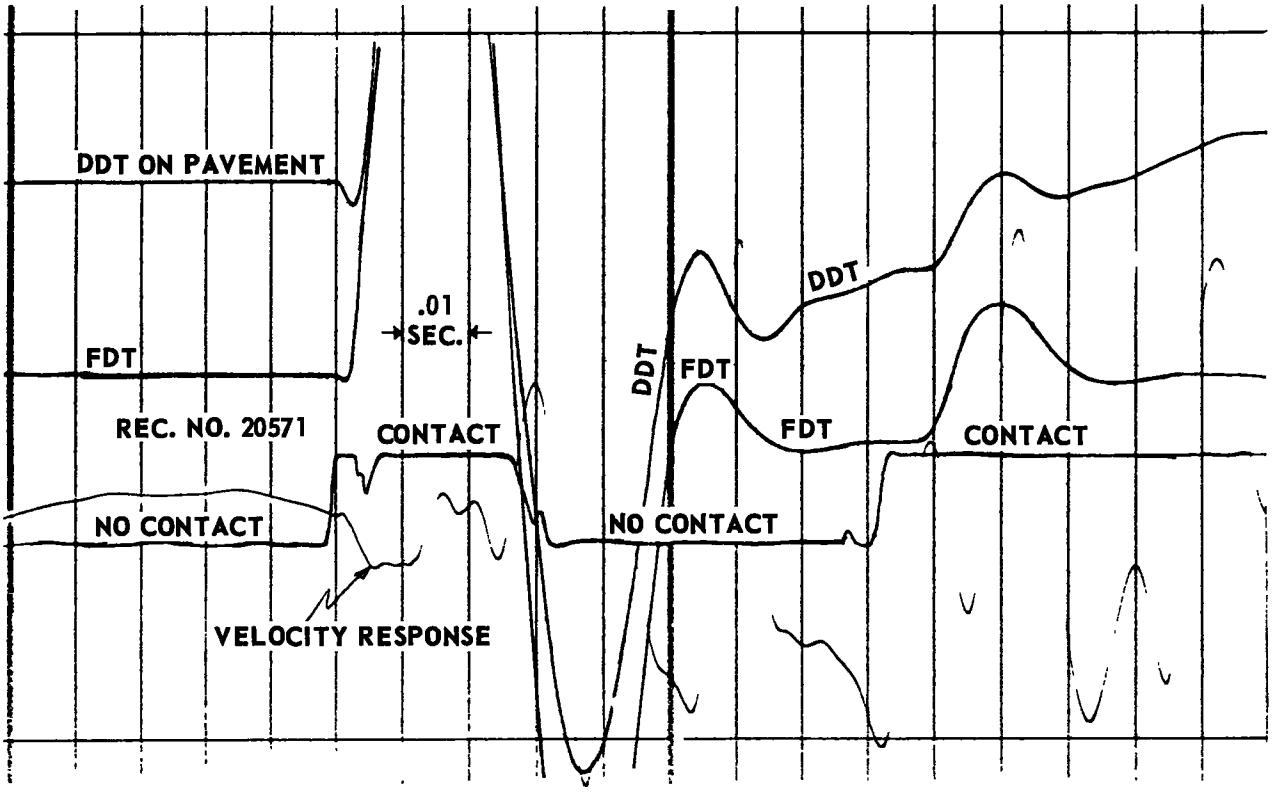
(b) RESPONSE FROM OSCILLOGRAPH
UPPER TRACE: DDT ON PIPE
LOWER TRACE: CEC ON PIPE

Figure B-1. Dynamic response of pipe due to impulsive load; drop height 2 ft.



(a) DISPLACEMENT RESPONSE FROM OSCILLOSCOPE
UPPER TRACE: FROM FDT
LOWER TRACE: FROM DDT
SWEEP: 0.02 SEC. PER BOX
GAIN: 2 VOLTS PER BOX

OSCILLOGRAPH RECORD NO. 20571



(b) DISPLACEMENT RESPONSE FROM OSCILLOGRAPH

Figure B-2. Dynamic response of pavement due to impulsive load; steel striker plate, drop height 2 ft.

It can be seen that the displacement trace as monitored by the DDT is predominantly due to the road up to the first peak, whereas the later part, which is of a low frequency in nature, is predominantly due to the transient response of the DDT suspension system.

The displacement data were handled in two ways. The magnitude of the first peak deflection was read directly from the trace and is considered to be a good measure of the *relative* magnitude of the maximum dynamic deflection of the road. In addition, the low-frequency response of the DDT instrument was extrapolated back to zero time (i.e., the instant of impact) by fairing in the low-frequency wave. The difference between this faired-in trace and the first peak deflection (called "corrected DDT deflection") is considered to be a fair measure of the *absolute* magnitude of the maximum dynamic deflection of the road.

The determination of the impulse duration and rebound time was straightforward. The propagation time was difficult to determine accurately from the velocity pickup data because of the problem of reading from the traces the precise point in time at which the velocity wave disturbance begins.

Five parameters were obtained from the oscillograph records, as follows: (a) DDT first peak deflection, (b) corrected DDT deflection, (c) rebound time, (d) impulse duration, and (e) wave propagation time. The values of the three drops were averaged and the tabulated values (Appendix C) in turn were averaged and plotted. The plots (Figs. 16 through 23) are discussed in Chapter Three.

CHAPTER THREE

INTERPRETATION OF TEST RESULTS, CONCLUSIONS, AND SUGGESTED RESEARCH

The five response parameters obtained from the oscillograph records are discussed in the following.

DDT FIRST PEAK DEFLECTION

The first peak deflection values as indicated by the DDT are shown in Figures 16, 17, 18, and 19. As might be expected, these figures show that the good road does not deflect as much as the poor road.

Figure 16 shows the average of the DDT first peak deflections for the nine drops at each station (3 drops at each of 3 positions) performed on a given day. The dispersion or variability of the data for each station in Figure 16 can be seen from Figures 17 and 18. The standard deviation for station 1A (see Fig. 16) for the test performed on April 20, 1965 (where the spread is largest in the spring series of tests) is ± 0.0075 in. (± 12.6%) whereas that for station 1 is ± 0.0056 in. (± 8.4%). It can be seen from the plot of the overall average of 18 drops (3 drops on each of 6 positions) for the poor road that the fall series shows a minimum of about 0.037 in., whereas the spring tests show a maximum of 0.063 in., indicating a maximum seasonal variation of about 70%. The maximum occurs in April, which correlates well with the bottoming of Motl's load-carrying capacity curve (Fig. 1). Also, the good road shows lower deflections than the poor road, with its maximum occurring later (i.e., in May instead of in April). The reasons for this delay may be of some significance and may be worthy of future investigation. It might be due to slight differences in surface elastic properties as a function of seasonal temperature or to differences in drainage properties. The maximum deflection for the good road is far below the minimum deflection for the poor road and is on the order of 2½ times less than the maximum deflection of the poor road. This relationship is held to be reasonable in light of Eq. 6. Vibratory tests (*6, 13, 14*) show that the natural frequency for bituminous pavements (about 30 cps) does not vary much. Hence, the maximum deflection may be assumed as inversely proportional to the spring, k. The spring rate obtained through vibratory tests ranges from 116 tons/cm for unsurfaced, well compacted, silty clay (*14*) to 770 tons/cm for an extremely good European pavement (*6, 16*). The reported ratio of maximum deflections of 2½ between the good road and the poor road is within the ratio of these values, which served as the basis of this study. It will, however, be an extremely good idea to include the detailed response of each layer and the soil in future studies. It is believed that the soil foundation, in addition to the base and pavement layers, responds to the impulsive load because of the following:

1. The response at the edge of the road is significantly higher than the responses at the middle of the lane and center of the road.
2. The impulsive load application is completed much earlier than the time of occurrence of the peak response.
3. The agreement with the design basis (i.e., the vibratory data obtained by researchers throughout the world from the point of view of both maximum displacement and natural frequency).

Figure 17 shows the variation of deflection with transverse location at one station. Each point is the average of three drops. There is good repeatability—i.e., the three values are very close to their average (within approximately

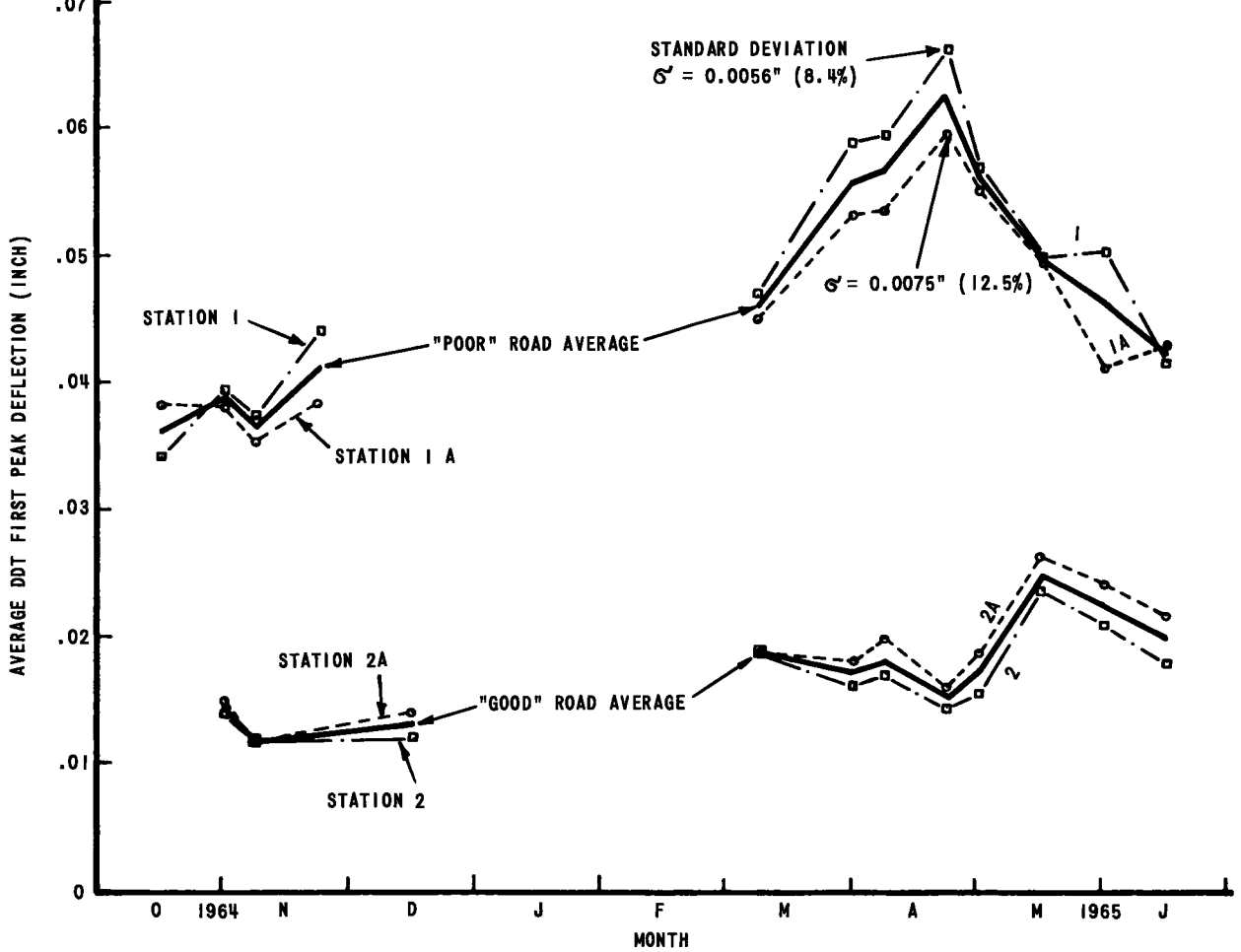


Figure 16. Seasonal variation of average DDT first peak deflection.

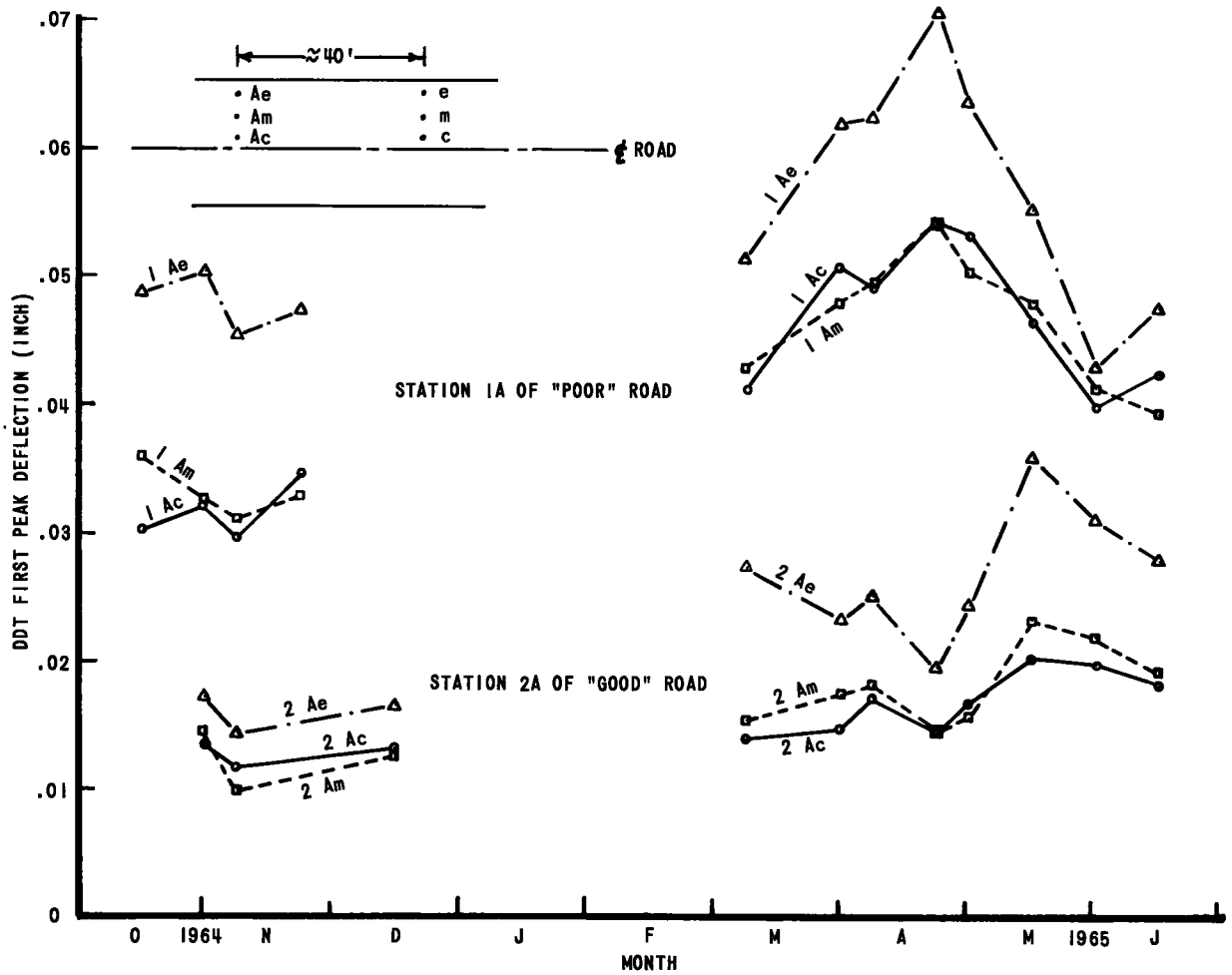


Figure 17. Effect of transverse position on DDT first peak deflection (Stations 1A and 2A).

± 10%). The deflections for the poor road are significantly higher at all transverse positions. Figure 18 shows the effect of transverse location at a station about 40 ft from that of Figure 17. The general trend of Figure 17 is repeated.

Figure 19 is a re-plot of the data of Figures 17 and 18 with the inclusion of the mid-station test point $1B_m$. This shows the mid-station responses to be not too far different from those at the end stations.

The relatively small dispersion in the data of Figure 16—that is, the small spread between the station averages (e.g., between 1 and 1A)—and the marked difference of the overall averages shown from fall to spring, would appear to support the conclusion that use of a deflection transducer of the seismic mass type (the DDT) is a reliable and unambiguous way of measuring the relative peak deflection response to impulsive loads. Comparing the spring data of Figure 16 with Motl's bearing capacity curve (Fig. 1) (data not being available on the load-carrying capacity under static load) it may be further concluded that there is a good qualitative correlation between peak DDT deflection and bearing capacity, peak deflection being roughly inversely proportional to bearing capacity. Motl's curve was selected because the deflections and frequencies observed by means of the impulsive loading technique were within the range reported by several investigators using the vibratory technique. In addition, it was decided that testing of a good road (should it result in lower displacements, as it later did) should be as fruitful as a static test correlation. This correlation is more evident for the poor road (Fig. 16), but is also evidenced for the good road. Finally, it may be concluded that the DDT peak deflection is a good way of measuring load-carrying capacity as a function of season, as a function of the road condition (i.e., distinguishing between a "good" or a "bad" road), or, to a lesser extent, as a function of transverse location on a given road.

CORRECTED DDT DEFLECTION

The averages of the corrected DDT deflection, which, it will be recalled, is believed to be a fair measure of the absolute maximum dynamic deflection, are plotted in Figure 20.

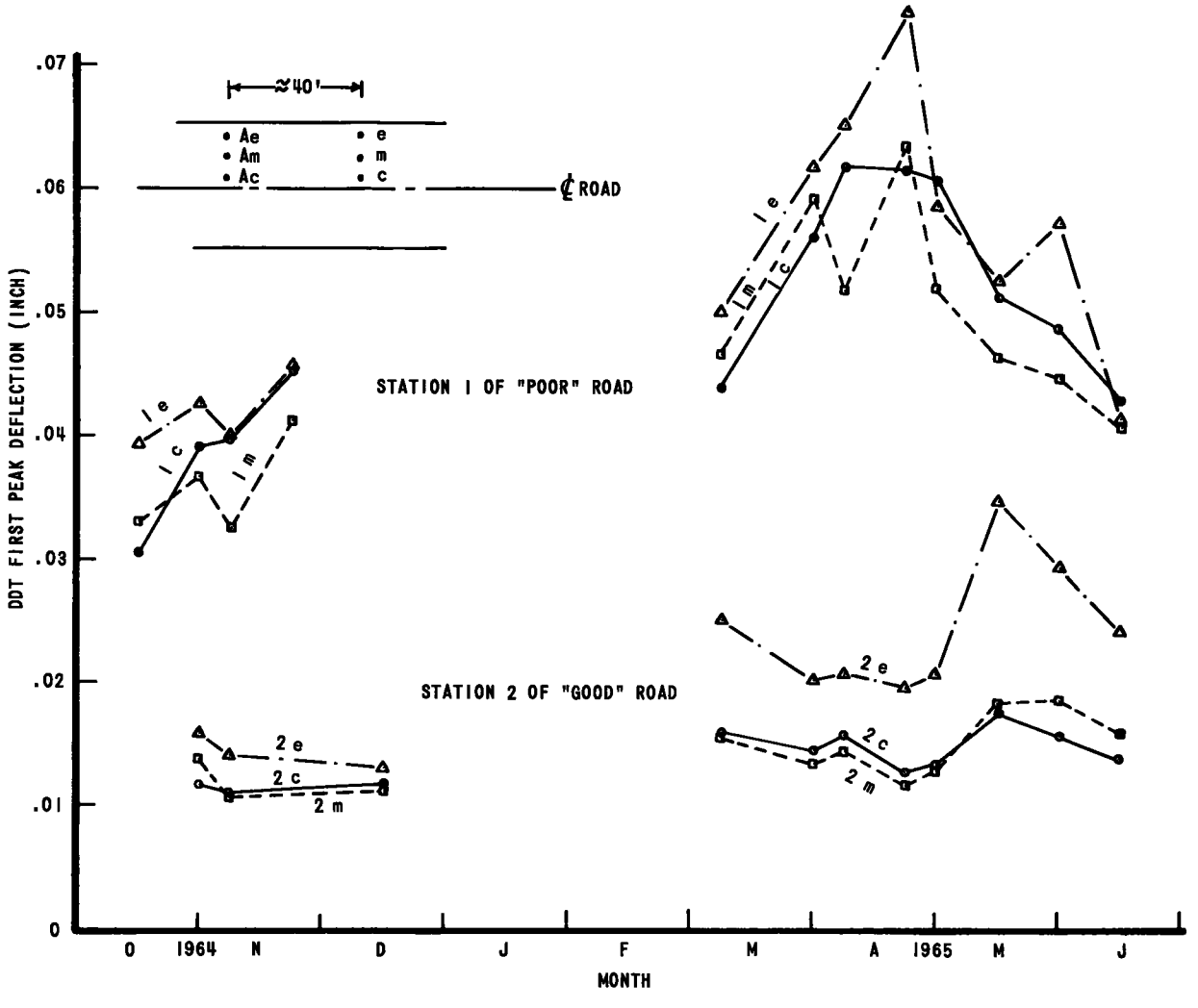


Figure 18. Effect of transverse position on DDT first peak deflection (Stations 1 and 2).

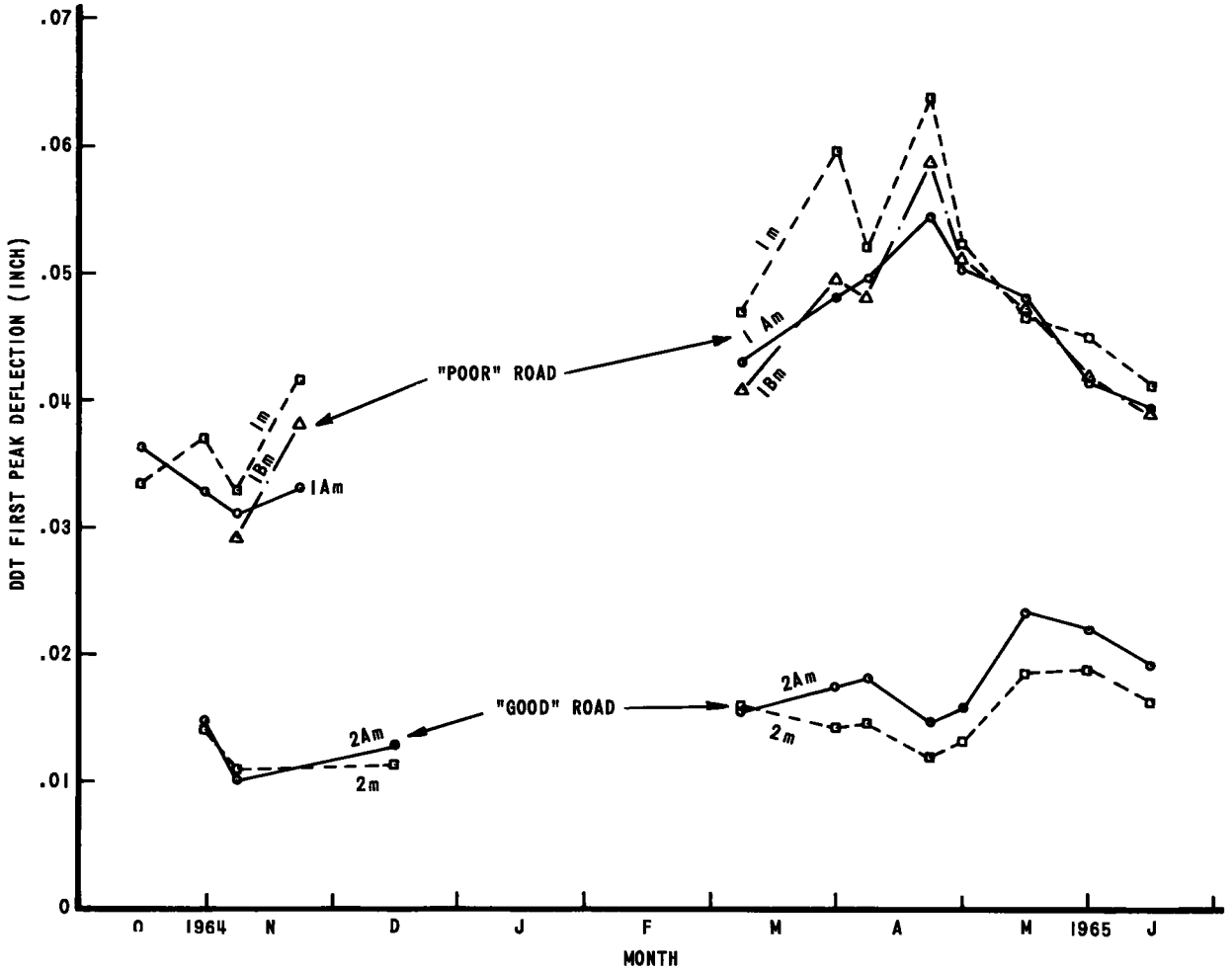


Figure 19. Effect of longitudinal position on DDT first peak deflection near middle of lane.

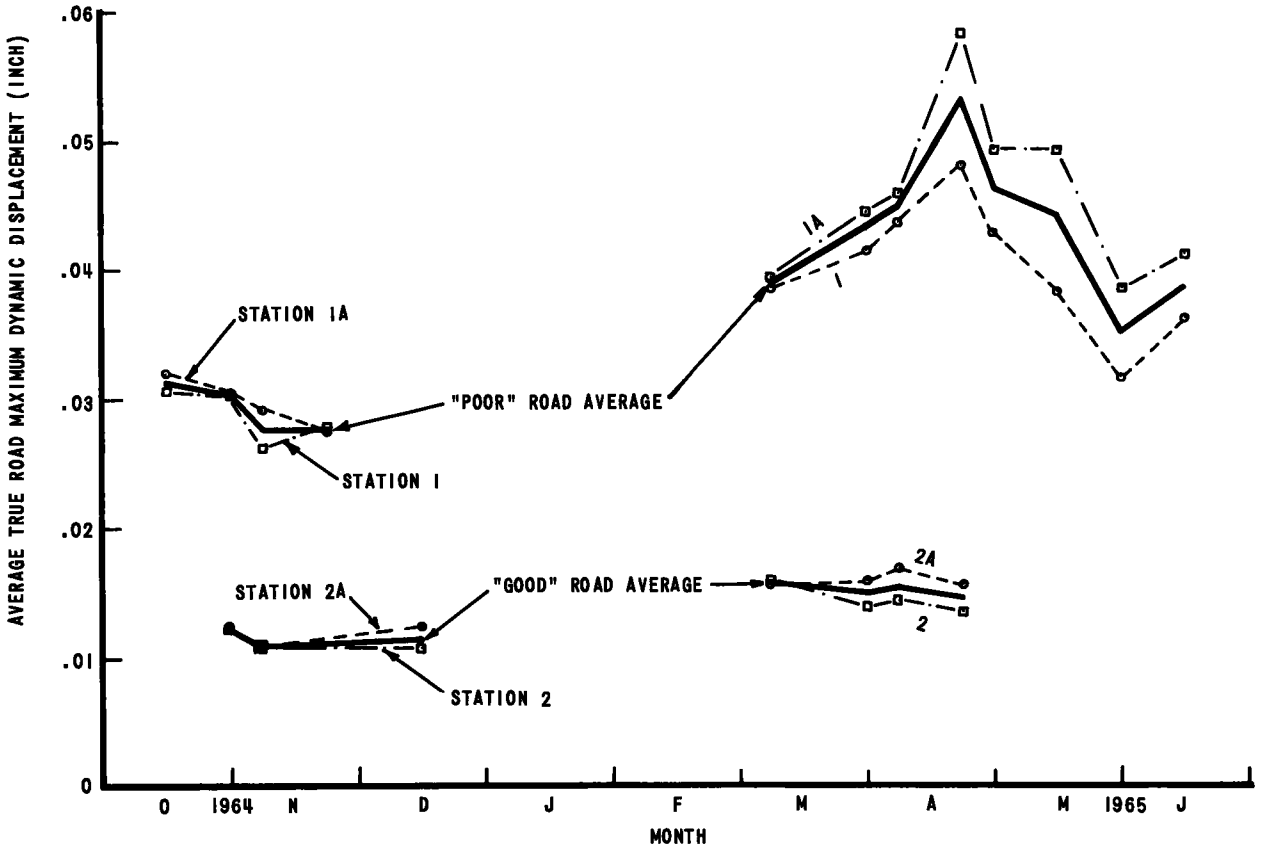


Figure 20. Seasonal variation of average corrected DDT deflection.

proximate expressions, from Lamb's classical solutions (*8*), for the response of points near the source of the impulse was not carried out. In addition, it is felt that a single degree of freedom model yields a solution that is adequate for the purposes of this study.

Flexible Pavement Response to an Impulsive Load

No solutions that are amenable to the development of approximate formulas were found in the literature which gives the response of a flexible pavement to impulsive loads. This is not surprising because a flexible pavement is a layered system; the materials are viscoelastic, nonhomogeneous, and nonisotropic. In addition, realistic field loadings always have finite duration and finite area of application instead of idealized impulse duration (approaching zero time) and point loading.

FUNCTIONAL RELATIONSHIPS APPROPRIATE FOR FIELD USE

Most of the approximate expressions relating the response of pavement variables to different loadings were developed for ideal cases. These relationships will be affected by nonideal field conditions, such as the presence of damping and nonlinearities. They would also have to be modified to meet the requirements of rapidity and simplicity in the execution of the experiments and interpretation of the data. With this in mind, the following functional relationships were considered to be potentially useful for field use.

Maximum Dynamic Displacement of a Damped Equivalent Single Degree of Freedom System

The maximum displacement of a damped equivalent single degree of freedom system subjected to an ideal impulse is less than for the undamped case; that is,

$$u_{max}\ (\text{damped}) = u_{max}\ (\text{undamped}) \times \text{factor} \quad (13)$$

The factor is less than unity and depends on the damping. It may be obtained by differentiating the response to an ideal impulse, equating the derivative to zero and solving for the time at maximum displacement, then substituting back to the original solution to obtain the maximum displacement. Hence, the maximum dynamic displacement will depend not only on the elasticities of the pavement layers and subgrade but also on their damping properties, making it a possible measure of load-carrying capacity.

Rebound Time

Although the impulse energy-maximum dynamic displacement relationship discussed previously is applicable for field use, it is found to be not the most efficient. Instead, for a given drop height, h_1, the maximum displacement of an equivalent undamped single degree of freedom system is

$$u_{max} = \frac{m_b\,(\sqrt{2\,g\,h_1} + \sqrt{2\,g\,h_2})}{\sqrt{k\,m}} \quad (13)$$

Thus, the rebound height, h_2, depends on the maximum displacement (though not linearly), which in turn depends on the elasticities of the pavement layers and the subgrade. The rebound height may thus be considered a possible measure of the load-carrying capacity of flexible pavements.

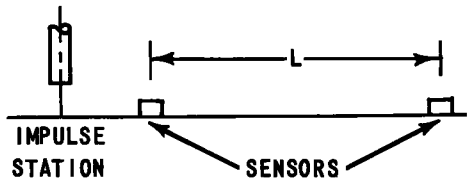
The rebound height discussed in the foregoing may be expressed in terms of the rebound time, which is the time required for the bar to go up and then drop down.

Impulse Duration

If an elastic bar is dropped onto an ideal rigid road, the impulse duration will depend on the traveling stress wave in the elastic bar. However, if the road is elastic the impulse duration will depend not only on the bar properties but also on the road properties. Hence, the impulse duration may also be used as a possible measure of the load-carrying capacity of the pavement.

Wave Propagation Time

Although it is possible to obtain wave velocities in the field, it is difficult to use wave velocity directly. This is because it requires that the distance between sensors be divided by the wave propagation time. If, however, the distance between sensors is specified, the wave propagation time becomes a direct measure of the wave velocity; that is,



$$t = L/c \quad (14)$$

where

t = wave propagation time;

L = distance between sensors; and

c = wave velocity.

If the pavement were an ideal elastic half-space, the propagation time could be considered a possible measure of the load-carrying capacity because it is a function of the wave velocity. However, a real pavement is far from being an ideal elastic half-space, so there is only little hope that the propagation time will be a successful parameter for detecting seasonal variations in the load-carrying capacity.

Maximum Space Rate of Change of Dynamic Displacement

It is known that pavement stresses depend on the pavement elasticities and pavement strains, the latter being manifested partially by the space rate of change of displacement. The space rate of change may thus be considered a possible measure of the load-carrying capacity of flexible pavements. However, because of the priority given to the previous parameters, the difficulty of constructing a fixed mechanical ground (see later discussion) without blocking traffic, and the fact that no portable commercial transducer(s) are available for measuring dynamic displacements at various pavement locations simultaneously, it was not possible to verify experimentally whether the maximum space rate of change of dynamic displacements can be useful in detecting seasonal variations in load-carrying capacity. It is hoped that future studies will include consideration of this parameter.

USE OF ELECTRONIC COMPUTERS

The electronic digital computer can be used in the processing of displacement time data. This is particularly useful if one wishes to perform a Fourier analysis of recorded data. Inquiries made of available programs revealed that programs have been written for this purpose. However, it was found that this relatively sophisticated technique of analysis was not necessary for the purposes of this feasibility study.

DESIGN AND FABRICATION OF AN EXPERIMENTAL IMPULSE GENERATOR AND INSTRUMENTATION SYSTEM

Impulse Generator

The design of an experimental generator was based on a range of pavement deflections from 0.01 in. to 0.05 in., based on considerations of pavement failure (*2, 6*), and an impulse duration which is much less than the expected road natural period of about 0.033 sec based on information by

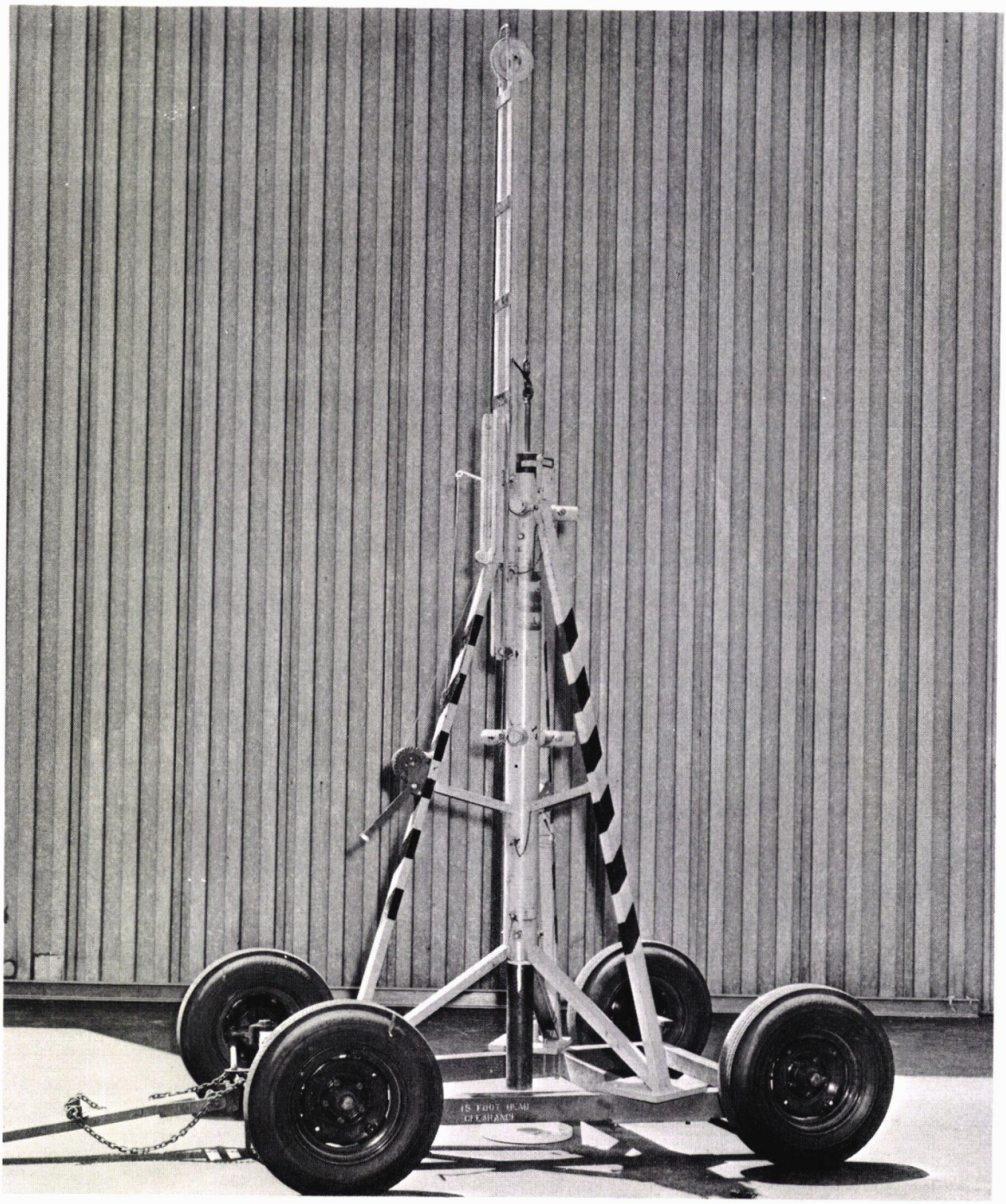


Figure 6. CAL experimental gravity-type road impulse generator.

Figure 7. Close-up of operating area, CAL experimental gravity-type road impulse generator.

Heukelom (*6*) and the physical constants of the prior section on "Approximate Expressions Relating Response to Pavement Variables." These requirements resulted in a gravity-type impulse generator equipped to drop a steel bar 8.5 ft long by 4.25 in. in diameter, weighing about 500 lb, with an adjustable drop height of from 0 to 4 ft. The support structure is a tripod 2.5 ft on a side and mounted on a four-wheeled pneumatic-tired trailer that can be towed from site to site. The structure is equipped with quick release and lifting mechanisms. The trailer wheel base, track width and road clearance are 70 in., 63 in., and 5 in., respectively. The striker plate is a round aluminum plate 1 in. thick by 15 in. in diameter resting freely on the road.

More detailed discussions of the design and fabrication of the experimental road impulse generator can be found elsewhere (*4*). Figure 6 shows a general view and Figure 7 shows the working area of the experimental road impulse generator. The instrument held by the operator in Figure 7 is the dynamic displacement transducer (DDT), which is discussed subsequently.

Experimental Instrumentation System

The design of the instrumentation system consisted of the design of the displacement transducer, the associated circuitry, and the supporting instrumentation.

The main requirement for the displacement transducer was that it be capable of sensing deflections of up to ± 0.050 in. at about 30 cycles per second. Two transducers were assembled—one using a fixed mechanical ground and a commercial electronic linear variable differential transformer, called the "fixed displacement transducer" (FDT), and another which is portable (see Fig. 7). The latter, called the "dynamic displacement transducer" (DDT), has a suspended mechanical ground (a natural frequency of about 1.7 cycles per second, which is much less than the road frequency of about 30 cps), and a built-in linear variable differential transformer (LVDT). The DDT is basi-

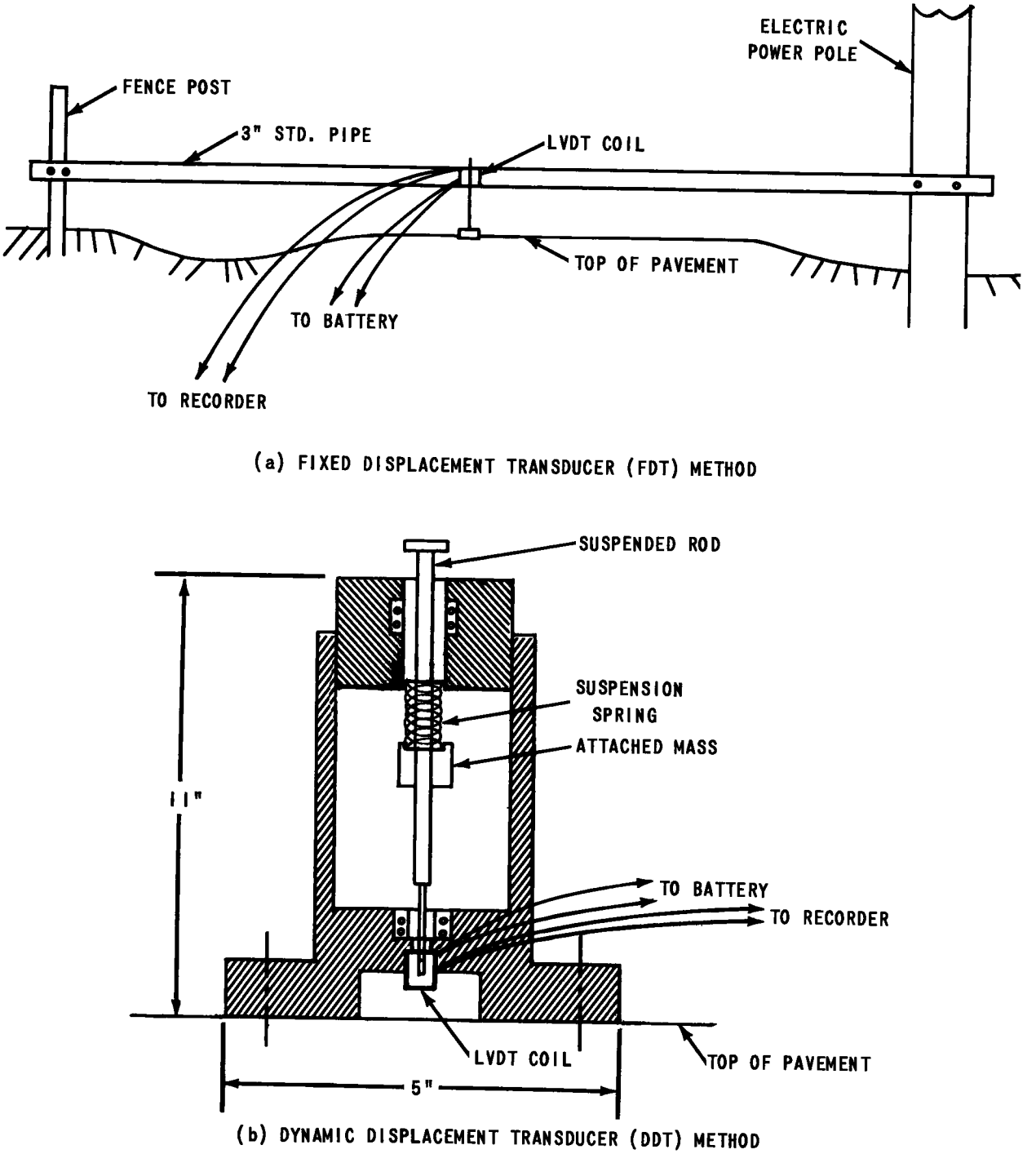


Figure 8. Two methods of measuring displacements.

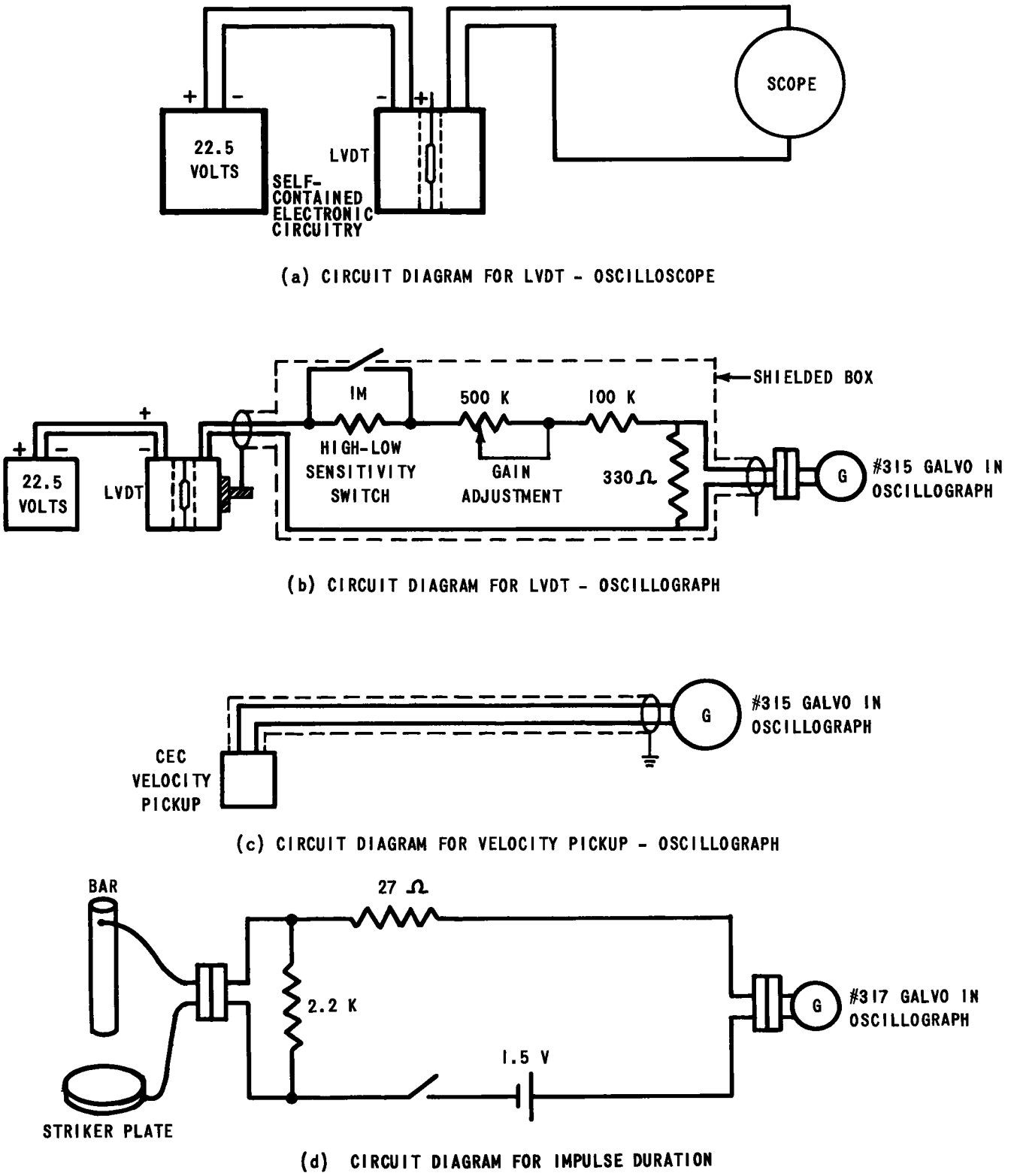


Figure 9. Circuit diagrams.

cally a seismic mass displacement sensor that is fastened to the ground in use. Both displacement measurement methods are shown in Figure 8; further details are given in Appendix B.

The electronic circuitry for the instrumentation used in the preliminary road tests to check out the equipment is shown in Figure 9a. Figures 9b, 9c and 9d represent equipment used in the highway field tests.

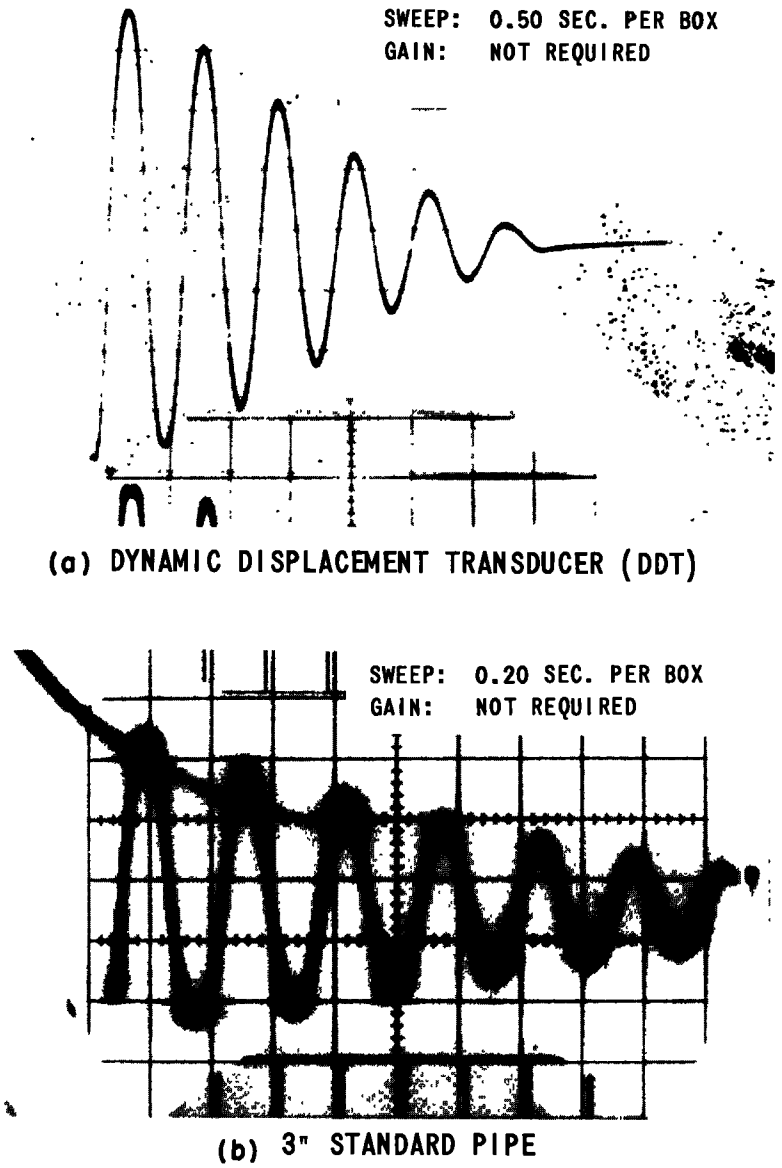


Figure 10. Free vibrations.

The supporting instrumentation consisted of a Tektronix type 502A oscilloscope with single sweep feature, a Polaroid Land camera, and a CEC recording oscillograph (type 5-114) with built-in timing mechanism.

PERFORMANCE OF ROAD TESTS

Preliminary road tests to check out the experimental impulse generator and instrumentation system (see Figs. 8 and 9 for the test setup and circuit diagrams) were performed inside the CAL compound on a two-lane secondary road about 23 ft wide and consisting of a 3-in. macadam surface over a 6- to 8-in. base (a mixture of stones and screenings).

Preliminary Road Tests

The free vibration of the dynamic displacement transducer was excited by displacing the suspended rod (see Fig. 8) and then releasing it. The displacement response was recorded by the oscilloscope and the oscillograph (see Fig. 10).

To check out the suitability of the DDT for measuring dynamic displacement in the field, a reference measurement of displacement was required. For this purpose a 3-in. standard pipe was bridged across the road, as shown in Figure 8a. This provided a fixed mechanical ground for the fixed displacement transducer (FDT). The free vibration of the pipe was excited by displacing the center and then releasing it. Both oscilloscope (see Fig. 10) and oscillo-

graph records of the displacement response were obtained. Drop tests were made with the impulse generator located near the pipe. Several drop heights were used and different striker plates tried. The displacement response of the road to the impulsive loads was measured with both the FDT and the DDT. Also, pipe deflection and velocity were measured (Fig. B-1) in order to insure that the impulsive loads on the road did not set the pipe in vibration (see Appendix B for additional details).

Some of the results of these preliminary tests were as follows:

1. Free vibration of the DDT indicated a natural frequency of about 1.7 cps, which is low compared to the road natural frequency of about 30 cps.

2. Free vibration of the 3-in. pipe indicated a natural frequency of about 3 cps and an undamped system.

3. The displacement and velocity responses of the 3-in. pipe, due to an impulsive load on the road, were negligible, making it a valid fixed mechanical ground.

4. There is good agreement between the road displacement as measured by the DDT and the FDT (the reference measurement). This is shown in Figure B-3. The DDT provides a good absolute measurement of dynamic displacement, but not as accurate a measurement as the FDT. This is due to the fact that despite the large frequency separation between the road vibration ($\cong$ 30 cps) and the DDT natural frequency as a spring-mass system (1.7 cps) there is a slight "contamination" of the road response, even at the beginning of the transient response. This contamination becomes quite evident as time increases (Fig. B-3). The DDT cannot, of course, measure static displacement. As a result of these tests it was concluded that the DDT is perfectly adequate for the measurement of transient road vi-

Figure 11. Road test setup.

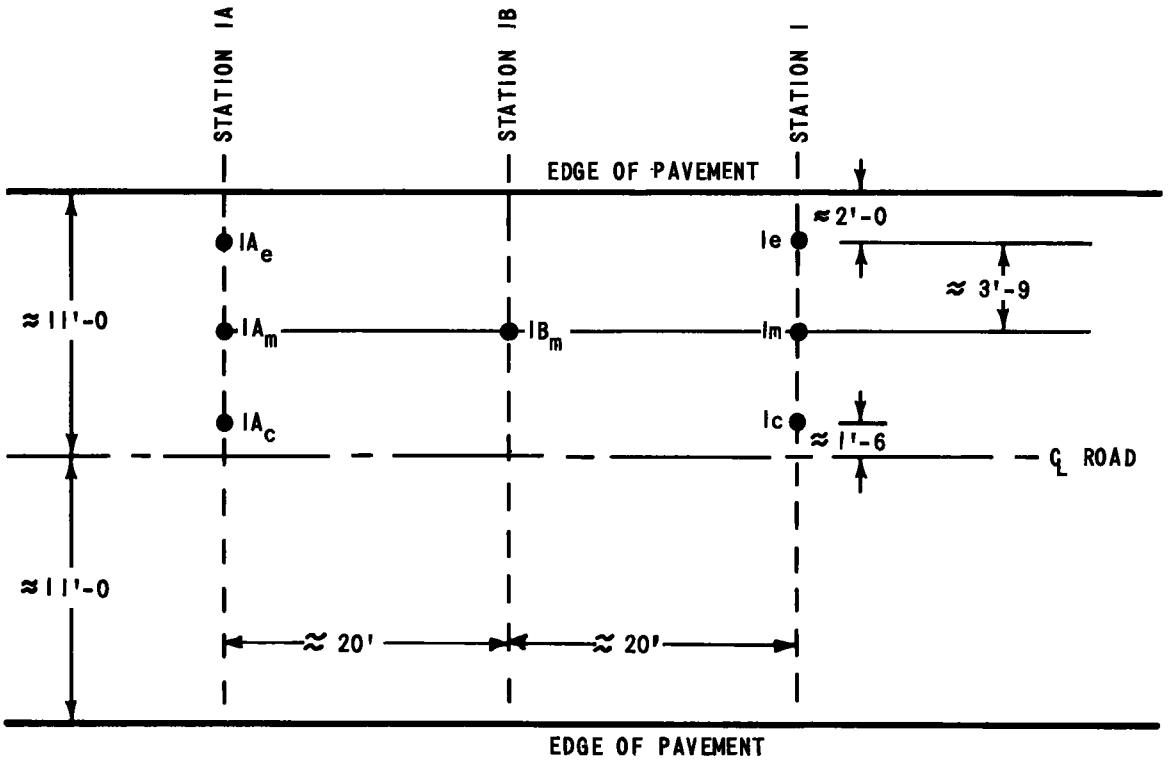


Figure 12. Location of test positions, poor road.

brations immediately after impact (i.e., the first and perhaps the second peak of the traces of Fig. B-3).

5. A drop height of 1 ft, used in conjunction with a 15-in. diameter × 1-in. thick striker plate, does not damage the road.

6. The velocity data were of limited usefulness due to the presence of higher harmonics. A 1-in. thick by 15-in. diameter aluminum striker plate gave better performance among the several that were tried; thinner plates tended to deform and smaller diameter plates tended to dent the road surface.

8. The beginning of the wave propagated is hard to pinpoint (see later discussion), making wave propagation time difficult to calculate.

Highway Tests

As mentioned previously, two test sites were chosen and designated simply as the poor road and the good road. The poor road is built with 12 in. of gravel, 4 in. of penetration macadam, and 2½ in. of asphalt (1½ in. binder). The good road is built with 12 in. of crushed stone, 4 in. of penetration macadam, and 2½ in. of asphalt (1½ in. binder). Other specifics are as follows:

1. Poor road versus good road:
   (a) The load-restriction value of 8 tons/axle is posted in the spring time.
   (b) Failures observed are mainly alligator cracking, shear failure, longitudinal crack, and rutting.
   (c) There is no way of determining whether the test sites represent average conditions, as only two stations of each were tested. However, the data obtained show that they do not vary considerably. (Fig. 19).
2. Soils:
   (a) Poor road—yellow silt, hence classified as poor soil by the Erie County (N. Y.) Highway Dept.
   (b) Good road—unknown type, but classified as fair soil by the Erie County Highway Dept. and used as fill material by the New York State Highway Dept.
3. Age and traffic data:
   (a) Poor road—built in 1958 with traffic counts showing a range of 1,147 to 4,700 mixed vehicles per day (no breakdown available).
   (b) Good road—built in 1963 with traffic counts showing a range of 1,031 to 4,200 mixed vehicles per day (no breakdown available).

4. Climate and frost penetration—In spite of the fact that climatic and frost penetration factors are beyond the scope of the study, the discussion by Yoder (*3*, pp. 123-145) was explored briefly. It was concluded that the limited funding could not allow correlation with air temperatures nor field measurements of frost penetration.

   (a) The following air temperatures were obtained:

+ 3° C — 3- 8-65	+ 14° C — 4-26-65
+ 2° C — 3-31-65	+ 18° C — 5-13-65
+ 7° C — 4- 8-65	+ 15° C — 6- 3-65
+ 7° C — 4-20-65	

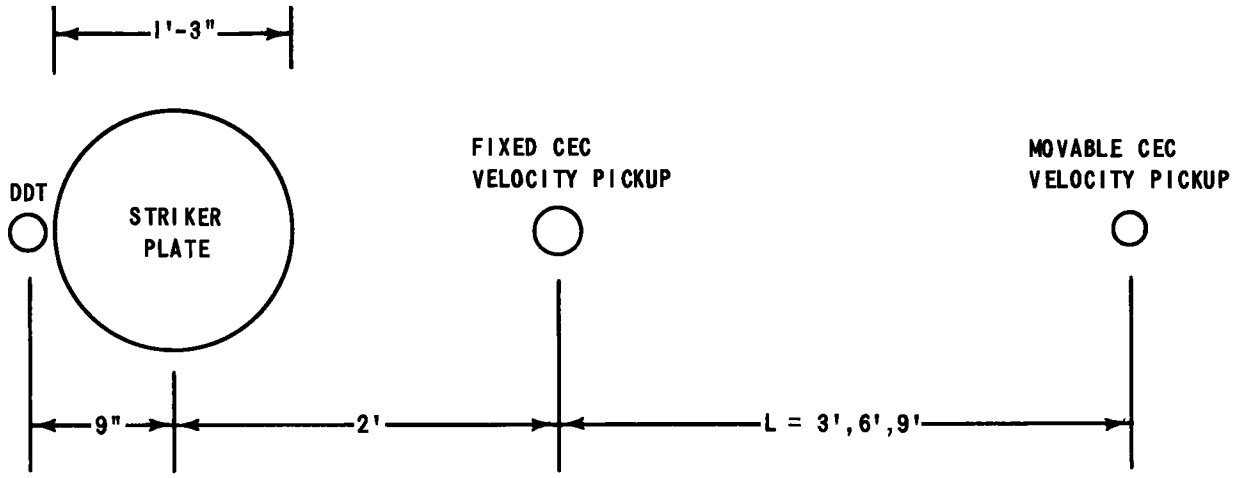


Figure 13. Location of sensors.

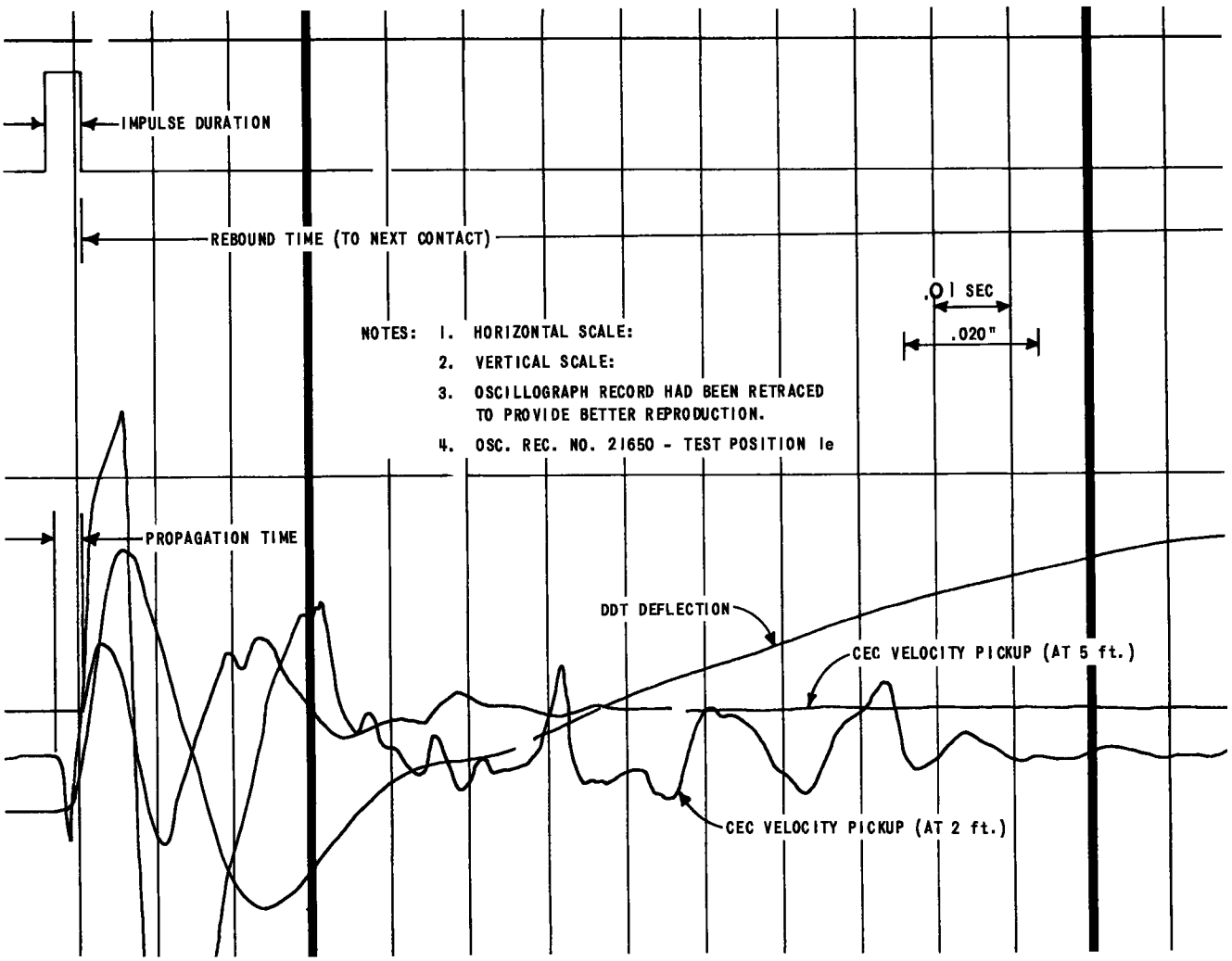


Figure 14. Example of recorded response of poor road.

(b) It was observed that no water was present near the good road, whereas water was present in the ditch of the poor road during spring thaw.

(c) No tests, in this feasibility study, were made during the winter months when the roads were frozen.

Two transverse stations 40 ft apart were selected at each test site. The test point identification system is shown in Figure 12 for the poor road. The stations are designated 1 and 1A and the 3 transverse positions at each station are identified by subscripts e (edge), m (middle), and c (center). The stations for the good road are identified as 2 and 2A. Occasionally, data were taken at a station midway between the two shown in Figure 12. This station is 1B for the poor road or 2B for the good road.

In all of the tests performed the test stations were marked and the striker plate repositioned exactly from week to week, month to month, etc. The location (see Fig. 7, 11, and 13) of the dynamic displacement transducer (9 in. from the center of the striker plate) and the drop height (1 ft) of the bar were likewise held fixed. Three drops were made at each station, after two trial drops to settle the striker plate. CEC oscillograph records were obtained in all tests.

In the wave propagation setup as shown in Figure 13, one CEC velocity pickup (Model 4-102-A) was held fixed (2 ft from the center of the striker plate) and the other was placed, along a radial line, at 5 ft from the center of the striker plate for the first drop and then moved to 8 ft and 11 ft away from the center for the second and third drops, respectively. These resulted in propagation distances, L, of 3 ft, 6 ft, and 9 ft between the CEC velocity pickups. All velocity records were obtained with the CEC oscillograph and the propagation time determined from the timing marker traces in the recorder. As can be seen from the oscillograph records, the effects of external vibrations on the test results were negligible, as the bar was dropped only after the road vibrations died out.

The impulse duration and rebound time were likewise obtained from the CEC oscillograph records and the timing markers.

Examples of the recorded data are shown in Figure 14 for the poor road and Figure 15 for the good road. In these records the time between the grid lines is 0.01 sec.

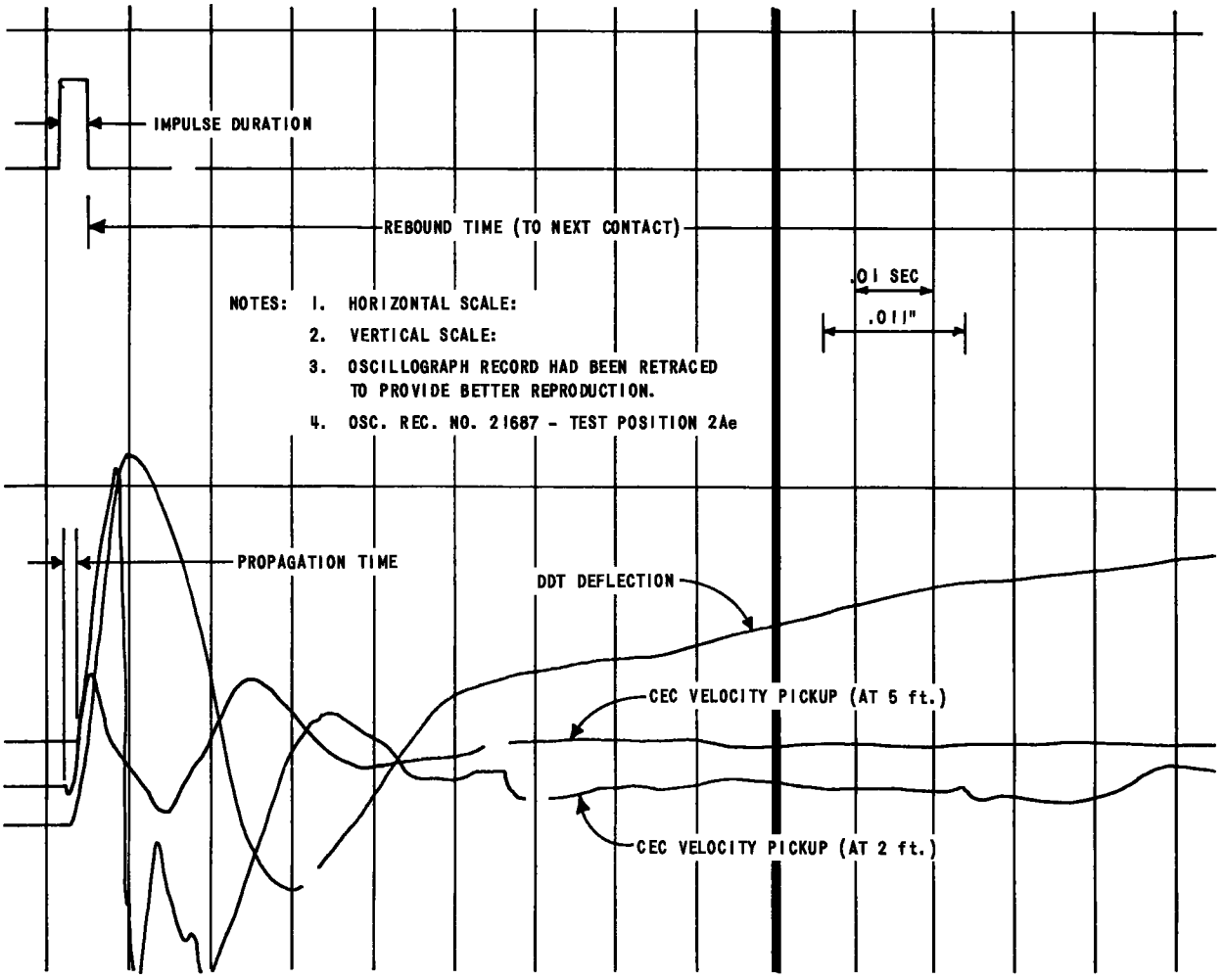


Figure 15. Example of recorded response of good road.

used, (b) the physical measurements to be made, (c) the number and location of physical measurements, and (d) the methods of correlating the data obtained via impulsive loading techniques with known measures of load-carrying capacity.

Phase 2.—Assuming that the results of the Phase 1 activity are promising, design and construct an experimental road impulse generator. Examine the possibility that with little or no modification the existing impulse generator design will be applicable to the present task.

Design and construct, or purchase, specific instrumentation pickups for measuring the chosen road response variable(s).

Phase 3.—Conduct a series of tests on flexible pavements in order to prove out the impulsive loading technique, the content of the program, to be conducted in the Buffalo area, to be agreed upon by CAL and the sponsor at such time in the course of the project as it becomes feasible to do so.

CHAPTER TWO

DESCRIPTION OF WORK PERFORMED AND FINDINGS

LITERATURE SEARCH

The first portion of the literature survey was mainly directed toward the search for approximate theories and experimental data on the response of flexible pavements to dynamic loads. A later portion was directed toward the search for formulas and experimental data involving propagation of alastic waves which might be pertinent to impulsively loaded flexible pavements.

Among the many studies reported on pavement dynamics, the following information was found to be more pertinent.

Seasonal Variations in Load-Carrying Capacity of Flexible Pavements

Figure 1 is based on the work of Motl (*3*), which shows a considerable drop in bearing capacity starting in the middle of March (when the spring thaw begins), a bottoming around early April, and considerable recovery by the end of May. These data were used in planning the field test program and were also helpful in the interpretation of test data.

Flexible Pavement Considered as an Equivalent Single Degree of Freedom System

As a first approximation, a flexible pavement may be considered as a single degree of freedom system consisting of an equivalent spring, k, an equivalent mass, m, and an equivalent damper, c, as shown in Figure 2. The displacement response, $u(t)$, due to an impulse $I\delta(t)$ is (5):

$$u(t) = \frac{I}{k}\frac{\omega^2}{\omega_c}\exp\left(-\frac{c}{2m}t\right)\sin\omega_c t \qquad (1)$$

in which

$\omega = \sqrt{k/m}$;

$\omega_c = \omega\sqrt{1-\zeta^2}$;

$\zeta = c/2\sqrt{k\,m}$;

$\delta(t) =$ Dirac function; and

$I =$ magnitude of impulse, in lb-sec.

One source (*6*) gives values of the system constants of $k \approx 3{,}910{,}000$ lb/in, $m\,g \approx 24{,}600$ lb, and $c \approx 4{,}060$ lb-sec/in.

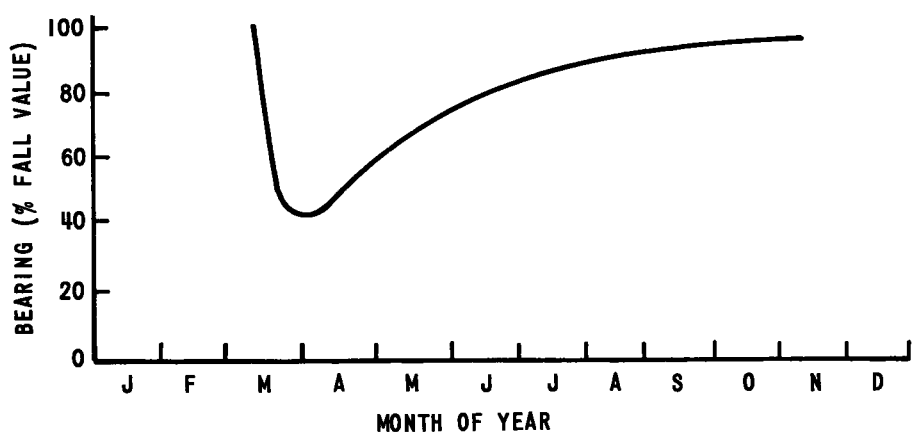


Figure 1. Variation of bearing capacity with season (after Motl).

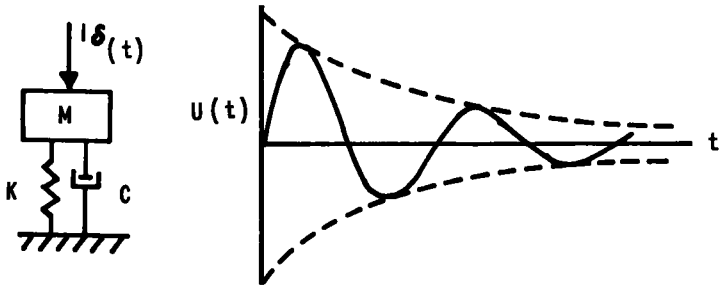


Figure 2. Response to an impulse.

Different values for these constants may be found in other sources (*13* and *14*), and k values are stated as ranging from 115 tons/cm (for unsurfaced, well-compacted, silty clay) to 400 tons/cm (for heavy airfield flexible pavement). These values were obtained through vibratory tests.

No reliable data were found on impulsive loading of flexible pavements, although some preliminary work has been reported by French investigators (*15*).

Flexible Pavement Considered as a Multi-Degree of Freedom System

A proposal has been made (*6*) to represent the pavement by a series of masses, springs, and dampers. Because of the complexity of experimental verification, this representation, which is given in more detail in Appendix A, was not pursued in greater depth in this study.

Surface Wave Propagation Velocities in an Ideal Elastic Half-Space

At a far distance from a disturbance, three types of surface waves are propagated in an elastic half-space (*7*). These are the longitudinal (P), transverse shear (S), and Rayleigh (R) waves, whose propagation velocities are, respectively,

$$C_P = \sqrt{\frac{E(1-\nu)}{(1+\nu)(1-2\nu)\rho}} \tag{2}$$

$$C_S = \sqrt{G/\rho} \tag{3}$$

$$C_R \approx 0.955\, C_S \; (\nu = 0.5) \tag{4}$$

in which

C = wave propagation velocity;
E = Young's modulus of elasticity;
G = shear modulus;
ν = Poisson's ratio; and
ρ = mass density.

Experimental and calculated Rayleigh wave velocity data from the literature for bituminous pavement ranged from about 5,000 to about 16,000 in/sec. The range of values was obtained from the vibratory tests of Heukelom and Foster (*16*) and calculated from the data given by Yoder (*3*) for E, ρ, and G.

Response of an Ideal Elastic Half-Space to an Impulsive Force

Lamb (*8*), in his paper on the propagation of tremors over the surface of an elastic solid, has calculated the displacement response due to a surface point source defined by

$$F(t) = \frac{\overline{R}}{\pi}\frac{\tau}{t^2+\tau^2} \tag{5}$$

in which

$F(t)$ = impulsive force;
t = time; and
$\overline{R}$, π, τ = constants.

His solution, at a far distance from the source, shows the arrival of a compression or P wave (minor tremor), and a period of quiescence followed by a Rayleigh or R wave (main shock). The shape of the impulsive force and the solution are shown in Figure 3. The choice of zero time, at the peak of the impulsive force, in Figure 3(a) was made by Lamb to take advantage of symmetry, which simplified the procedures in his analytical solution plotted in Figure 3(b).

APPROXIMATE EXPRESSIONS RELATING RESPONSE TO PAVEMENT VARIABLES

A number of approximate relationships were investigated relating the properties of the pavement to the dynamic responses. Some of these are discussed in the following.

Maximum Dynamic Displacement of an Undamped Equivalent Single Degree of Freedom System

The maximum displacement of an undamped equivalent single degree of freedom system (see Fig. 2) subjected to an ideal impulse of magnitude I is found to be

$$u_{max} = \frac{I\omega}{k} = \frac{I}{\sqrt{km}} \tag{6}$$

and is dependent on the equivalent spring if the impulse magnitude and the equivalent mass are assumed constant. The equivalent spring constant, k, is in turn dependent on the moduli of elasticity and rigidity of the pavement layers and subgrade. For example, if an impulse I = 125 lb-sec (see Eq. 8 and calculations) is applied to an extremely good

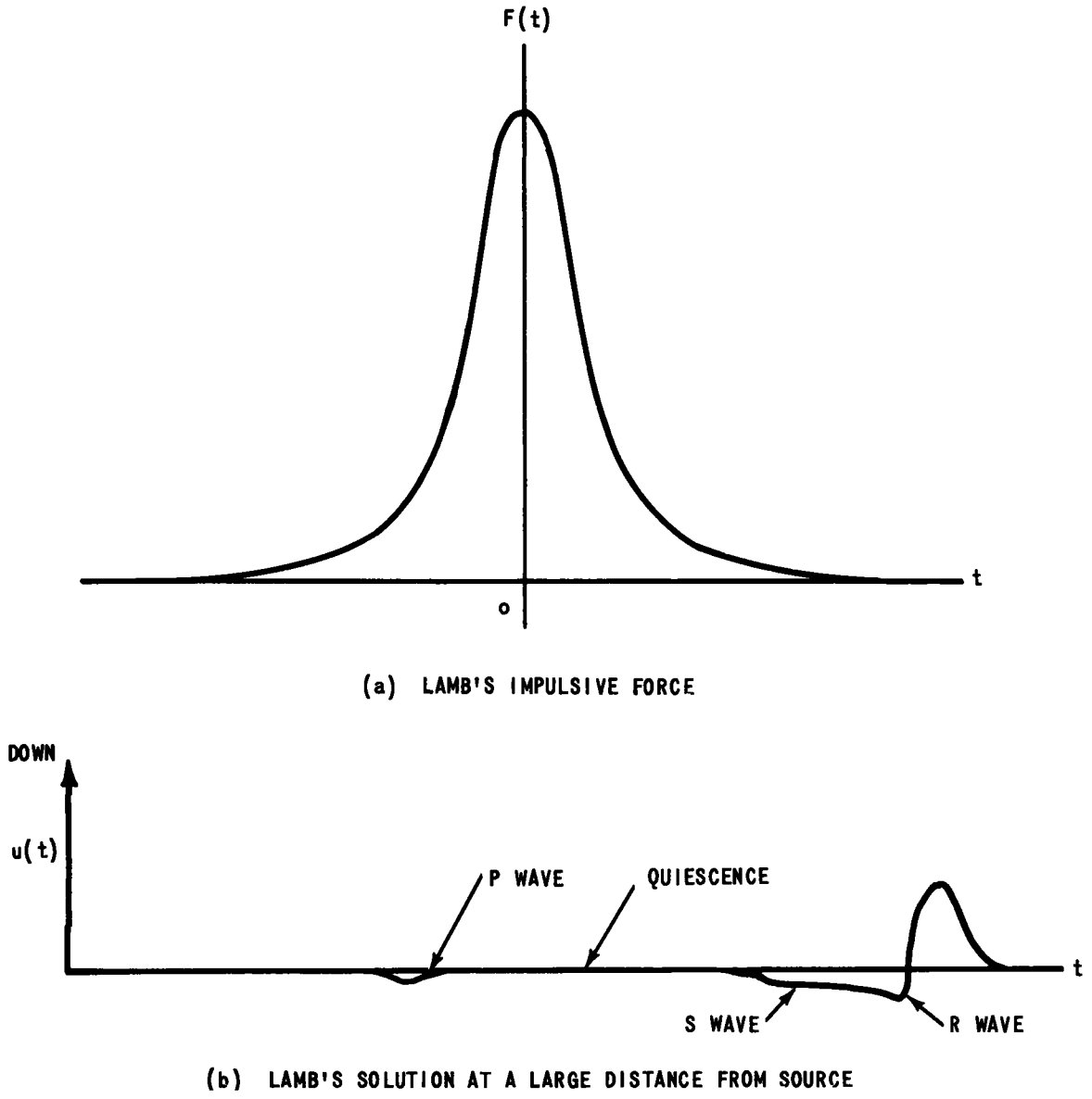


Figure 3. Lamb's propagation of tremors over the surface of an elastic solid.

road with $k = 3.91 \times 10^6$ lb/in. and $m = 62.1$ lb-sec²/in., then $u_{max} \approx 0.008$ in. If, on the other hand, it is assumed that the natural frequencies remain the same (as observed in the vibratory tests), then a loss of one-half the spring rate (*14*) would mean a doubling of the maximum deflection. The foregoing values are the basis of the design of the impulse generator and the instrumentation system.

Impulse Energy — Maximum Dynamic Displacement Relationship

Consider an elastic half-space (Fig. 4) subjected to impact by a bar of mass m_b falling from a height h_1 and rebounding to a height h_2. The impulse energy, U_I, imparted to the road is

$$U_I = m_b\, g\, (h_1 - h_2) \tag{7}$$

Using the impulse momentum equation of dynamics, the corresponding force impulse is

$$I = m_b\, (\sqrt{2\, g\, h_1} + \sqrt{2\, g\, h_2}) \tag{8}$$

If $m_b\, g = 500$ lb, $h_1 = 1$ ft, and $h_2 = 0$, then $I \approx 125$ lb-sec. This impulse magnitude was used in the highway tests.

Eliminating the rebound height, h_2, from Eqs. 7 and 8 gives

$$I = m_b\, (\sqrt{2\, g\, h_1} + \sqrt{2\, g\, [h_1 - U_I/(m_b\, g)]} \tag{9}$$

Thus, the relationship between the impulse energy imparted to the road and the maximum deflection for an undamped equivalent single degree of freedom system is

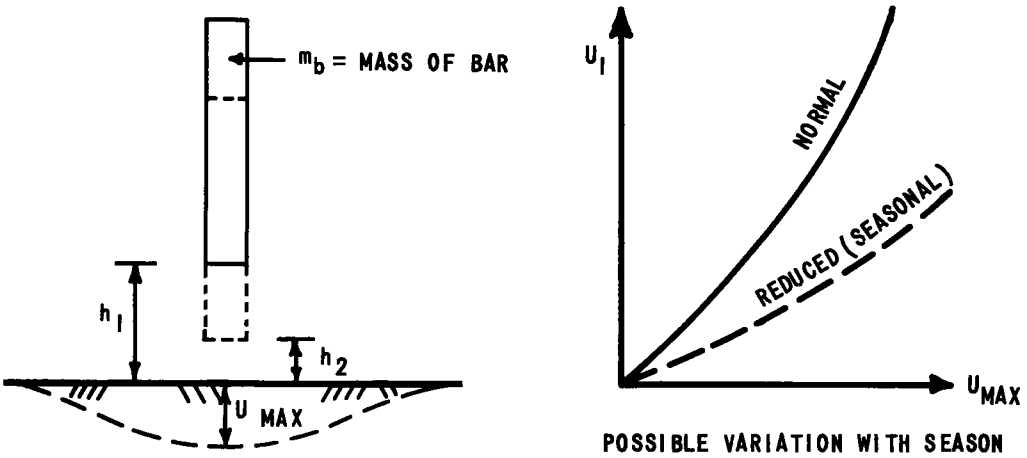


Figure 4. Relationship between impulse energy and maximum deflection.

$$u_{max} = \frac{m_b\left(\sqrt{2\,g\,h_1} + \sqrt{2\,(g\,h_1 - U_I/m_b)}\right)}{\sqrt{k\,m}} \qquad (10)$$

Correlation Between Impulsive and Static Loading Techniques

Consider again an elastic half-space (Fig. 5) subjected to a static load P and deflected by an amount u_{st}. The elastic energy stored in the road is

$$U_{st} = \tfrac{1}{2}\,P\,u_{st} \qquad (11)$$

Assuming a perfectly elastic impact, this static load energy can be equated to the impulse energy imparted to the road to yield

$$U_I = \tfrac{1}{2}\,u_{st}\,P \qquad (12)$$

It was not possible to verify this relationship experimentally because no static load-deflection data were available for the roads tested.

Wave Velocities in an Elastic Half-Space

The waves and wave velocities are fairly simple phenomena if one considers a point which is at a far distance from the point of impact. These velocities were discussed previously, under "Literature Search."

The surface waves near the point of impact are complicated and are difficult, if not impossible, to isolate from each other. Because of this, and the fact that flexible pavements are far from being an elastic half-space, approximate expressions for wave phenomena near the point of impact of an elastic half-space were not derived.

Response of an Ideal Elastic Half-Space to an Impulsive Force

The response of a point at a far distance from the impact has been discussed under "Literature Search." For the same reasons given for wave velocities, the development of ap-

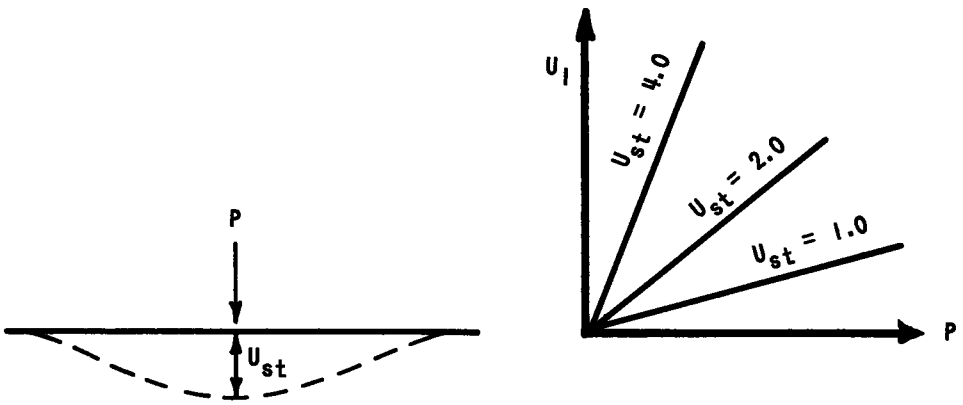


Figure 5. Correlation between impulsive and static loading techniques (ideal elastic half-space).

FOREWORD

By Staff

Highway Research Board

Highway engineers continue to seek reliable methods for detecting the effects of seasonal changes in the load-carrying capacities of flexible pavements. This report will be of interest in this regard for it describes a research effort which has attempted to develop a method adequate to the need. Knowledge has been gained which sheds light on the feasibility of formulating a technique around the concepts of studying pavement behavior under impulsive loading.

Spring thaws and other periods of adverse weather periodically necessitate reduction of load limits on many thousands of miles of existing pavements. Current methods for setting these restrictions are mostly empirical in nature and ultimately depend on the application of engineering judgment to the various factors of time for application of the load limit, the reduction in axle or wheel loading, the duration of the restriction, and the safe time for lifting the restrictions. A need thus exists for a rapid, simple, nondestructive, and accurate method which will indicate the relative load-carrying capacity of pavements when compared with capacities during fall or other seasons so that restrictions in load limits can be more objectively applied.

The Cornell Aeronautical Laboratory has researched this problem in terms of investigating the displacement response of flexible pavements to impulsive loadings as a measure of the seasonal changes in the elastic properties. The scope of the research encompassed (1) an analytical phase to test the feasibility of impulsive loading techniques when idealizing the pavement system as an elastic half-space and assuming a single degree of freedom system, (2) the design and construction of an impulse generator, and (3) a series of field tests on existing roads with and without load restrictions. Of particular interest are the conclusions drawn as to the dynamic displacement variable which appears to be the most promising as a measure of the changes in load-carrying capacity.

This document constitutes a final report on the research, and it recognizes that additional research is necessary to realize a total solution to the problem in terms of utilizing impulsive loading techniques. Recommendations for such research have been made on the basis of the study. It is particularly felt that further research should be concentrated on (1) the development of equipment possessing the desired attributes of simplicity, rapidity, etc., (2) large-scale field testing to provide an adequate statistical sample, and (3) the development of a final field technique to establish standards which relate dynamic deflection readings and load-carrying capacity.

CONTENTS

FIGURES

TABLES

ACKNOWLEDGMENTS

The research reported herein was performed under Nelson M. Isada as Principal Investigator, acting for the Transportation Research Department of the Cornell Aeronautical Laboratory.

Special acknowledgment is made to E. North, Deputy Highway Commissioner, Erie County (N.Y.) Highway Department, for assistance in the choice of the test sites and in directing traffic; and to CAL staff members T. McDade, who designed the electrical instrumentation, F. Dell'Amico, who supervised the project, and S. Smith, A. Pulley, and R. Patterson, who performed the field tests.

The Laboratory furnished facilities and support conducive to the effective prosecution of the work.

DETECTING VARIATIONS IN LOAD-CARRYING CAPACITY OF FLEXIBLE PAVEMENTS

SUMMARY

The study reported herein was made to determine the feasibility of detecting seasonal variations in the load-carrying capacity of flexible pavements by using an impulsive loading technique. This study was implemented in a three-phase program.

The first phase involved analytical investigations to establish the applicability of impulsive loading techniques. The chosen technique involves dropping a bar onto a striker plate and measuring the vertical dynamic displacement of the pavement, the rebound time of the bar, the duration of contact between the bar and the striker plate (impulse duration), and the wave propagation time. This phase concluded with a method of correlating the impulsive loading technique with a static loading technique for an idealized elastic half-space.

The second phase consisted of the design and construction of a towable impulse generator, a portable dynamic displacement transducer, and instrumentation and recording equipment. For the measurement of propagation time it was decided to use commercial velocity pickups and a recording oscillograph, with a built-in timing mechanism, because of their availability at the research agency's laboratories.

The third phase of the program consisted of a series of field tests. One test road (designated the "poor" road) was an old poorly-drained flexible pavement on which load restrictions are regularly imposed and the other (designated the "good" road) was a newly-built road without load restrictions. These tests showed a seasonal variation of about 70 percent in the dynamic displacement transducer (DDT) first peak deflection for the poor road and a slightly higher percentage for the corrected DDT deflection (a measure of absolute displacement). The tests also showed the poor road deflection to be about 2½ times as large as that of the good road. The rebound time and impulse duration showed results that reflected seasonal variations, but to a lesser degree, whereas the data on wave propagation time are inconclusive.

Based on these tests, it is concluded that the impulsive loading technique is a feasible method of detecting seasonal variations in the load-carrying capacity of flexible pavements, the first peak deflection indicated by the dynamic displacement transducer (DDT) being the most promising variable to be measured. The technique also appears to be a promising method of distinguishing a poor road from a good one.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

The research problem statement for the project reported here reads as follows:

> Many thousands of miles of roads are in existence which require reductions of load limits during spring thaw or other adverse periods. Extensive research already reported gives details of the effect of frost and other adverse climatic factors on pavement strength. The time for application of a load limit, the reduction in axle or wheel loading, the duration of the restriction, and the safe time for lifting the restrictions are based on engineering judgment. A need exists for an accurate method which will indicate the relative load-carrying capacity of pavements when compared with capacities during fall or other seasons so that restrictions in load limits can be more objectively applied. Such a procedure should be rapid and simple in operation and nondestructive to the pavement. Existing procedures do not appear to offer the desired solution to this problem.

The objective of that same research problem statement was "development of rapid and simple instrumentation for detecting seasonal variations in the load-carrying capacity of pavements."

The discussion given in the Cornell Aeronautical Laboratory (CAL) prospectus (*1*), issued in response to the project statement, is based on the fact that flexible pavement failures are due to cracking and excessive permanent deflection, as pointed out by Monismith (*2*) and discussed in detail by Yoder (*3*). These failures suggest that seasonal variations in load-carrying capacity are due to two basic causes, as follows:

1. Seasonal variations in the elastic properties of the pavement layers.
2. Seasonal variations in the ultimate strengths of the pavement layers and subgrade.

Ultimate strength test methods require trenching of the pavement and the testing of each layer to destruction, either in the field or in the laboratory. There appears to be no known method of nondestructive ultimate strength testing.

Seasonal variations in elastic properties can be determined by nondestructive testing techniques such as plate bearing tests, vibrating tests, sonic tests, and impulsive loading tests. The plate bearing technique is well known and widely used, but is found to be slow and expensive and to require bulky equipment. The vibration technique uses a shaker which applies a sinusoidal load to the pavement. At a given frequency of excitation, deflection and wave velocity measurements are made. The sonic test has been used, but primarily for the purpose of measuring road parameters like thickness, density, etc.

The Cornell Aeronautical Laboratory, under previous research in road loading mechanics, had developed an impulse-load generator for use as a research tool in the study of dynamic loading in rigid pavements (*4*). The device consists of a mechanism by which a steel bar of known weight can be dropped and caught on the first bounce. The bar strikes a base element which imparts an impulsive load to the road through an annular area, thus permitting deflection, velocity or acceleration measurements to be made directly under the load. The time duration of the impulse generated by the device is of the order of 0.001 sec.

It appeared that an impulsive loading technique, using a device of the type just described, offered promise of providing a simple, rapid and nondestructive test method, such as that being sought under this project. Accordingly, it was proposed to investigate this technique by analytical methods and, if the method proved to be promising, to construct an experimental impulse generator and the necessary instrumentation, and conduct field tests to demonstrate the feasibility of the concept.

Development of a practical field-operational machine was not contemplated in the proposal.

RESEARCH APPROACH

To meet the broad objective sought in the project and to establish the feasibility of the impulsive loading technique in particular, the following objectives were listed in the original prospectus:

1. To determine analytically the feasibility of detecting seasonal variations in the load-carrying capacity of flexible pavements by using impulsive loading techniques.
2. If this technique is found to be promising, to design and construct an experimental impulse generator, including the necessary instrumentation.
3. To perform a series of field tests to develop the measuring techniques and to evaluate the effectiveness with which the impulsive loading method can be used to detect seasonal variations in the load-carrying capacity of flexible pavements.

To attain these objectives the prospectus defined a program outline and a three-phase working plan, as follows:

Phase 1.—Conduct analytical investigations designed to establish the applicability of impulsive loading techniques to the task of determining the load-carrying capacity of flexible pavements.

Utilizing existing theoretical formulations of the problem of impulsive loading of an elastic half-space and by extending this theory in an approximate fashion, if necessary, determine the optimum loading measurement technique. Some questions to be resolved are (a) the kind of loading to be

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Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

THE NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

THE NATIONAL RESEARCH COUNCIL was organized as an agency of the National Academy of Sciences in 1916, at the request of President Wilson, to enable the broad community of U. S. scientists and engineers to associate their efforts with the limited membership of the Academy in service to science and the nation. Its members, who receive their appointments from the President of the National Academy of Sciences, are drawn from academic, industrial and government organizations throughout the country. The National Research Council serves both Academies in the discharge of their responsibilities.

Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

THE DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

THE HIGHWAY RESEARCH BOARD, organized November 11, 1920, as an agency of the Division of Engineering, is a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.