EVALUATION OF CHEMICALLY STABILIZED SECONDARY ROADS

by

L.D. Squier, J.M. Hoover, & R.L. Handy

May 1974

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PROGRESS REPORT 1

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ISU–ERI–AMES–74104
ERI Project 1049-S

ENGINEERING RESEARCH INSTITUTE
IOWA STATE UNIVERSITY AMES

A cooperative research project in affilliation with:

Allis-Chalmers Construction Machinery Division

American Admixtures

American Can Company

Armak Highway Chemicals Department

Bitucote Products Company

CIBA-Geigy

Del Chemical Corporation

Dow Chemical U.S.A.

Emulsified Asphalts, Inc.

Flambeau Paper Company

ITT Rayonier, Inc.

Koehring Road Division

Linn County, Iowa

Linn County Engineer, W. G. Harrington, P.E.

Linwood Stone Products Company, Inc.

Macklin Inc.

National Ash Association

National Chemical Stabilization Association

National Lime Association

Salt Institute

Sandar Inc.

Saunders Petroleum Company

Scott Paper Company

This project was also partially supported by the Engineering Research Institute at Iowa State University, Ames, Iowa, 50010.

Introduction

For many years engineers have been studying new ways of constructing all-weather road surfaces that are low cost, durable, and easy to build.

The research project reported herein is concerned with field and laboratory analysis of some of the more common, and also some not so common, additives for the stabilization of roads. These additives, when applied to a roadway, will hopefully provide a lower cost alternate to more expensive, high-type pavements utilized in secondary road systems.

Three methods of construction were used on this project: (1) Dust palliation through surface application of additives, (2) an intermediate measure with additives mixed in to a depth of 4 inches, and (3) additives mixed to depths of 6-8 inches with possible subgrade treatment. A seal coat surface was applied to each, or a portion of each test section constructed under (3) while those test sections under (2) received no seal coat to full seal coat surfacing.

Three basic phases make up the research project: (1) Construction and construction control of the 1000 ft. long test sections; (2) field testing of the test and control sections along with performance evaluations, and (3) laboratory evaluations of the untreated and treated soils taken from the test sections.

Construction

Construction of the test sections began June 22 and was concluded July 16, 1973. Weather during construction was good, with moderate to warm temperatures and little rain.

Construction was carried out by Linn County personnel using the necessary county equipment as well as the MPH-1 stabilizer-mixer and the static and vibratory steel drum rollers provided by Koehring Company. A motor grader was also provided by Allis-Chalmers Company. All additives were donated by the individual producer and/or their agent. Supervision of construction was basically in the hands of each additive producer and/or their agent, with limited input or consultation from the Linn County Engineer, or his assigned personnel, and I.S.U. representatives.

The general sequence of construction operations was as follows:

1. Scarification to the depth of desired treatment.
2. One or more passes of the stabilizer-mixer for pulverization, coupled with a spray application of water as needed to attain desired moisture content for application of additive.
3. Additives, and the percentages thereof, were added according to manufacturer's recommendations. Several methods of additive application were used ranging from dry spreading, to asphalt distributor, to water dispersion/solution from a tank truck.
4. One or more passes with the stabilizer-mixer for mix distribution of the additive with occasional additional water being added if necessary to achieve optimum moisture content for compaction.
5. Compaction was accomplished with a sheep's foot, rubber tire, static or vibratory steel drum roller depending on the soil type and the additive manufacturer's recommendation.
6. Final shaping to crown and finish blading.
7. Final compaction by static or vibratory steel drum roller for reseal of surface hair cracking due to finish blading.

Construction of the dust palliative section consisted of the following operational sequence:

1. Surface was bladed to remove irregularities.
2. Additive was applied as a spray in two applications approximately 30 minutes apart.
3. Rubber roller was used to tighten and seal the surface.

Transpiration and evaporation of moisture from some types of stabilized soils is critical to the attainment of full stabilization benefits. For those sections slated for seal coat surfacing, where evaporation losses were considered detrimental, spray applications of water were made periodically from the point of final compaction (Step 7 above) to the point of seal-coat construction.

Location and Materials

Table 1 describes the test section locations superimposed on a Linn County map, Figure 1. Table 2 presents participant company sections, product utilized, soil classification of representative samples from each treated section just prior to application of product(s), and application rates of product(s) in each test section. Table 3 presents laboratory Atterberg Limits, density and optimum moisture content data of each of the section soils.

All soil samples removed from each section for laboratory evaluation, for both I.S.U. and as requested for shipment to the participant, were representative samples made up by removal of in-place soils prior to construction, from two or more locations throughout the test site. This was in order to provide an adequate quantity of representative site soils for all lab tests being considered. As recommended by the participant, crushed stone was added to sections T-5, T-12, and T-13 at rates of 350 tons, 50 tons, and 50 tons, respectively. Thus pre-construction sampling of these three sections was repeated during construction, immediately following incorporation of the additional aggregate to base soil, and prior to additive application. The data of Tables 2 and 3 reflects the physical properties of the repeated sampling operations.

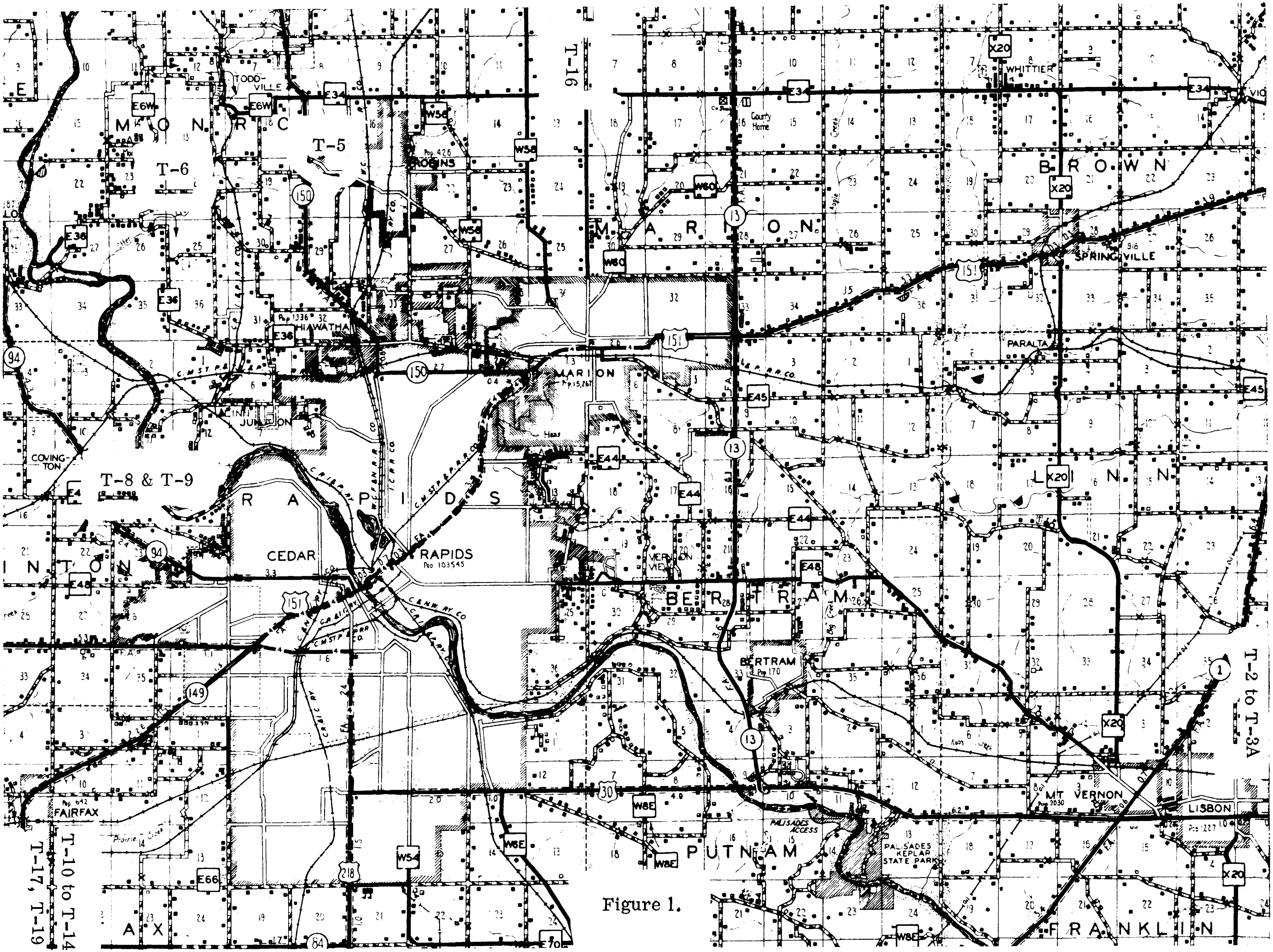
Samples of the treated materials, immediately following mixing and prior to compaction, were removed from each test section. On the site, standard density 1/30 cu. ft. specimens were molded, wrapped and sealed, then returned to the laboratory for storage in a humid room at 100% relative humidity and 72°F. A series of each of these treated specimens will be tested for unconfined compressive strength, for comparison with unconfined compressive strengths of core specimens removed from each section in late January to early February, each year.

Post-Construction Density and Moisture Content

A series of post-construction moisture tests were run with a Troxler nuclear moisture density device, a Speedy moisture meter, and the standard oven dry moisture method at 110°C. Figures 2 and 3 indicate the scatter of the data which in both cases uses the standard oven-dry moisture content as the independent

Table 1. Location Description

Test Section	
T-2	North of Lisbon, Iowa. Beginning 1000 ft. north of railroad tracks. T-2 is the first 500 ft. T-2A is the second 500 ft.
T-2A	North of Lisbon, Iowa. Beginning 1000 ft. north of railroad tracks. T-2 is the first 500 ft. T-2A is the second 500 ft.
T-3	First 500 ft. north of T-2A. T-3 is on the east shoulder.
T-3A	First 500 ft. north of T-2A. T-3A is on the west shoulder.
T-5	First 1000 ft. east of Highway 150 on the road leading into Robins, Iowa.
T-6	Three miles west of Highway 150 on Tower Trailer Road. Beginning 75 ft. west of intersection continuing 1000 ft. east.
T-8	3/4 mile east of Highway 94 on Ellis Park Road.
T-9	3/4 mile east of Highway 94 on Ellis Park Road.
T-10	First 1000 ft. north of county road E-70 on the road that leads into the east edge of Fairfax, Iowa.
T-11	Second 1000 ft. north of county road E-70 on the road that leads into the east edge of Fairfax, Iowa.
T-12	Third 1000 ft. north of county road E-70 on the road that leads into the east edge of Fairfax, Iowa.
T-13	Fourth 1000 ft. north of county road E-70 on the road that leads into the east edge of Fairfax, Iowa.
T-14	Fifth 1000 ft. north of county road E-70 on the road that leads into the east edge of Fairfax, Iowa.
T-16	First 1000 ft. north of county road E-34 on the 10th street extension north of Marion, Iowa.
T-17	Beginning at L intersection in Fairfax, Iowa and continuing 1000 ft. south towards section T-19.
T-19	Sixth 1000 ft. north of county road E-70 on the road that leads into the east edge of Fairfax, Iowa.



MONROE
T-5
T-6
T-16
TODDVILLE
ROBINS
HIAWATHA
MARION
BROWN
SPRINGVILLE
WHITTIER
PARALTA
LINN
T-8 & T-9
COVINGTON
RAPIDS
CEDAR RAPIDS
BERTRAM
VERNON VIEW
FAIRFAX
PUTNAM
PALISADES KEPLAR STATE PARK
MT VERNON
LISBON
FRANKLIN
T-2 to T-3A
T-10 to T-14
T-17, T-19

Figure 1.

Table 2. Test section data relative to sponsors, additives and classification of soil.

Section No.	Company	Material	AASHO Soil Classification	Mix depth, in.	Unified Class	Application: Quantity or rate, 1000' Section
T-2*	Scott, Flambeau, Rayonier, Macklin	Lignosulfonate	A-2-4(0)	6	SM	$1\frac{1}{2}$ gal/yd^2
T-2A*	Same & Ciba-Geigy	Lignosulfonate & Pramitol 25	A-2-4(0)	6	SM	$1\frac{1}{2}$ gal/yd^2 & 20 gal/acre
T-3*	Ciba-Geigy	Pramitol 25	A-2-4(0)	3	SM	15 gal/acre - shoulder only
T-3A*	Same	Same	A-2-4(0)	3	SM	20 gal/acre - shoulder only
T-5	Salt Institute	Sodium Chloride	A-3	6	SM	2 lbs/yd^2/inch of depth
T-6	Saunders Petroleum & Nat. Chem. Stab. Assoc.	Kelpak	A-2-4(0)	6	SM	50 gal. at 10 gal/ 1000 gal H_2O
T-8AS T-8BS	Same as T-6	Clapak/Claset	Subgrade treated, A-6-(9) (west end) A-4(3) (east end)	6	 CL ML	15 gal. Clapak/ 10 gal. Claset in 3000 gal. H_2O
 T-8AB T-9 T-8BB	Same as T-6	SA-1	Base treated A-4(1) west A-2-4(0) middle A-1-b east	6	 SM SM SM	10 gal. at 1 gal/ 1000 gal. H_2O

Table 2 continued

T-10	Armak and Emulsified Asphalts	Asphalt emulsion	A-2-6(1)	4	SC	4+%
T-11	National Lime Assoc. & Linwood Stone Products	Hydrated Lime	A-6(5)	8	CL	4%
T-12	National Ash Assoc. & Chicago Flyash Co.	Hydrated Lime & Flyash	A-6(4)	6	SC	4% Lime, 12% Flyash
T-13	Dow Chemical Co.	Liquidow	A-2-6(0)	6	SC	1/3 gal/yd^2
T-14	Bitucote Products	Asphalt Emulsion	A-6(2)	6	SC	4%
T-16	Del Chemical Co.	Terra-Seal	A-6(1)	6	SC	6 gal. at 1 gal/1000 gal. H_20
T-17	Dow Chemical Co.	Liquidow	(Surface application only)			1/3 gal/yd^2
T-19	Sandar, Inc.	Lignosulfonate & hydrated lime	A-7-6(12)	6	CL	1 gal/yd^2 + 2% lime

*500 ft. length sections.

Table 3. Atterburg Limits, standard density, and optimum moisture content of each test section soil, untreated.

Section No.	Liquid Limit, L.L., %	Plastic Limit, P.L., %	Plasticity Index P.I., %	Dry Density, pcf.	Optimum Moisture Content, %
T-2	21.6	16.3	5.3	133.	7.8
T-2A	21.6	16.3	5.3	133.	7.8
T-5	14.5		non-plastic	127.9	7.1
T-6	15.3	14.2	1.1	128.8	9.3
T-8AS*	31.7	16.5	15.2	119.5	12.9
T-8AB*	19.8	13.2	6.6	133.5	8.0
T-9 base	21.8	18.0	3.8	123.0	7.4
T-8BS*	20.0	16.8	3.2	127.5	9.5
T-8BB*	21.1	14.9	6.2	130.4	7.4
T-10	23.5	14.0	9.5	132.5	8.0
T-11	28.9	15.5	13.4	130.1	9.3
T-12	31.4	16.8	14.6	124.5	11.1
T-13	26.8	15.4	11.4	130.5	8.4
T-14	29.1	17.5	11.6	122.0	12.6
T-16	26.1	14.4	11.7	128.1	8.8
T-19	42.1	23.5	18.7	112.1	15 6

*AS represents west half of section T-8, subgrade.
AB represents west half of section T-8 base.
BS represents east half of section T-8 subgrade.
BB represents east half of section T-8 base.

variable. The dotted line on the figures represents the results of a linear regression of the data to obtain a "best fit" and also a correlation coefficient relating the data statistically to this line.

The nuclear unit required daily calibration to serve as a standardization. With this particular unit it was not possible to obtain a moisture reading without also taking a density reading. The procedure for taking the moisture reading was to set the nuclear probe at the surface and use the back-scatter principle to obtain a reading on the counter. To obtain a density reading the probe was inserted into a hole previously made in the surface to a depth where the density was desired. In this case both the back-scatter principle and direct transmission were employed. A reading is obtained from both the moisture and density trials and this reading divided by the standard calibration count for moisture and also for density will yield a density ratio and a moisture ratio. Tables provided by the manufacturer provide a wet density and a weight of water for each respective density ratio and moisture ratio. From these values the unit dry density and the moisture content can be determined.

This particular nuclear method derives its data from the hydrogen ion content in the combined soil/water mixture. Obviously any soil with high organic content or any type of asphaltic mix will give readings much too high. Note sections T-8BB, T-9, T-10, T-14 in Figure 2. These sections all have some type of asphalt in them. Section T-8BB contained portions of the old asphalt seal surface, section T-9 contained portions of the old surface seal and a slight quantity of emulsion originally mixed into the base.

A Speedy Moisture Tester is a device capable of giving moisture content readings in a matter of a few minutes. A small pressure vessel is provided into which a measured amount of a calcium carbide reagent is introduced along with a weighed amount of the soil to be tested. The calcium carbide reagent will react with the free moisture in the sample producing a gas whose pressure is displayed on a gauge calibrated to read directly as a wet percentage moisture content. The comparative results of this test with the standard oven-dry moisture

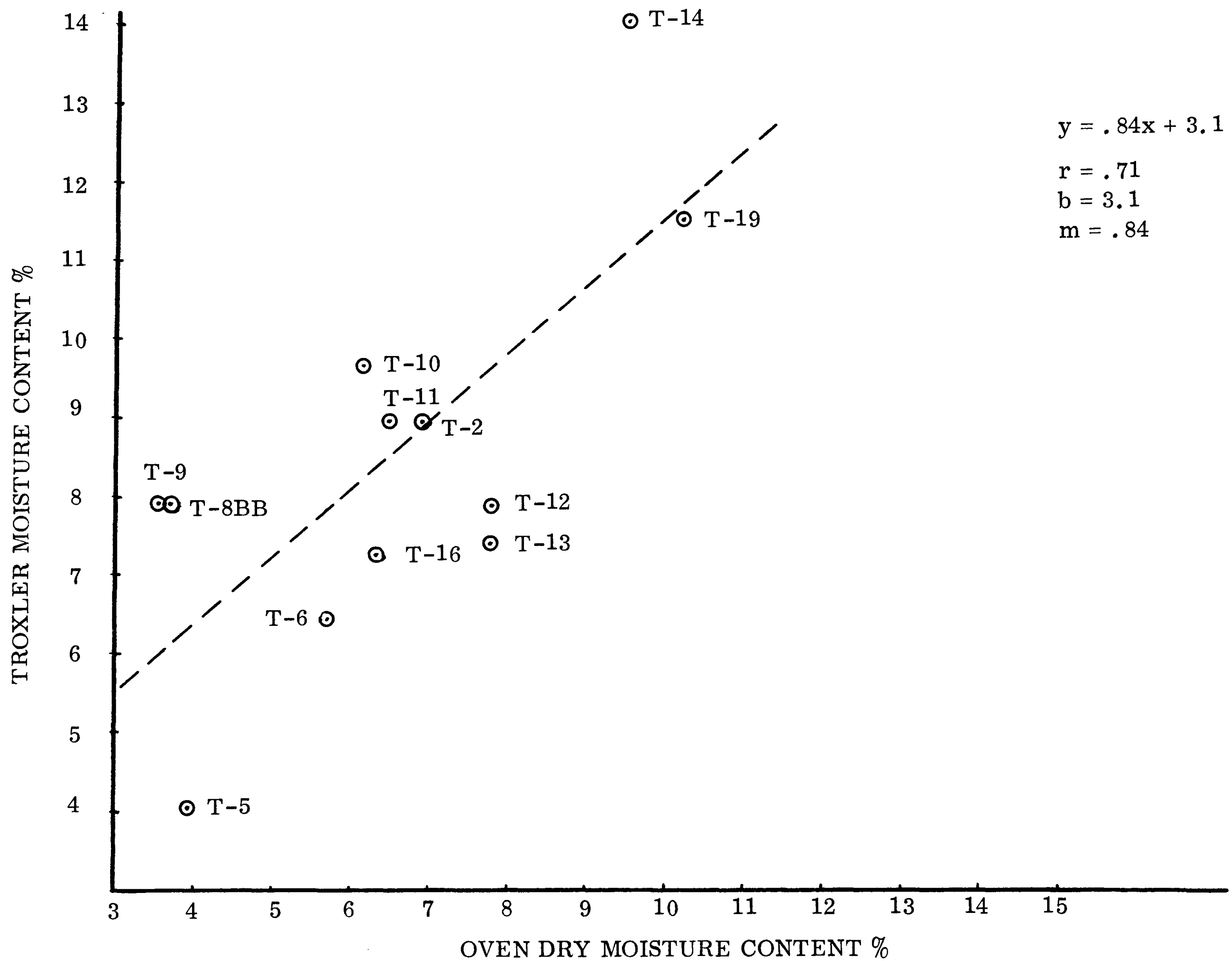


Figure 2. Plot of Troxler moisture content vs. oven dry moisture content.

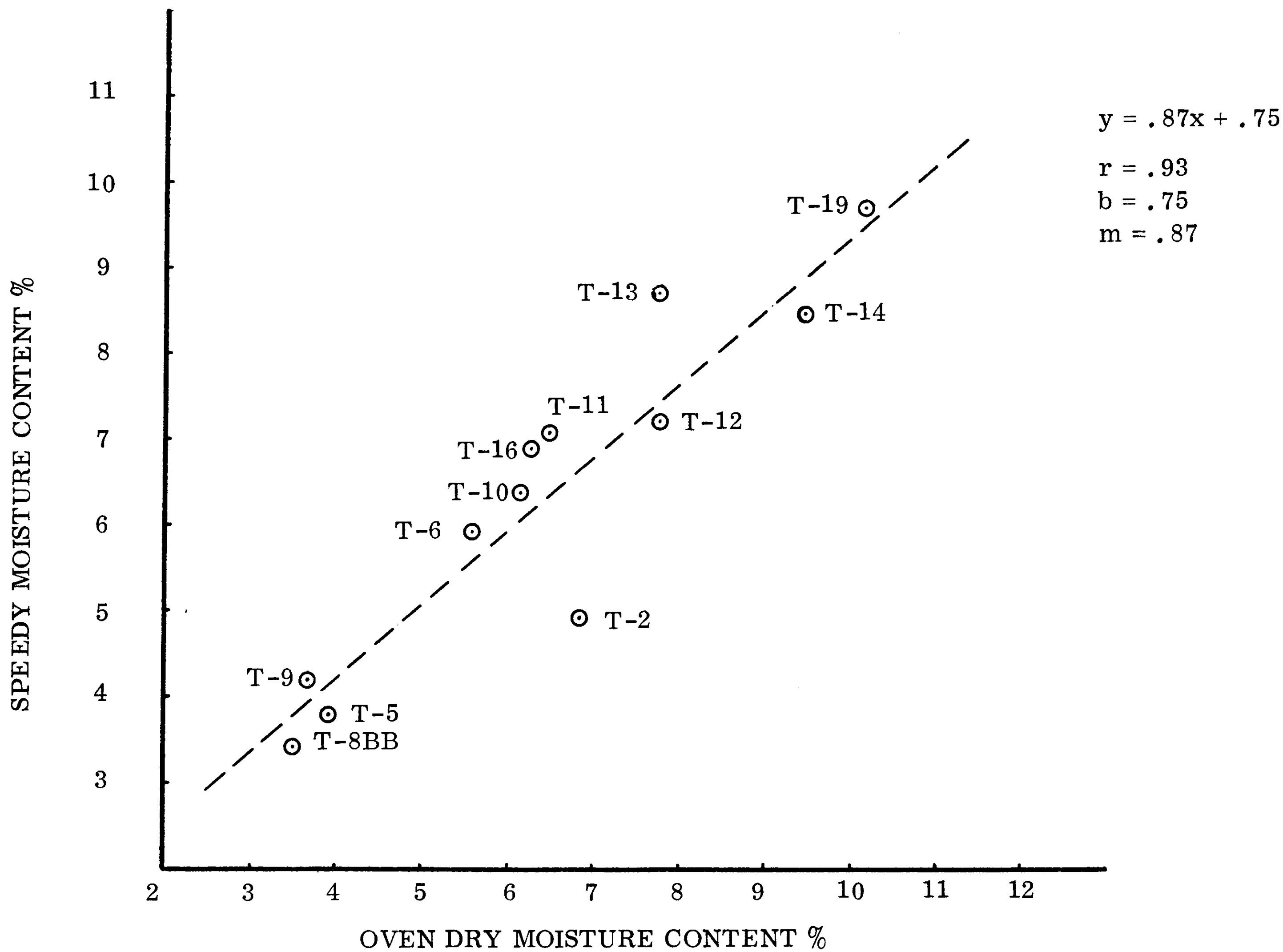


Figure 3. Plot of Speedy moisture content vs. oven dry moisture content.

are presented in Figure 3 along with the best fit line and the correlation coefficient.

It is apparent from Figures 2 and 3 that moisture content data from either of the two moisture devices was of questionable value when compared with standard determinations in a controlled temperature oven at 110^{o}C. In general, it may be observed that the carbide unit was somewhat more reliable than the nuclear device. From this short comparative evaluation it was concluded that all moisture content determinations on the test sections would henceforth be made only under standard drying oven conditions at 110^{o}C.

Comparison of post-construction oven-dry moisture contents of Figures 2 and 3 with optimum moisture contents of the untreated soils, Table 3, indicate moisture content variations between the laboratory untreated O. M. C. and field-treated conditions, of from near equal contents to significantly reduced field contents. This variation may be due to both (a) type of treatment utilized and (b) effect of time after construction before the test was conducted. With the former, laboratory moisture-density studies of the treated soils, presently underway, should confirm or deny the additives ability to reduce compaction moisture requirements. With the latter, it was occasionally necessary to withhold moisture content determination for 24 or more hours in an effort to keep ahead of construction with pre-construction in-place testing. It is known that some of the products utilized, absorb moisture as a part of the stabilization chemical reaction with soils, and it is suspicioned that several of the lesser known products may also absorb compaction water as a part of their ultimate reaction. It is anticipated that such reaction absorption will be further evaluated in the laboratory studies.

Due to time, density tests were not conducted on each section prior to construction. Post construction densities were determined by the Troxler Nuclear device. This data is presented in Table 4.

Comparison of the field density data of Table 4, and the lab untreated densities of Table 3 indicate that with few exceptions the desired field density was not achieved. This may be due to (a) inadequate compaction, (b) variations

Table 4. Field densities as determined by Troxler nuclear unit.

Section No.	Dry Density, pcf
T-2	128.6
T-2A	127.3
T-5	132.0
T-6	127.1
T-8AB	137.8
T-9	133.5
T-8BB	126.8
T-10	124.1
T-11	114.9
T-12	116.3
T-13	124.5
T-14	113.5
T-16	124.0
T-19	105.1

in field moisture versus laboratory optimum moisture content, or (c) type of product utilized since it is presently known that several of the products used cause a lowering of soil density due to the additive mechanism. Sections T-5, T-8AB, and T-9 indicate an increase in field density as compared to lab untreated soil density which may be due to a reversal of the reasoning noted above.

Additional moisture content and density determinations will be conducted at least annually on each test and control section for evaluation of change of moisture content and density versus time. Variations in field densities and moisture contents versus laboratory densities and optimum moisture contents will be further evaluated upon completion of the laboratory treated soil tests.

Benkelman Beam

In order to analyze the flexural capabilities of the test sections, Benkelman beam tests were used to measure deflection of the surface/base caused by a single rear axle wheel load of 17,300 lb. Each rear dual tire of the ERI Soils Lab load test truck was maintained at 75 psi air pressure. Since the maximum allowable single axle in Iowa is 18,000 lb., deflections thus determined were near maximum values.

Six or more observations of maximum deflection were made for each test section, two each at each of the section's quarter points. At each point of testing, deflection measurements were made of both the inside wheel track (IWT) and outside wheel track (OWT) of the load truck traveling within the normal traffic lane. All deflection measurements were then averaged for both IWT and OWT conditions and are presented in Table 5. Three series of beam deflections are reported in Table 5: (a) Pre-construction, i.e., immediately prior to construction; (b) post-construction, i.e., within 72 hours following construction; and (c) fall 1973, just prior to the first freeze, approximately three months after construction.

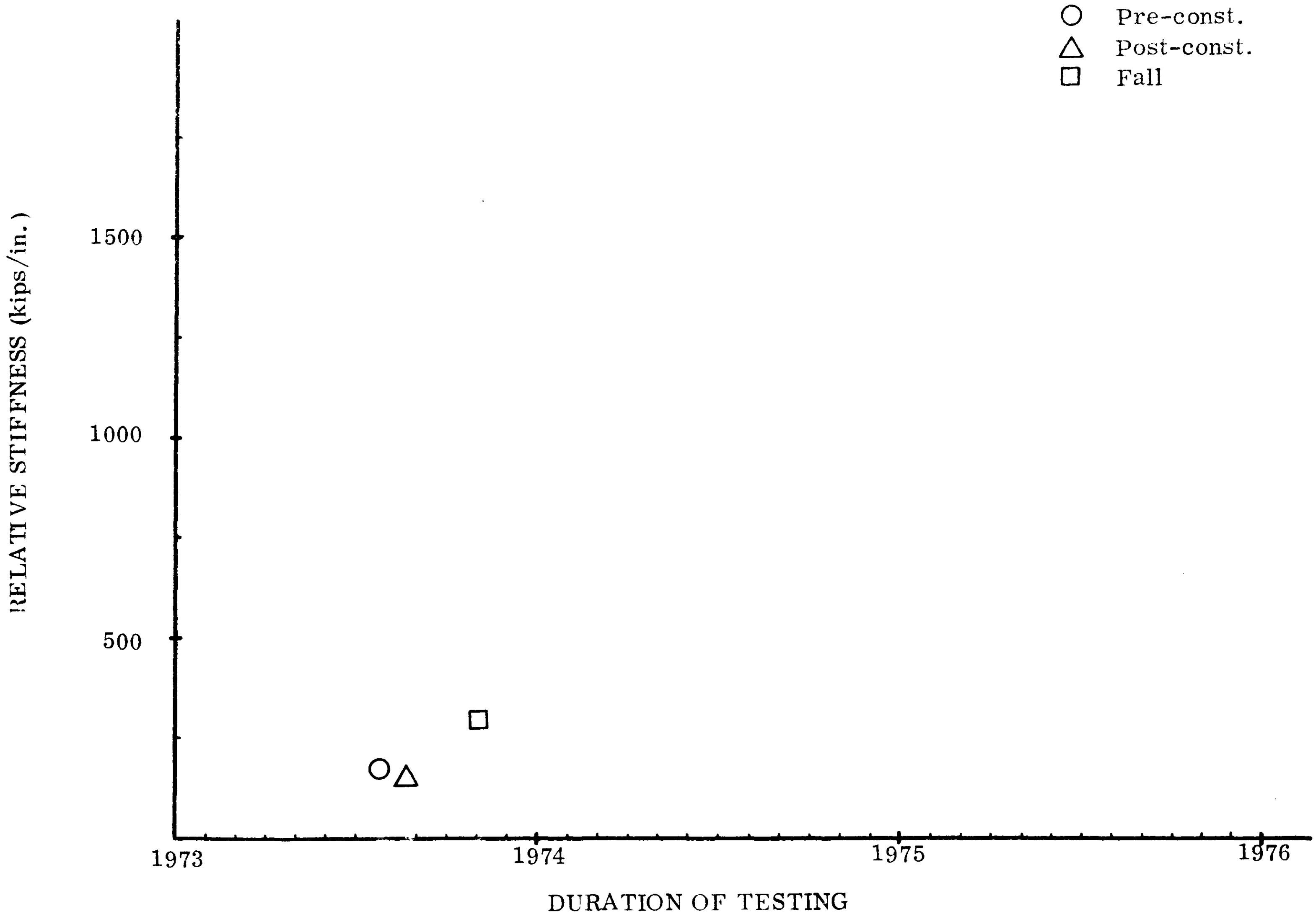
As a qualitative measure of the flexibility of each test section, IWT and OWT conditions, a relative stiffness factor was computed by dividing the axle load in thousands of pounds (kips) by the maximum deflection; the more flexible the material, the lower the relative stiffness factor. These results are also presented in Table 5. As might be expected, most of the outside edges of the roadway test sections exhibited the greatest deflection and least relative stiffness values. This is a normal situation on secondary roads, due to lower lateral restraint to surface applied stresses at the edge of the geometric cross-section of the roadway.

Figures 4 thru 17 present a plot of the relative stiffness values of each test section for the three series of Benkelman beam tests conducted in 1973.

Table 5. Benkelman beam field test results.

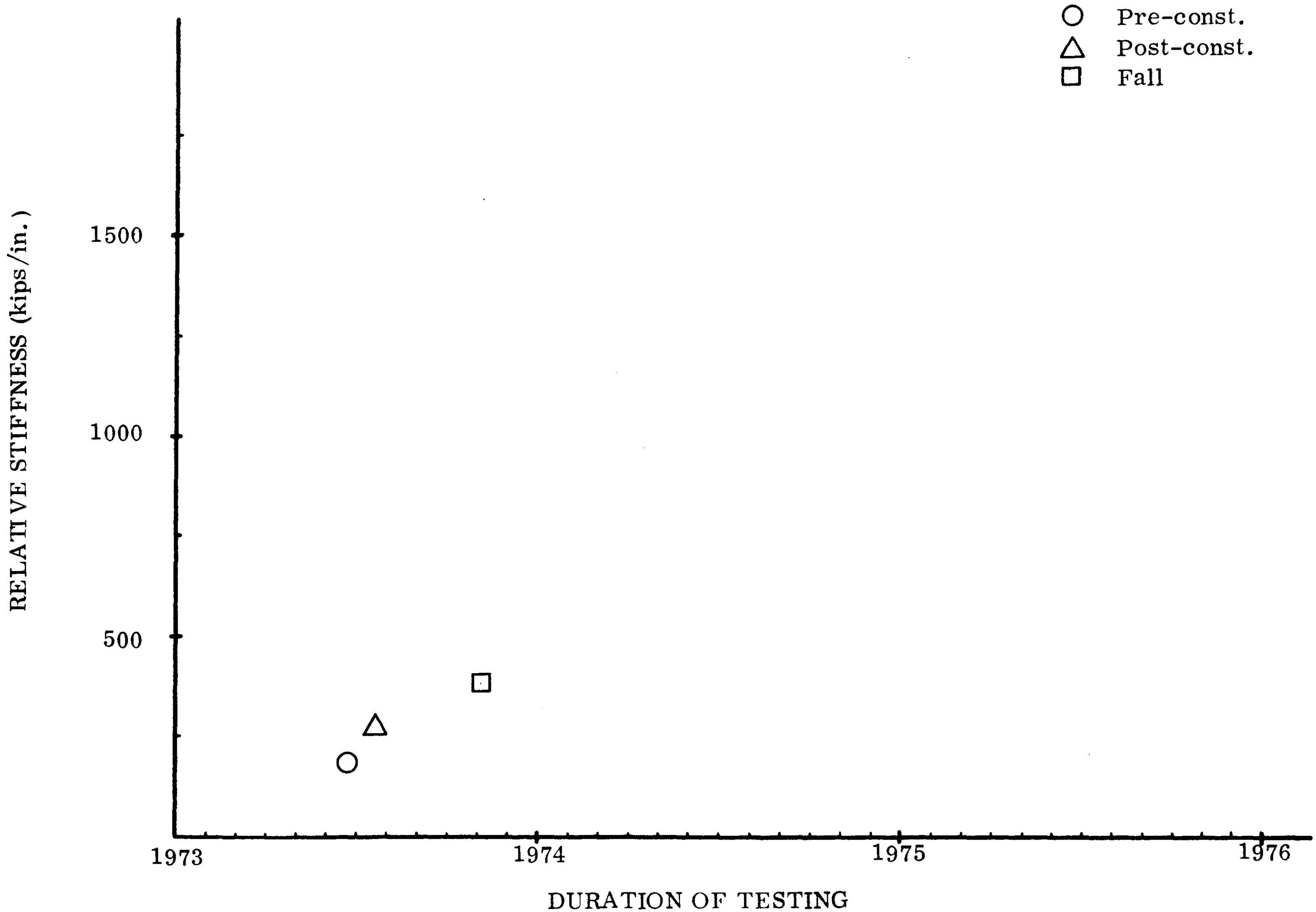
	Average Maximum Deflection, in.						Relative Stiffness, kips/in.					
	OWT*			IWT*			OWT*			IWT*		
	Pre Const.	Post Const.	Fall 1973	Pre Const.	Post Const.	Fall 1973	Pre Const.	Post Const.	Fall 1973	Pre Const.	Post Const.	Fall 1973
T-2	.109	.175	.101	.094	.092	.042	158.7	98.9	171.3	184.0	188.0	411.9
T-2A	.116	.081	.052	.076	.051	.043	149.1	213.6	332.7	227.6	339.2	402.3
T-5	.018	.019	.017	.014	.017	.016	961.1	910.5	1017.6	1235.7	1017.6	1081.3
T-6	.025	.021	.017	.015	.016	.016	692.0	823.8	1017.6	1153.3	1081.3	1081.3
T-8AB	.016	.017	.010	.010	.014.	.014	1081.3	1017.6	1730.0	1730.0	1235.7	1235.7
T-9	.016	.013	.013	.011	.010	.012	1081.3	1330.8	1330.8	1572.7	1730.0	1441.7
T-8BB	.015	.015	.014	.015	.012	.014	1153.3	1153.3	1235.7	1153.3	1441.7	1235.7
T-10	.054	.049	.026	.028	.031	.026	320.4	353.1	665.4	617.9	558.1	665.4
T-11	.043	.034	.022	.027	.026	.020	402.3	508.8	786.4	640.7	665.4	865.0
T-12	.037	.028	.020	.021	.024	.019	467.6	617.9	865.0	823.8	720.8	910.5
T-13	.041	.041	.032	.038	.037	.034	422.0	422.0	540.6	455.3	467.6	508.8
T-14	.059	.081	.046	.038	.082	.037	293.2	213.6	376.1	455.3	211.0	467.6
T-16	.036	.028	.028	.027	.038	.031	480.6	617.9	617.9	640.7	455.3	558.1
T-19	.092	.079	.043	.050	.057	.042	188.0	219.0	402.3	346.0	303.5	411.9

*OWT - outside wheel track, IWT = inside wheel track



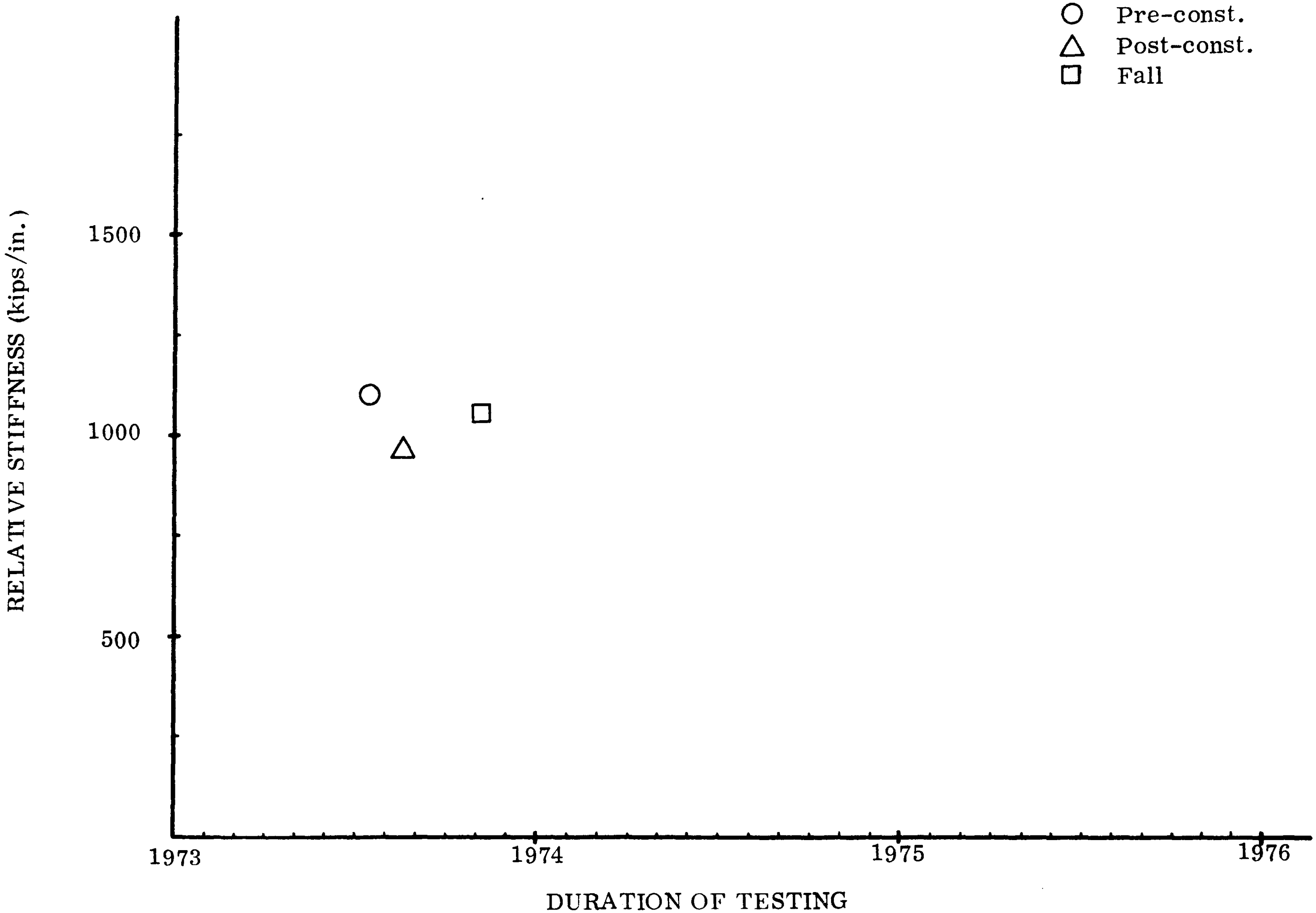
Test Section No. T-2

Figure 4. Benkelman beam-relative stiffness vs. time of year



Test Section No. T-2A

Figure 5. Benkelman beam-relative stiffness vs. time of year



Test Section No. T-5

Figure 6. Benkelman beam-relative stiffness vs. time of year

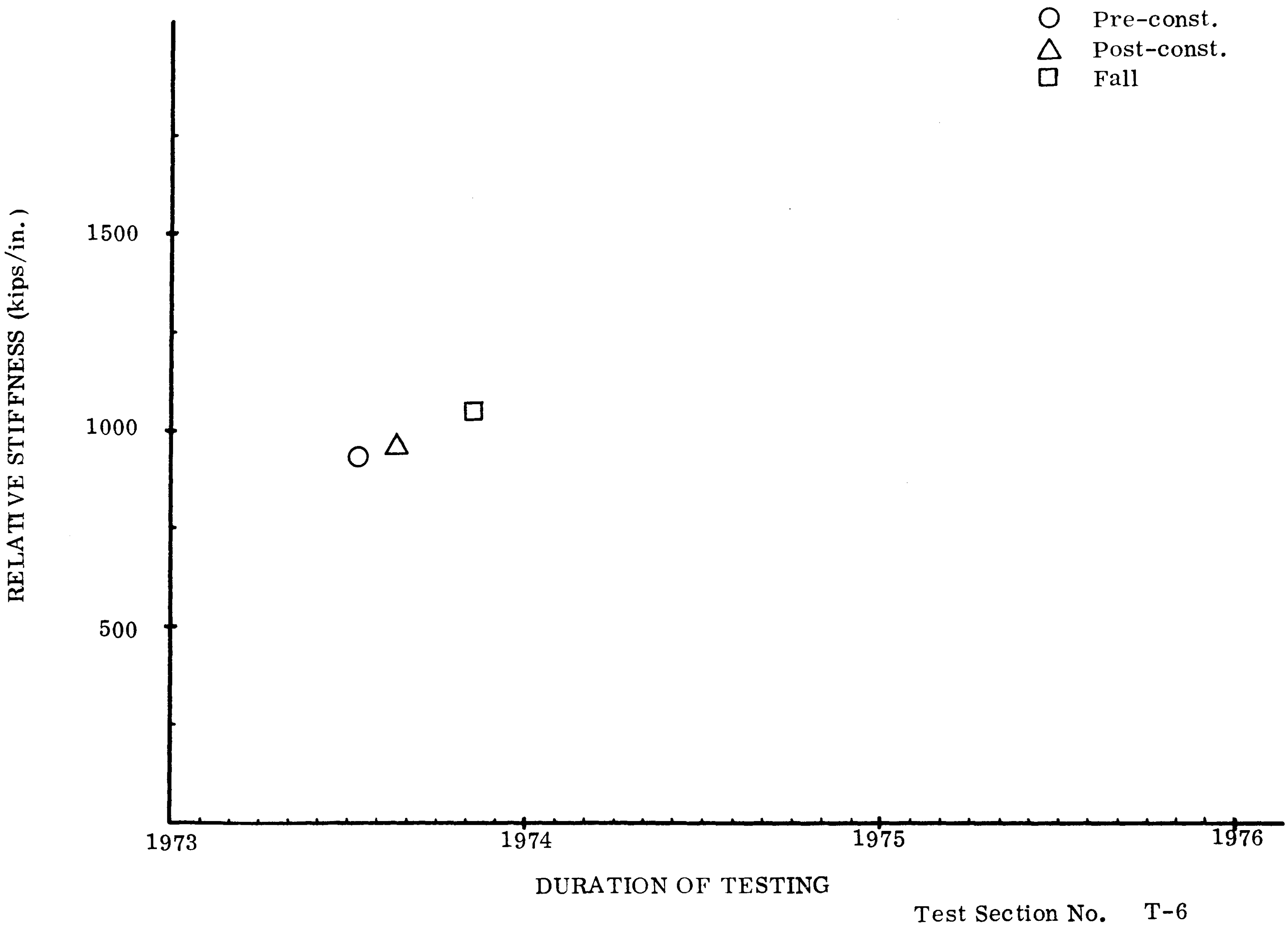
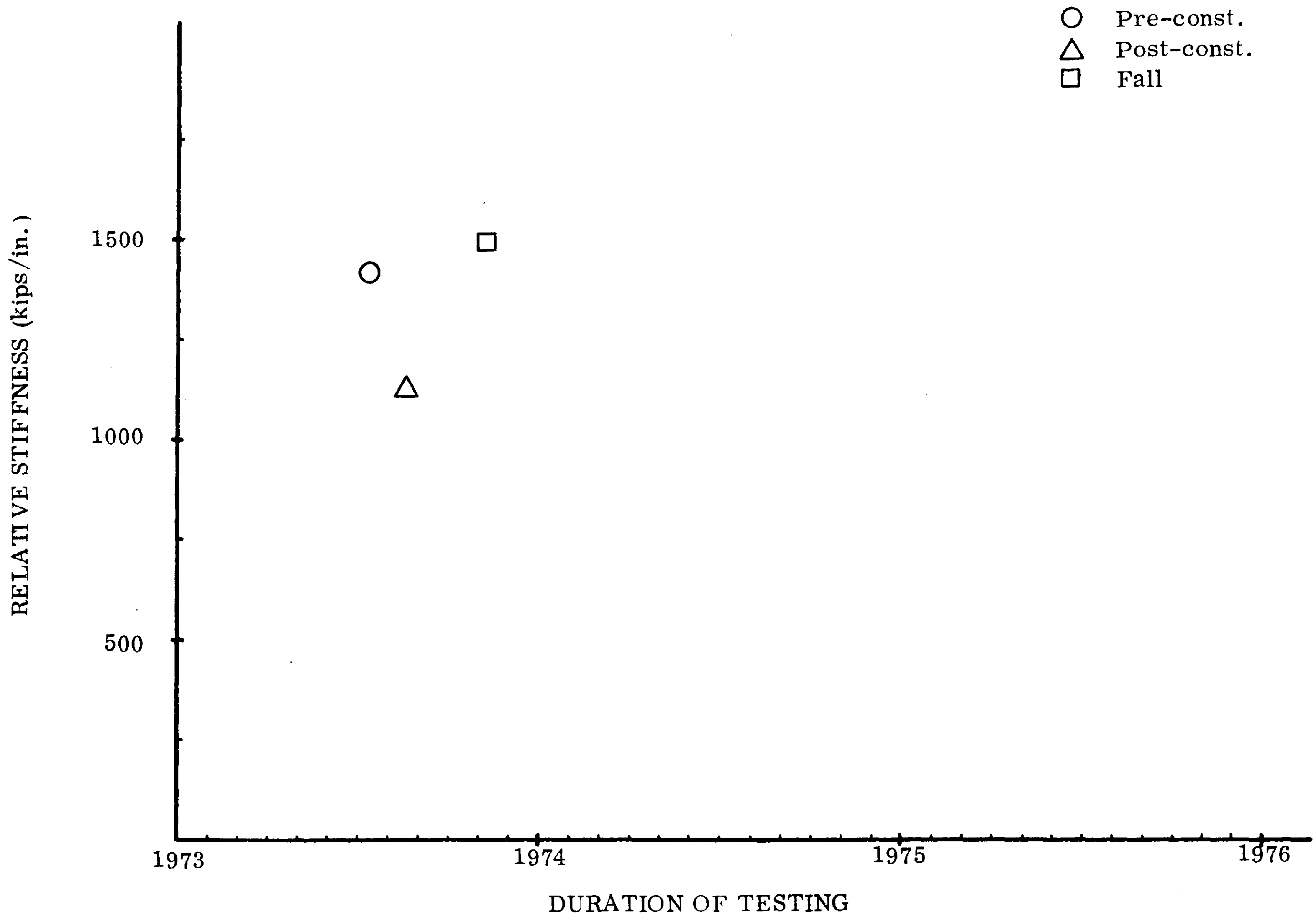


Figure 7. Benkelman beam-relative stiffness vs. time of year



Test Section No. T-8AB

Figure 8. Benkelman beam-relative stiffness vs. time of year

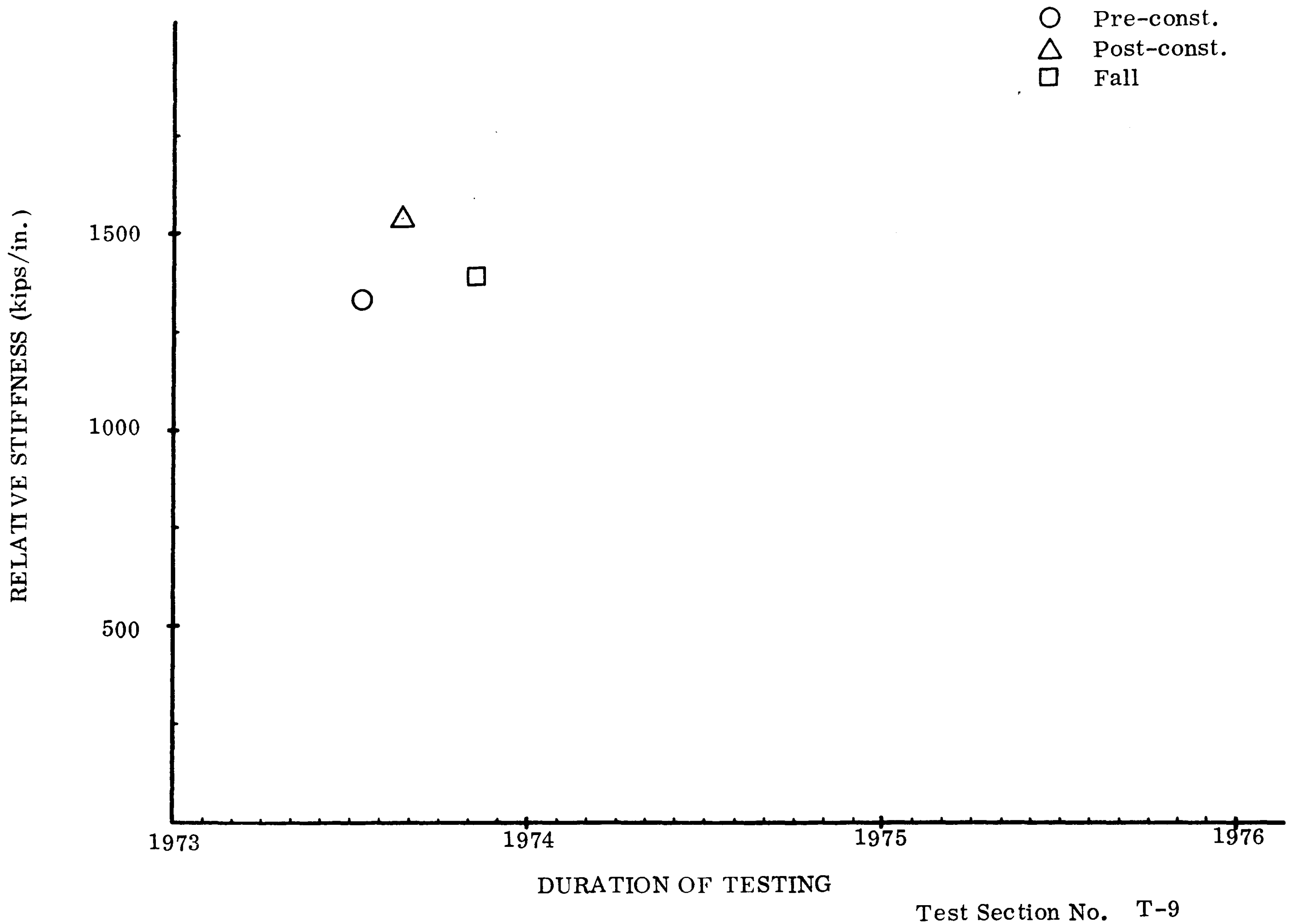
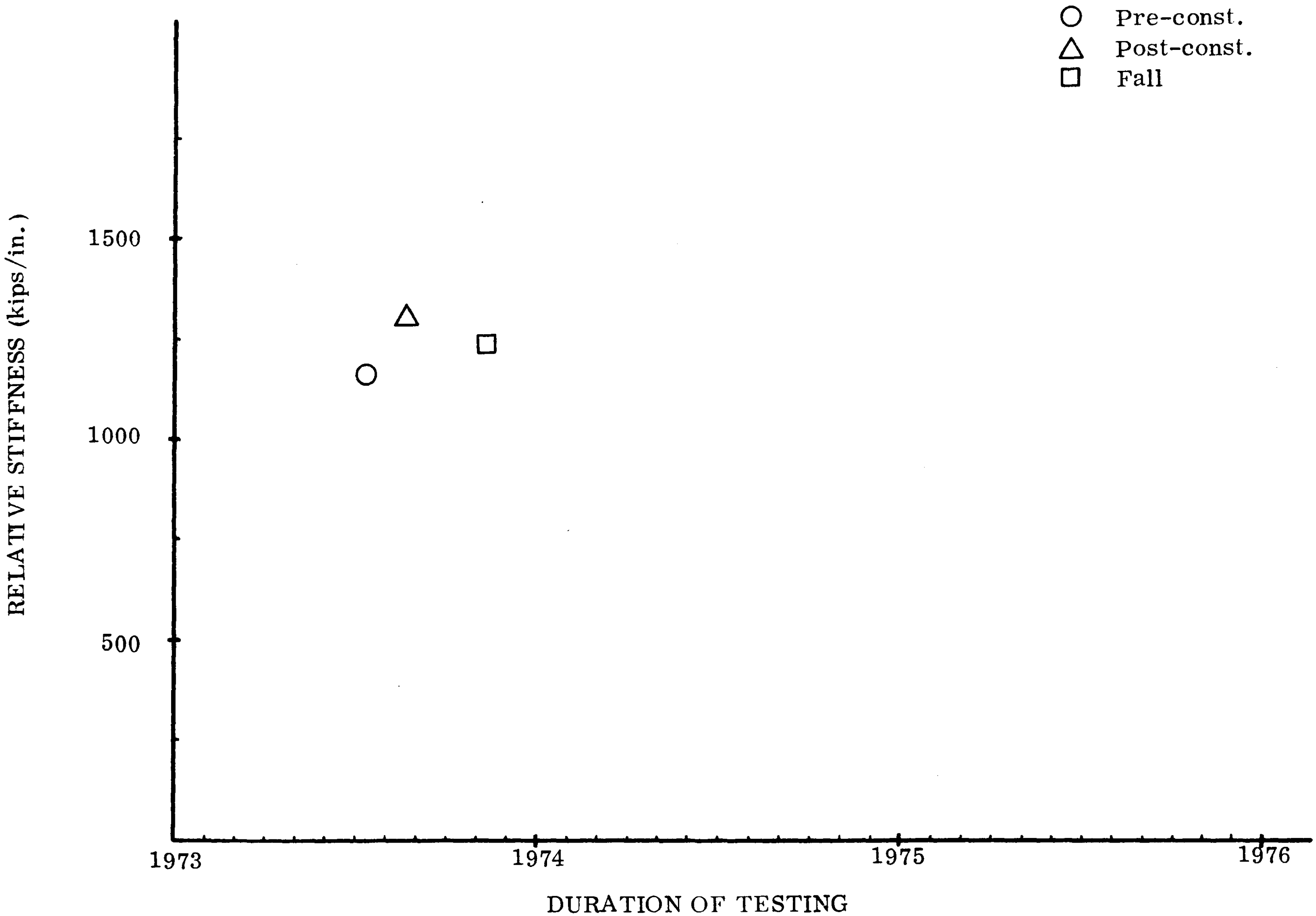
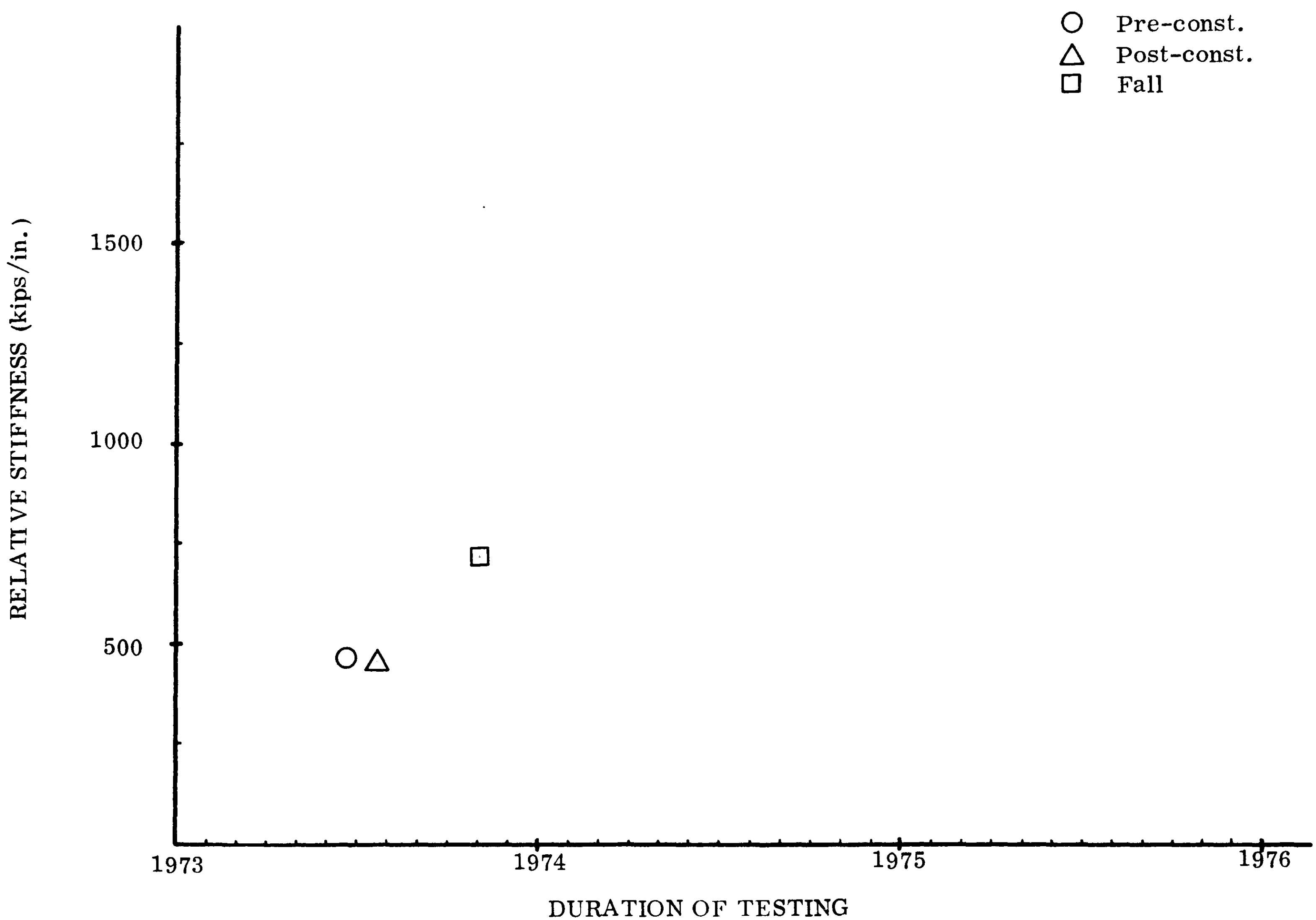


Figure 9. Benkelman beam-relative stiffness vs. time of year



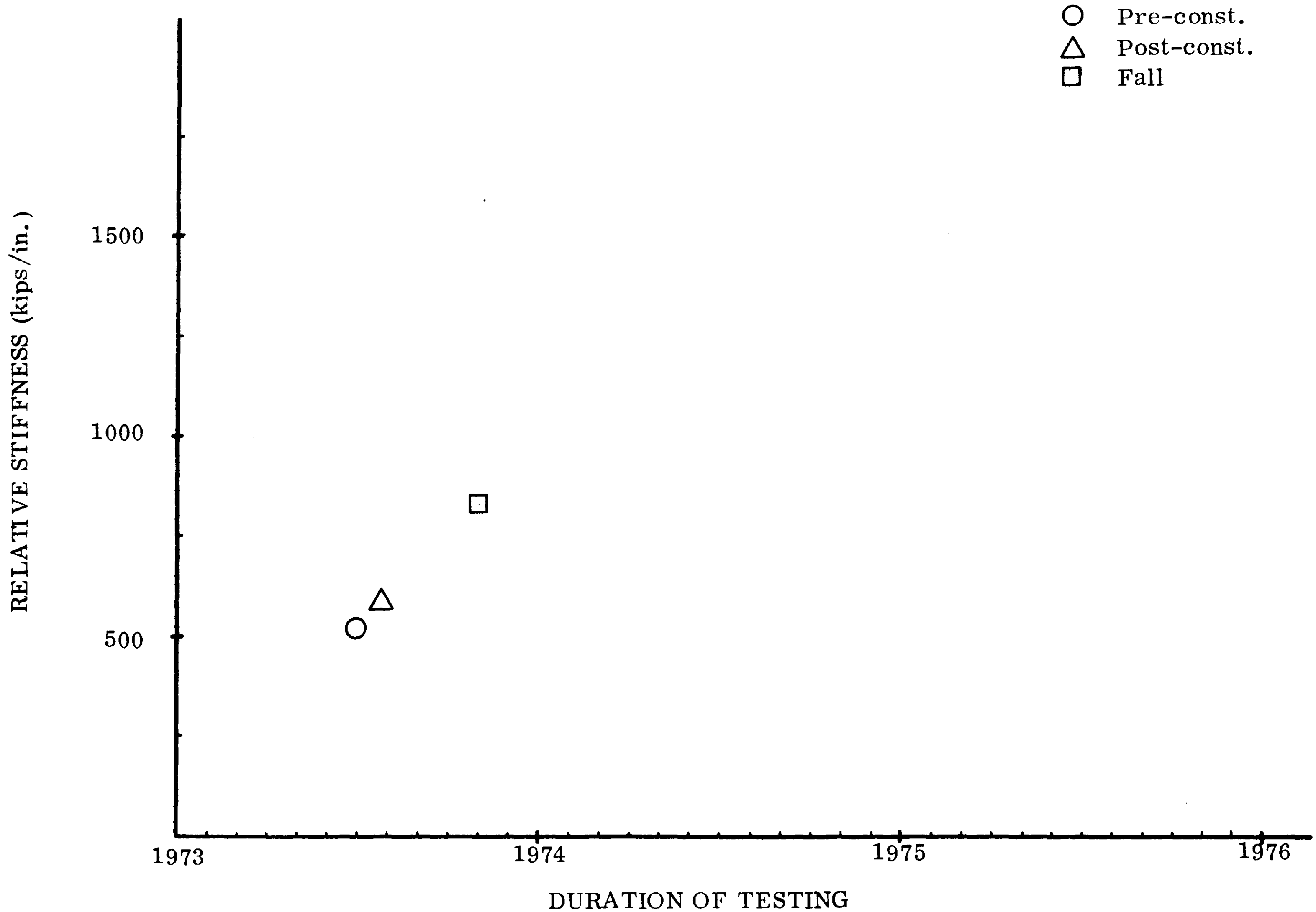
Test Section No. T-8BB

Figure 10. Benkelman beam-relative stiffness vs. time of year



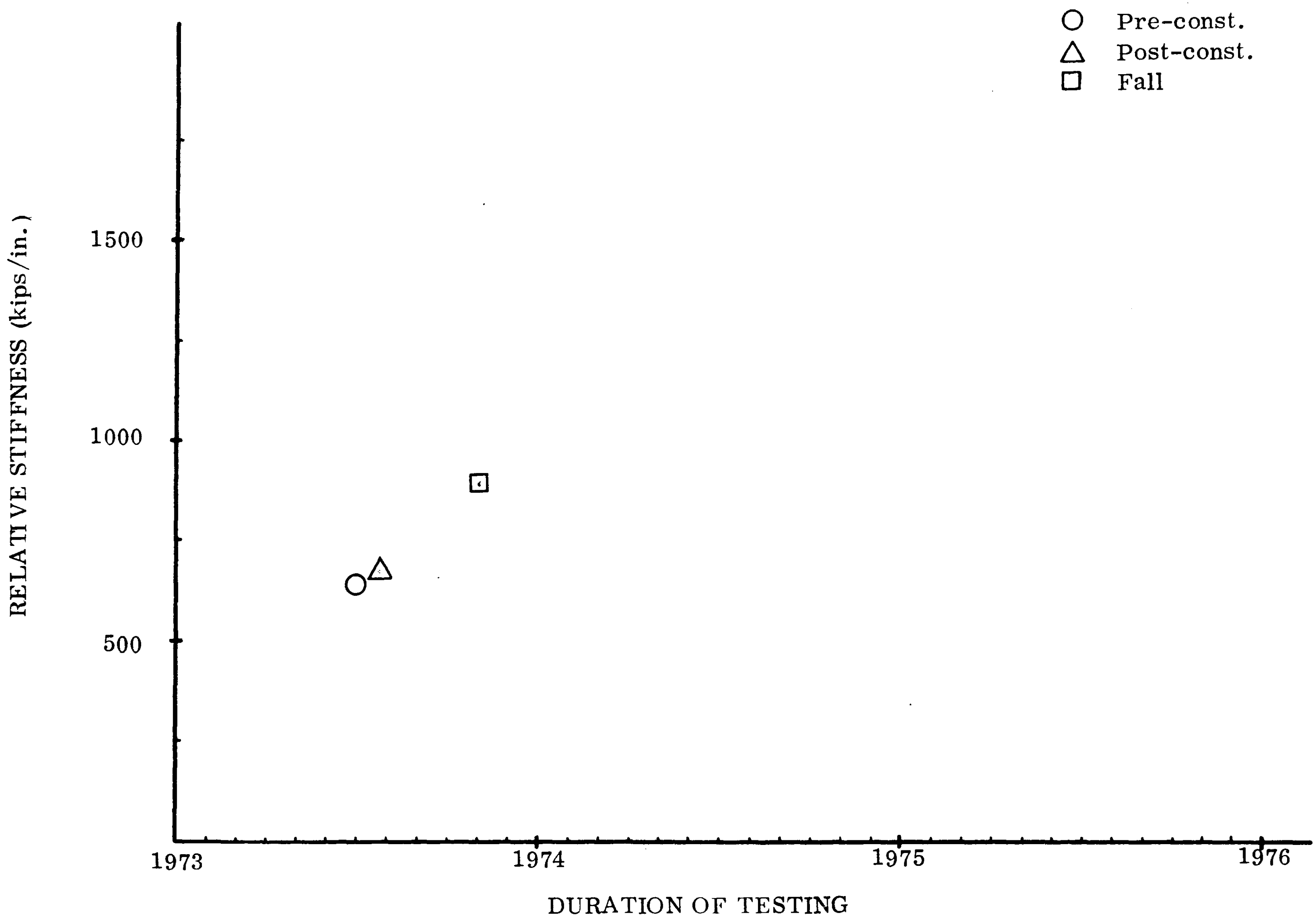
Test Section No. T-10

Figure 11. Benkelman beam-relative stiffness vs. time of year



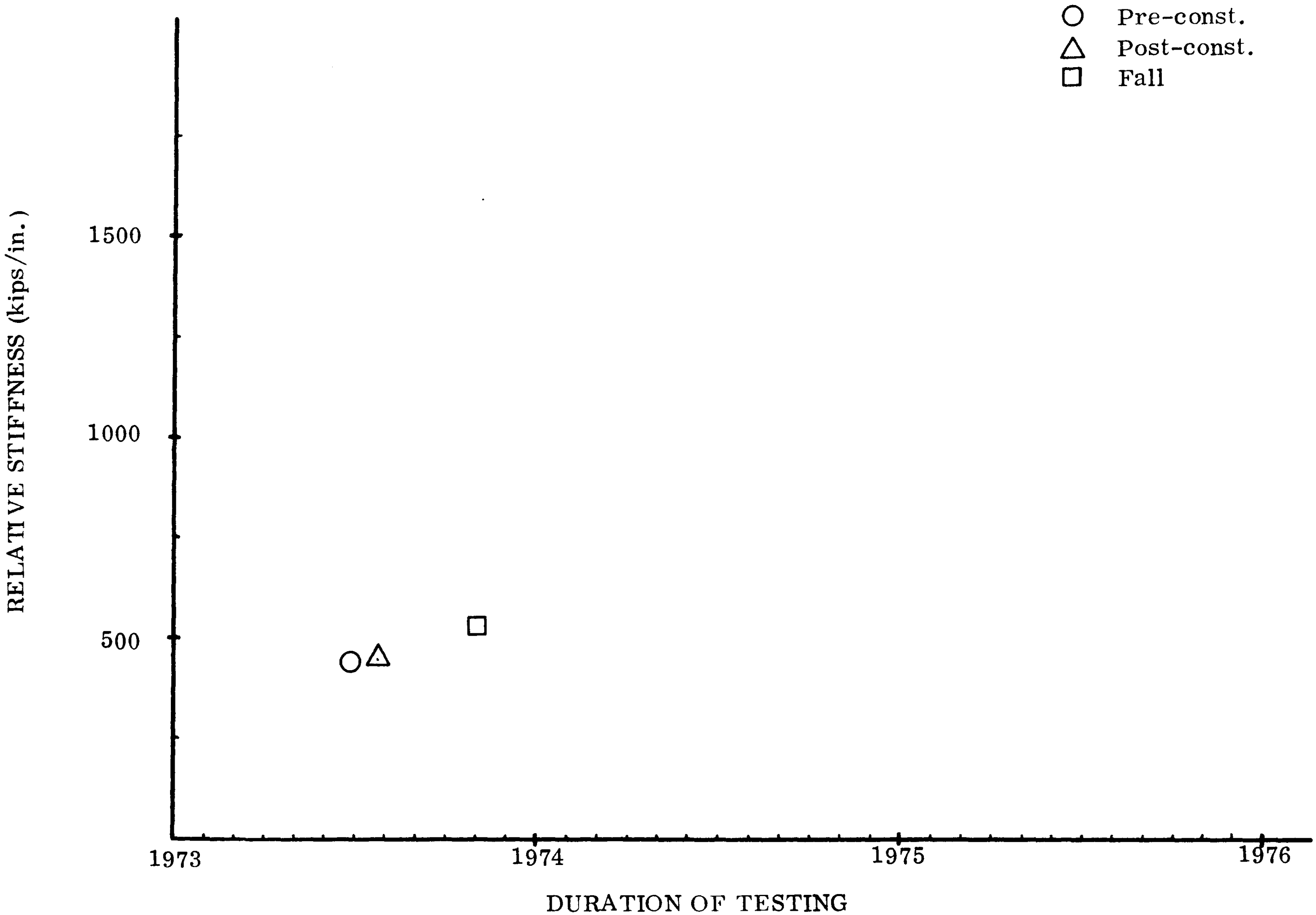
Test Section No. T-11

Figure 12. Benkelman beam-relative stiffness vs. time of year



Test Section No. T-12

Figure 13. Benkelman beam-relative stiffness vs. time of year



Test Section No. T-13

Figure 14. Benkelman beam-relative stiffness vs. time of year

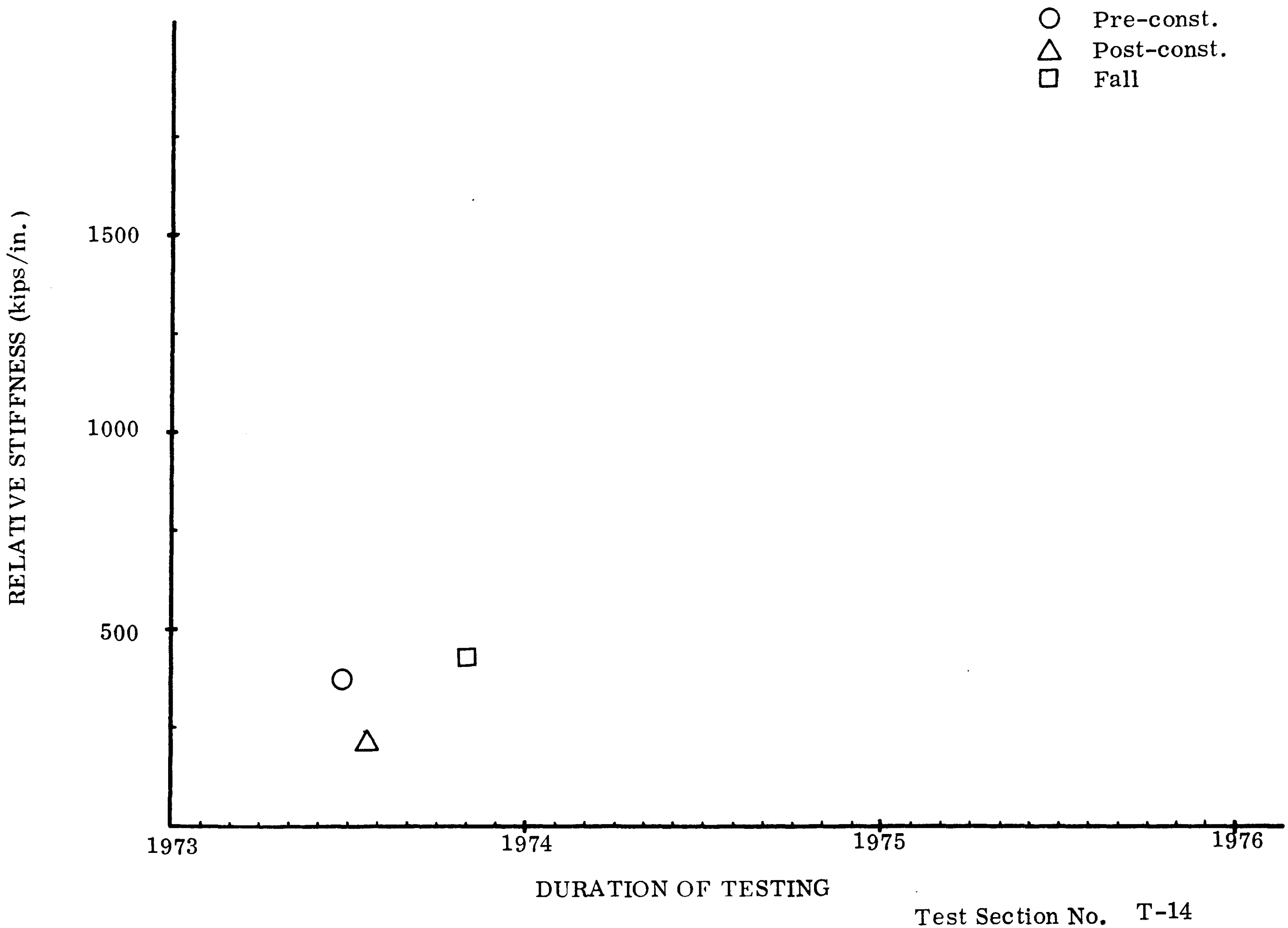
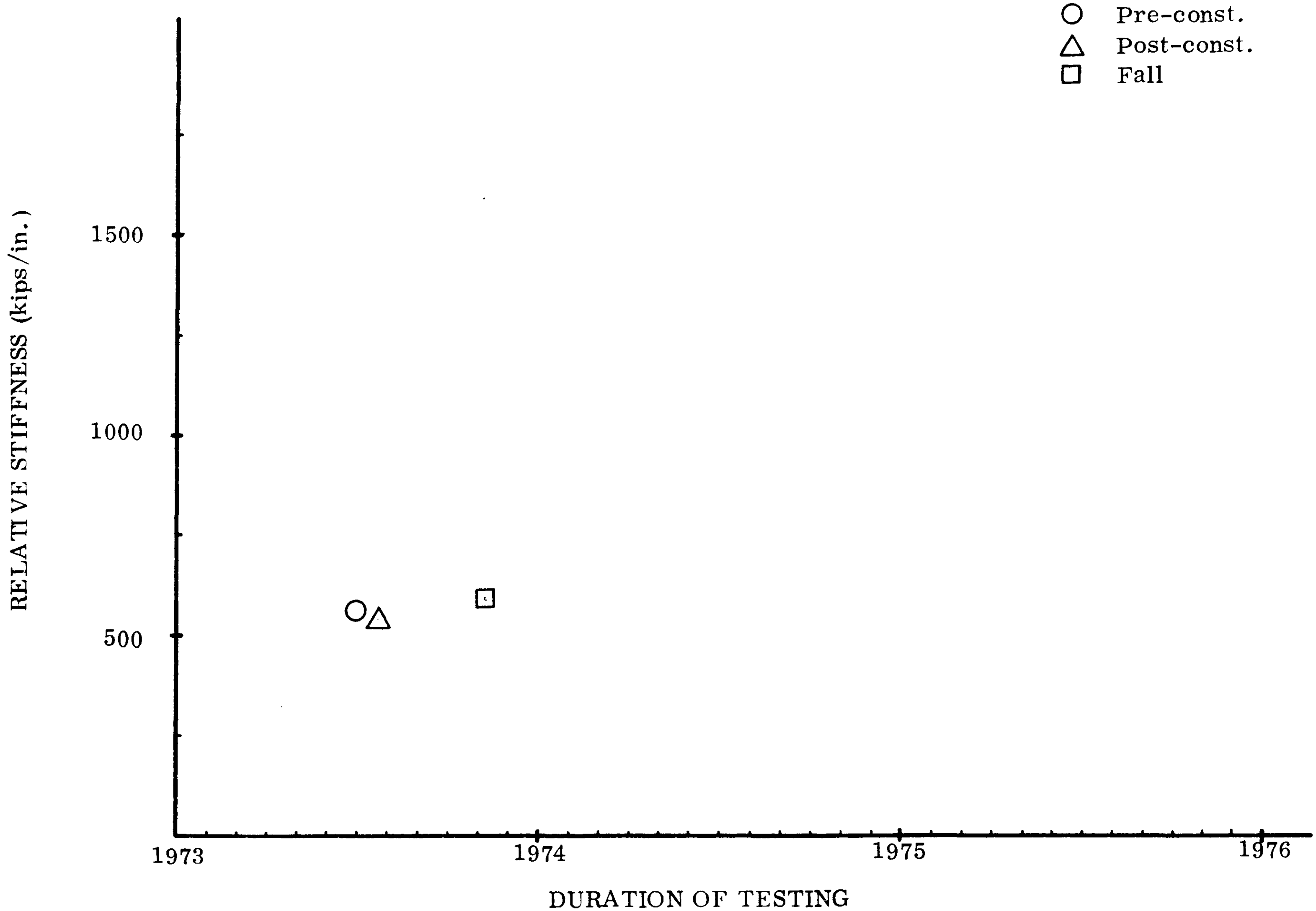
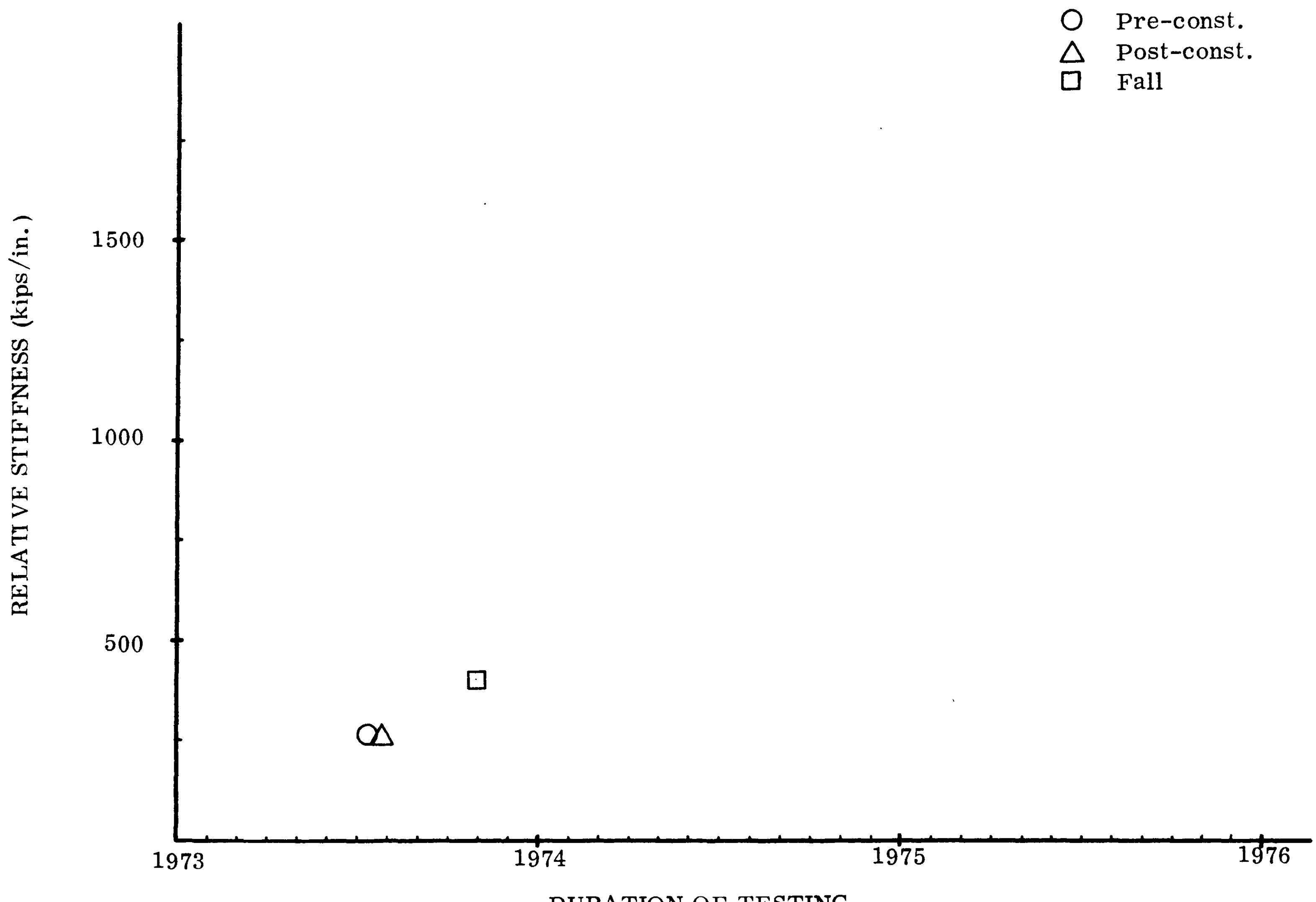


Figure 15. Benkelman beam-relative stiffness vs time of year.



Test Section No. T-16

Figure 16. Benkelman beam-relative stiffness vs. time of year



Test Section No. T-19

Figure 17. Benkelman beam-relative stiffness vs. time of year

Plotted values are the average of the combined OWT and IWT conditions*. Of the 14 test sections on which the Benkelman beam tests were conducted**, comparison of post- and pre-construction relative stiffness values indicate a definite lowering of three of the test sections, while seven of the sections remained relatively unaffected, and four sections showed immediate improvement due to construction and addition of the stabilization additives. The three sections showing a definite post-construction lowering of stiffness were T-5, T-8AB, and T-14; each of which were laid at somewhat above optimum moisture content.

Comparison of Fall 1973 with pre-construction relative stiffness values indicate none of the sections were still of lower stiffness, three sections were approximately equal to pre-construction stiffness, and 11 sections were definitely of greater stiffness. The three sections showing near equal pre-construction and Fall 1973 stiffness values were T-5, T-14 and T-16 though each of the sections showed a definite increase in stiffness in the approximate three months which had elapsed from the post-construction testing.

Maximum deflection tests under a slow moving load, as in the Benkelman beam test conducted in this study, are affected by the following variables;

(a) Base soil type and moisture content.

(b) Sub-grade soil type and moisture content.

(c) Stabilization additive utilized; i.e., cementing agent, binder, water proofer, etc.

(d) Base thickness.

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* Due to the rapid development of the project planning and the necessity for meeting construction deadlines, some pre-construction in-situ testing of the control sections was unavoidably eliminated. Pre-construction testing of all treated sections was, however, maintained. For purposes of this report, comparisons will be made only between the pre- and post-construction beam and SBV tests of the treated sections. Subsequent reports will also contain treated and control section comparisons of in-situ performance data.

**No Benkelman beam or spherical bearing value (SBV) tests are being conducted on Section T-17, surface application of Liquidow. Major in-situ tests on this section relate to dust measurements.

(e) Thickness of surfacing; though in this study it may be assumed that the seal coat has little or no effect on Benkelman beam deflections.*

(f) Time of year.

In general there is a direct relationship between Benkelman beam deflections and moisture contents of the base and sub-grade of a flexible highway. Moisture contents and deflections are high in the spring immediately following thawing, while both reach their lowest values in the fall when the water table is at its lowest elevation.

With the exception of sections T-8BB and T-9, all sections showed increased stiffness (lowered deflection) from post-construction to Fall 1973 tests. Sections T-8BB and T-9 showed a definite decrease in stiffness for this approximate three-month period. As of the period of testing presented in this report, the reasons for the reduction in stiffness of these two sections is not apparent. Spring and Fall 1974 deflections should assist in analyzing this discrepancy.

For the purpose of comparison of the test section deflections and relative stiffness values, limiting design deflections of flexible pavements normally range from 0.05 to 0.2 inch. Converted to relative stiffness, such limiting deflections thus range from 346.0 to 86.5 kips/inch, respectively, under the axle load of 17,300 lbs. Using Section T-2 as an example only, since its relative stiffness was amongst the lower values indicated in Table 5, the average of OWT and IWT were 171.4, 143.3 and 291.6 kips/inch for pre-construction, post-construction and Fall 1973 respectively. The preceeding relative stiffness values would thus indicate deflections of 0.10, 0.12, and 0.06 inch, respectively.

The above evidence should not be construed that various test sections were each a high performance pavement, since: (a) it has been shown that stiffness

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*This has been substantiated in unreported Benkelman beam tests conducted on unsealed and seal-coated six and eight inch thick soil-cement, soil-lime-flyash, and soil-lime bases, Webster County, Iowa, 1959 thru 1963.

of a 6-inch lime treated subbase, plus 7-inch soil-cement base, plus 3-inch asphaltic concrete surfaced pavement was in excess of 1000 kips/inch relative stiffness*; (b) deflection, or relative stiffness, is but one variable in total pavement performance; and (c) as has been previously indicated Fall values are normally high, whereas Spring values may be exceedingly low.

Spherical Bearing Value

The relative bearing capacities of each test section were analyzed insitu by the Spherical Bearing Value (SBV) test. This test has been shown to attain better reproducibility than either CBR or plate bearing tests**.

The SBV is the result of a stress-strain test in which hydraulic loads are applied to a 6 inch diameter spherically-shaped loading head and vertical deflections are recorded at various increments of load. Data obtained, is plotted with load as the ordinate and a function of deflection and diameter of the sphere as the abscissa. Slope of the plotted line is defined as the spherical bearing value (SBV), with units of psi. With the SBV data obtained on the test sections, a linear regression analysis was run on each data set in order to assure a "best fit" line, and subsequent SBV.

Three or more SBV tests were conducted on each test section; at least one each at each of the section's quarter points, alternating to each traffic lane approximate center line. All SBV values thus determined for each test section are presented in Table 6. As noted, the SBV tests were conducted at the same time as the Benkelman Beam tests.

Figures 18 thru 31 present a plot of the SBV of each test section for the

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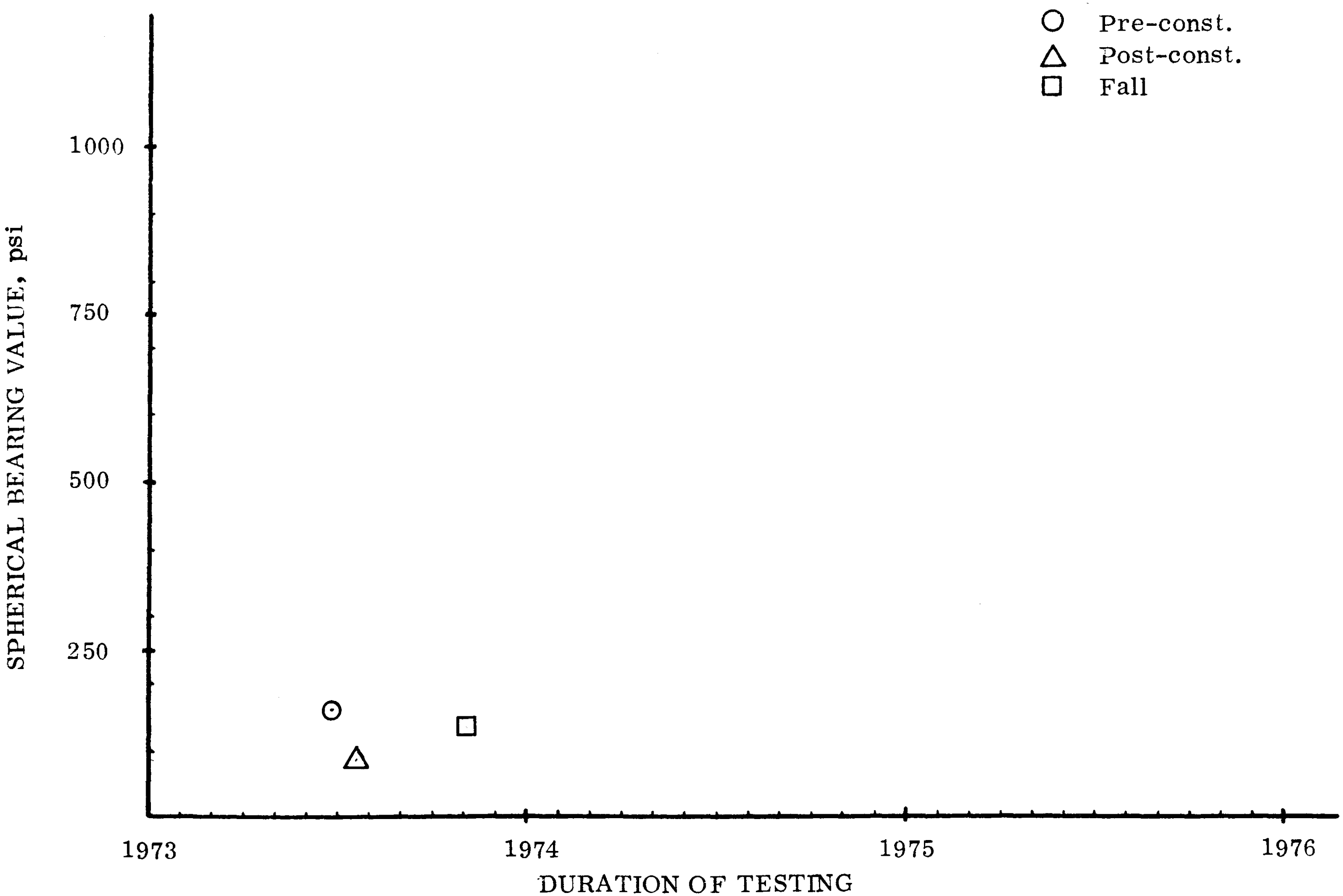
*Hoover, J. M., Huffman, R. T. and Davidson, D. T. Soil Stabilization Field Trials, Primary Highway 117, Jasper County, Iowa. Highway Research Board, Bulletin 357, pp. 41-68, 1962.

**Butt, G. S., Demirel, T., and Handy, R. L. Soil Bearing Tests Using a Spherical Penetration Device. Highway Research Record No. 243, pp. 62-74, 1968.

Table 6. Spherical Bearing Value Field Test Results

	SPHERICAL BEARING VALUE, psi		
	Pre-Const.	Post-Const.	Fall 1973
T-2	161.0	83.0	140.0
T-2A	217.5	126.0	140.0
T-5	535.0	194.3	230.0
T-6	320.0	417.3	266.2
T-8AB	228.0	470.0	330.0
T-9	220.0	600.0	340.0
T-8BB	190.0	140.0	230.0
T-10	406.7	86.3	260.0
T-11	206.7	244.0	313.3
T-12	193.3	307.0	483.3
T-13	160.0	146.7	146.7
T-14	393.3	63.3	136.7
T-16	246.7	183.3	130.0
T-19	270.0	103.3	130.0



Test Section No. T-2

Figure 18. Spherical bearing value vs. time of year

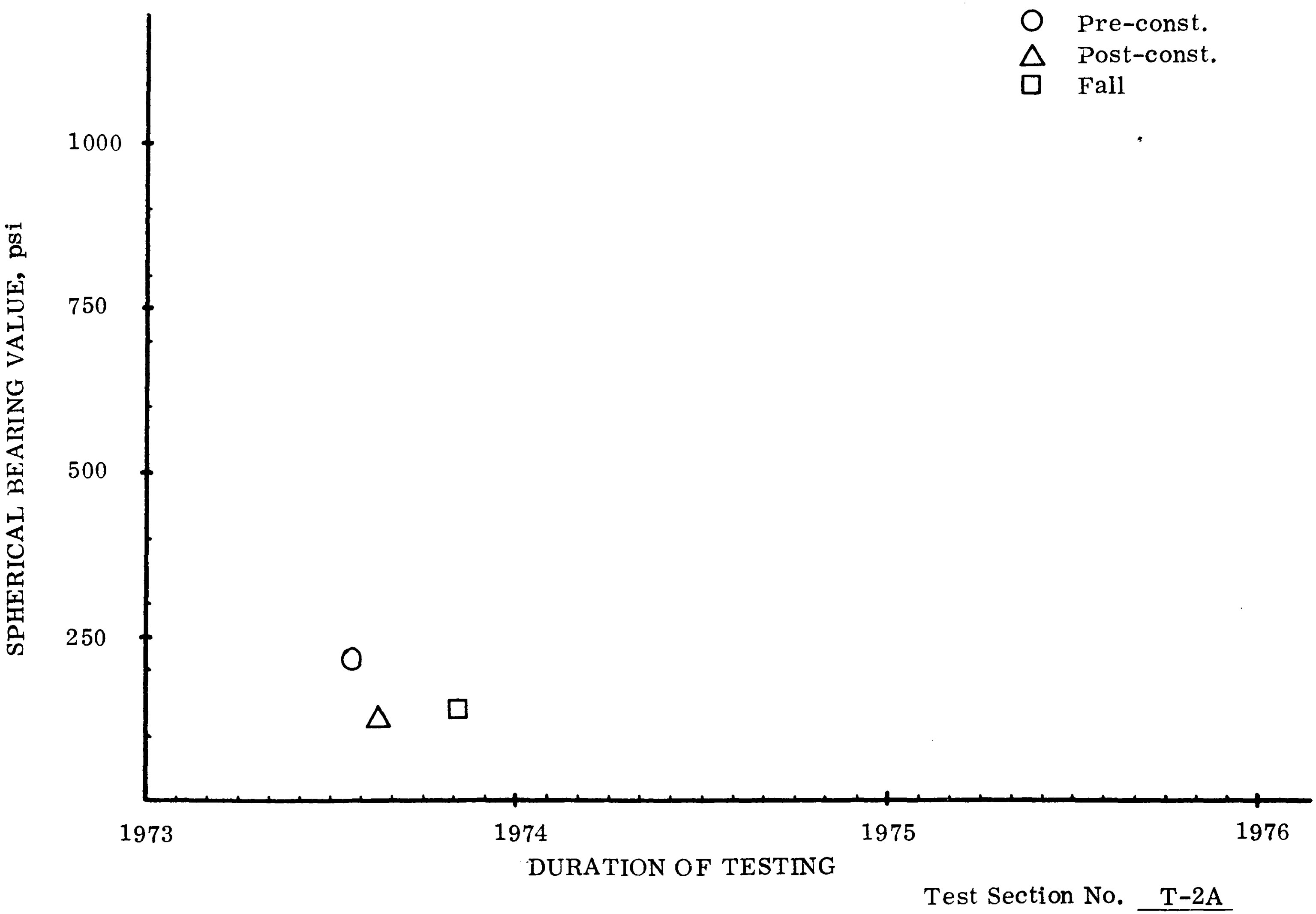


Figure 19. Spherical bearing value vs. time of year

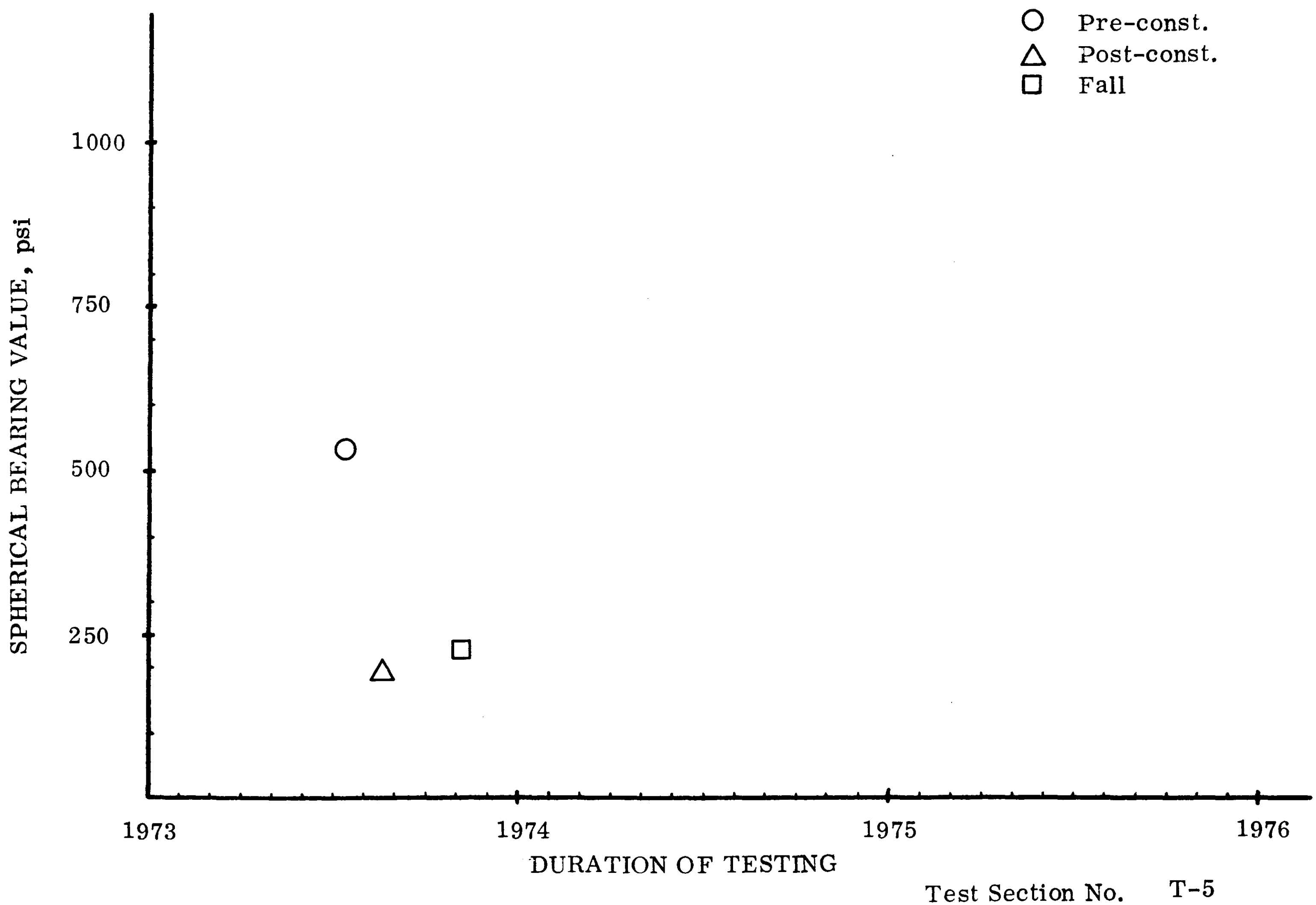


Figure 20. Spherical bearing value vs. time of year

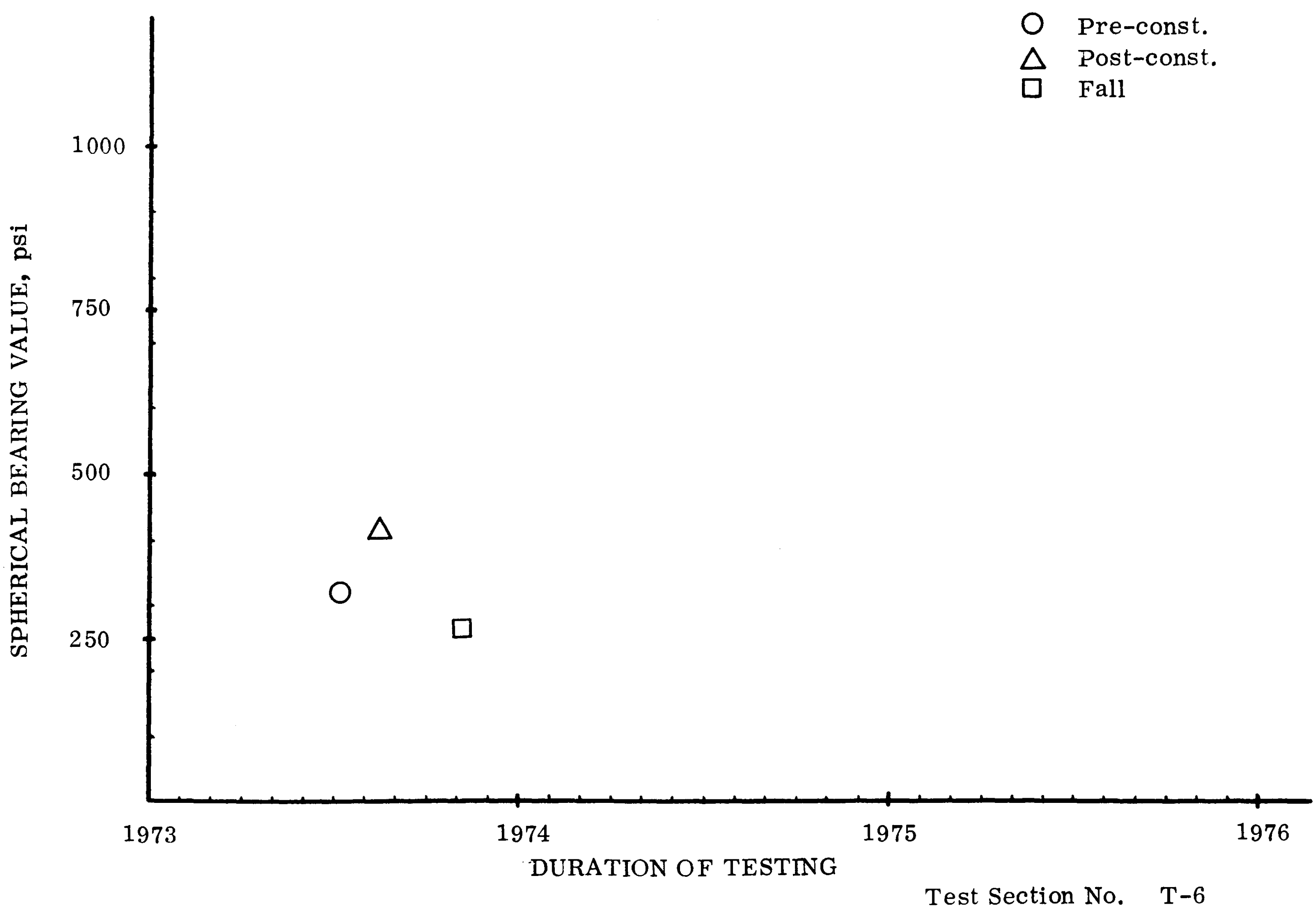


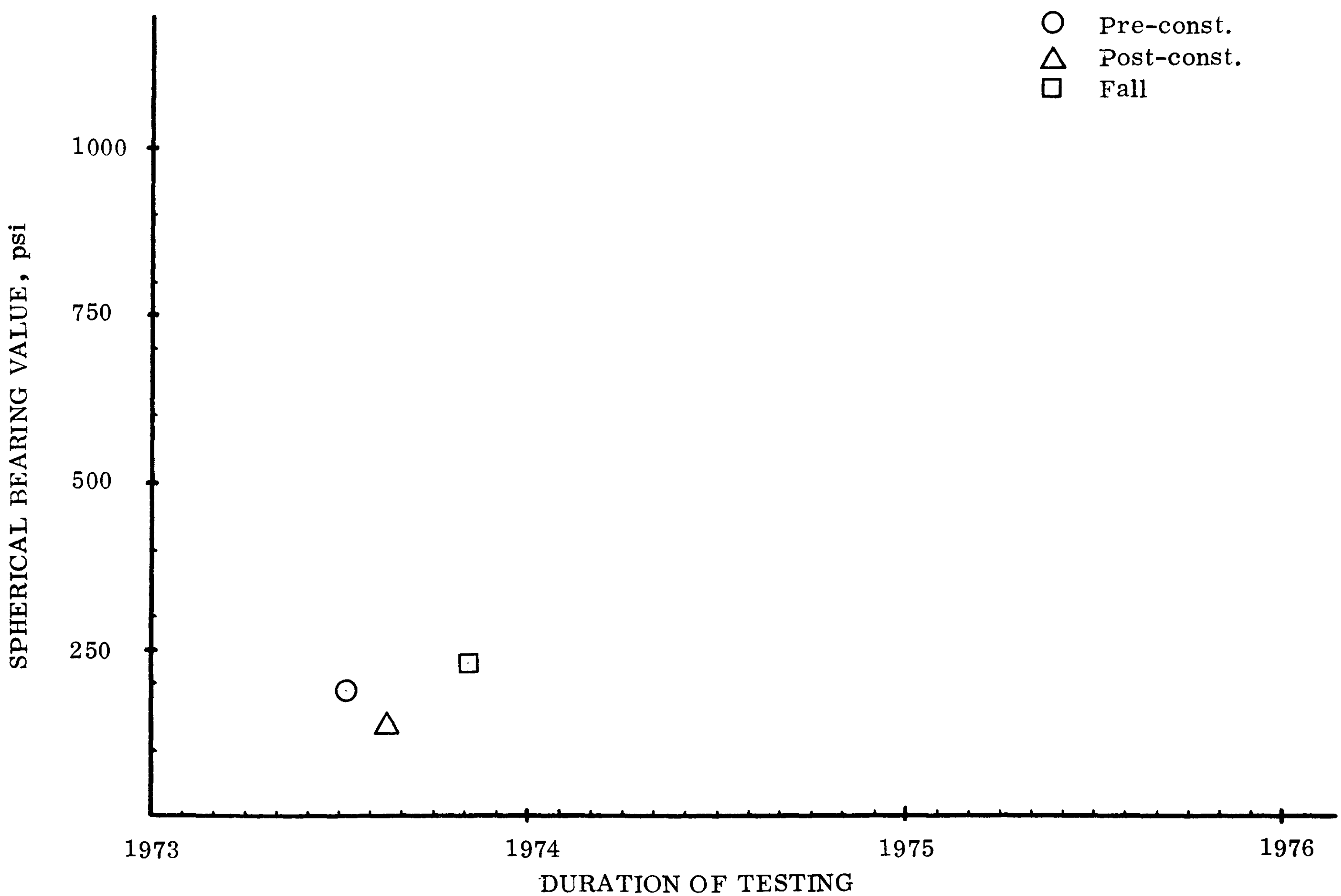
Figure 21. Spherical bearing value vs. time of year

Test Section No. T-8AB

Figure 22. Spherical bearing value vs. time of year

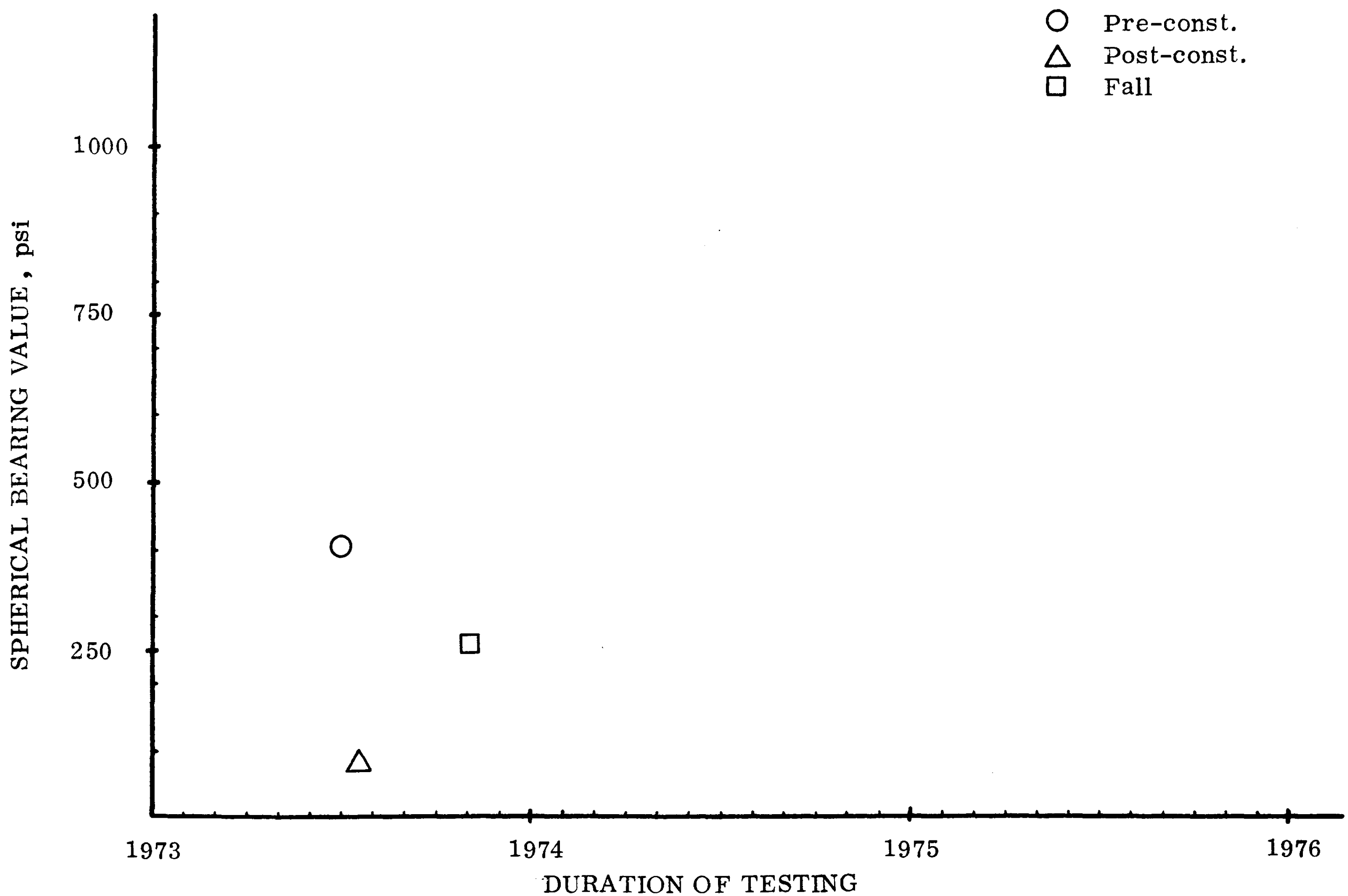
Test Section No. T-9

Figure 23. Spherical bearing value vs. time of year



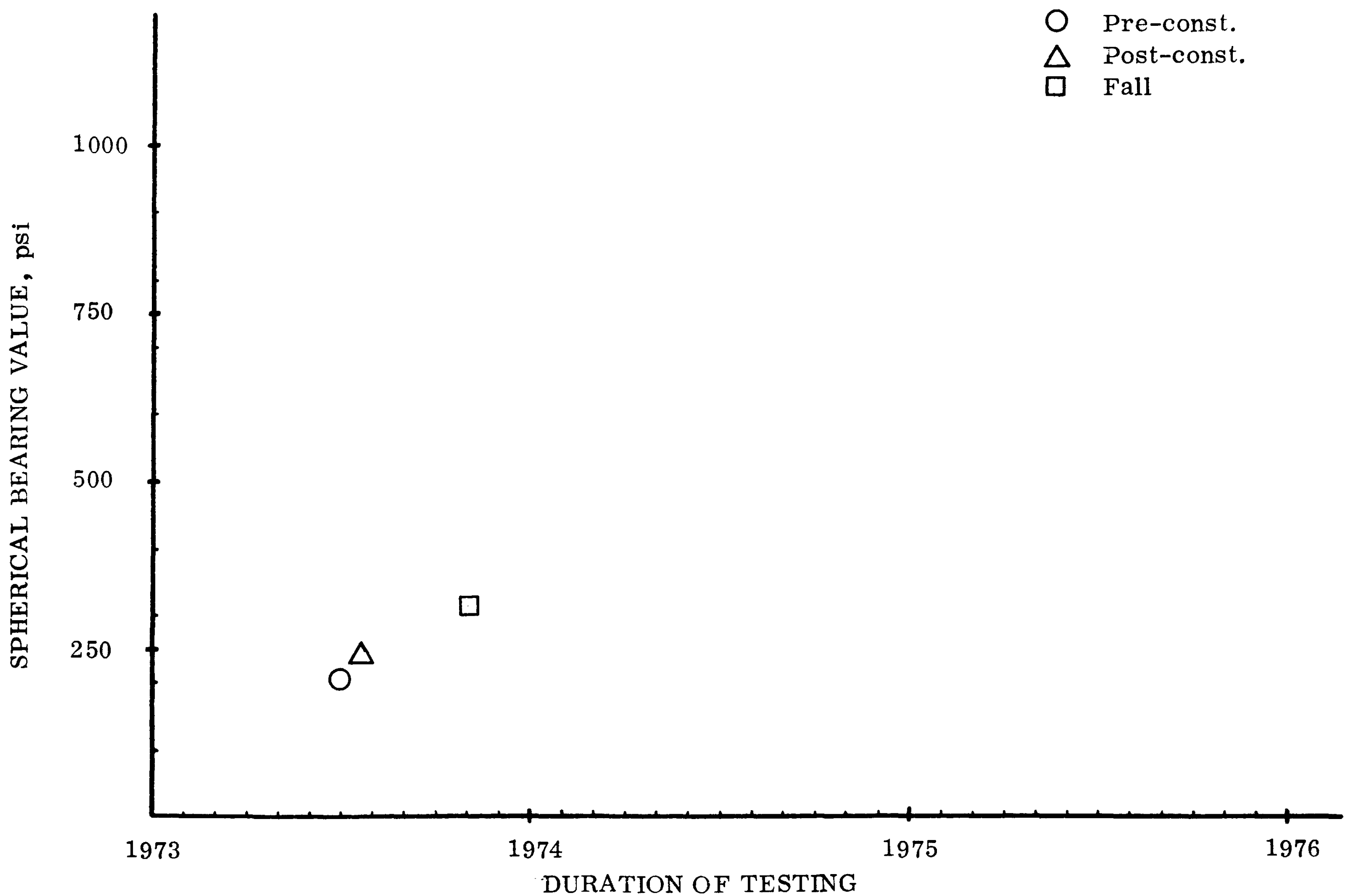
Test Section No. T-8BB

Figure 24. Spherical bearing value vs. time of year



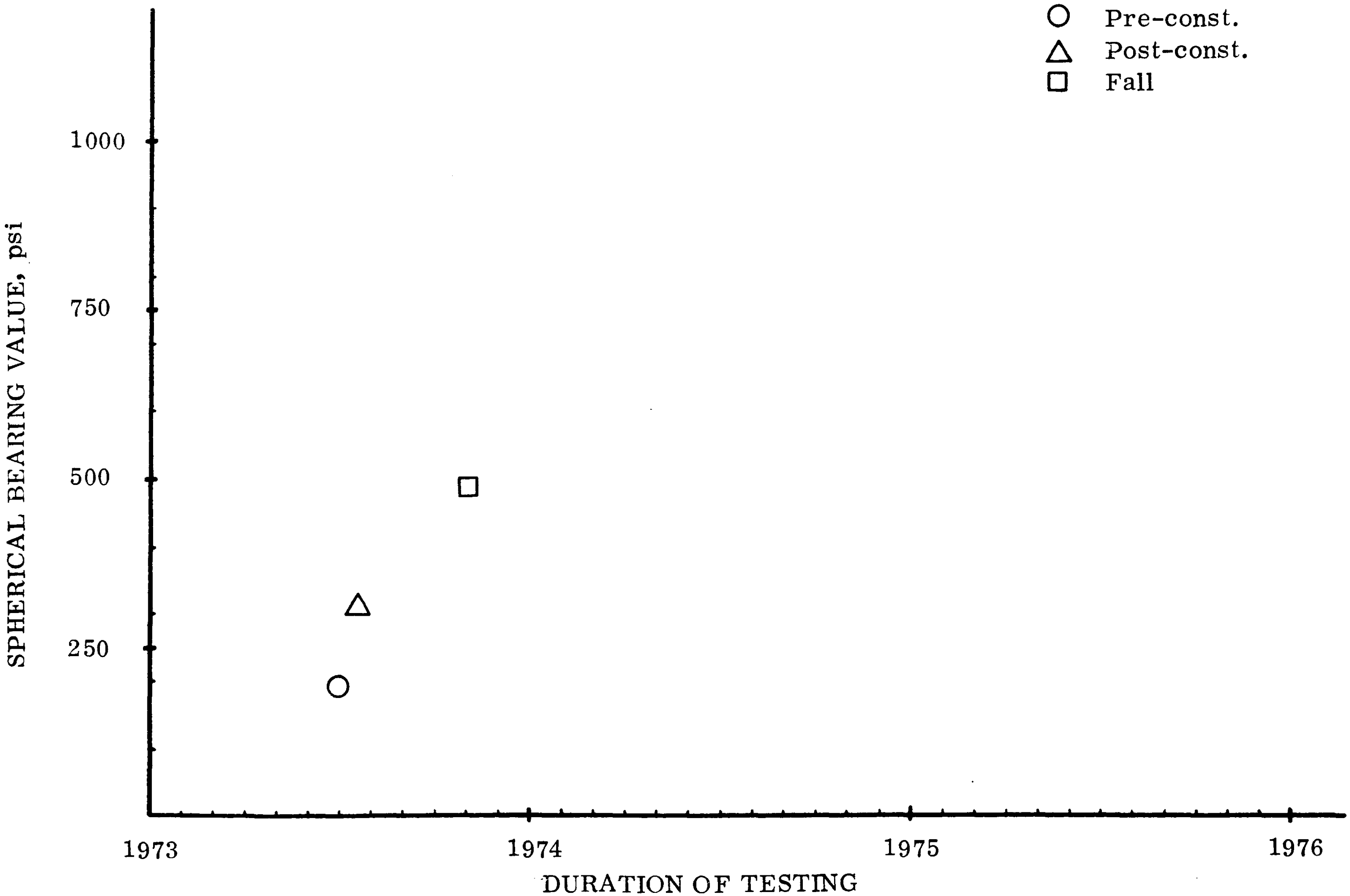
Test Section No. T-10

Figure 25. Spherical bearing value vs. time of year



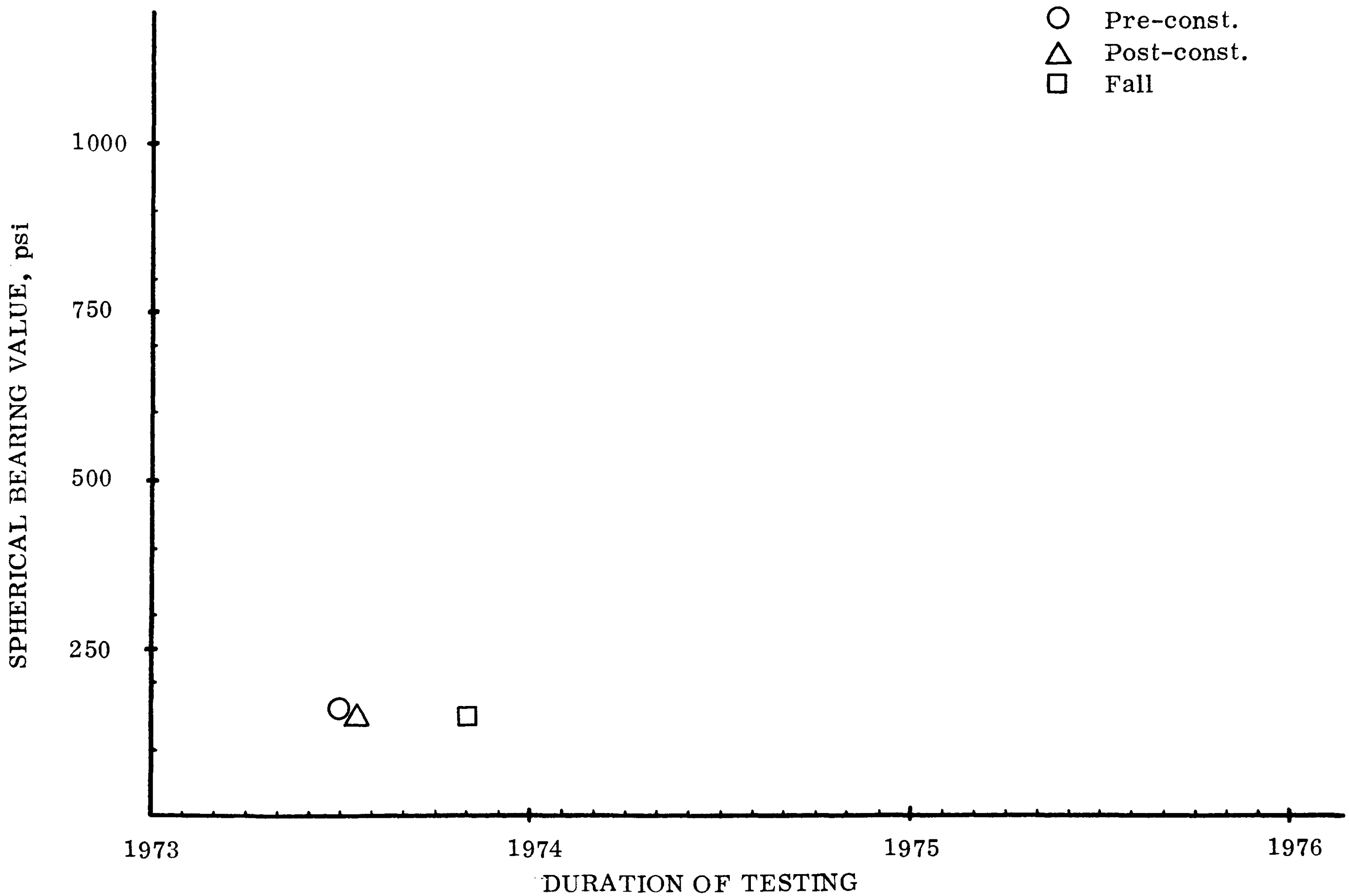
Test Section No. T-11

Figure 26. Spherical bearing value vs. time of year



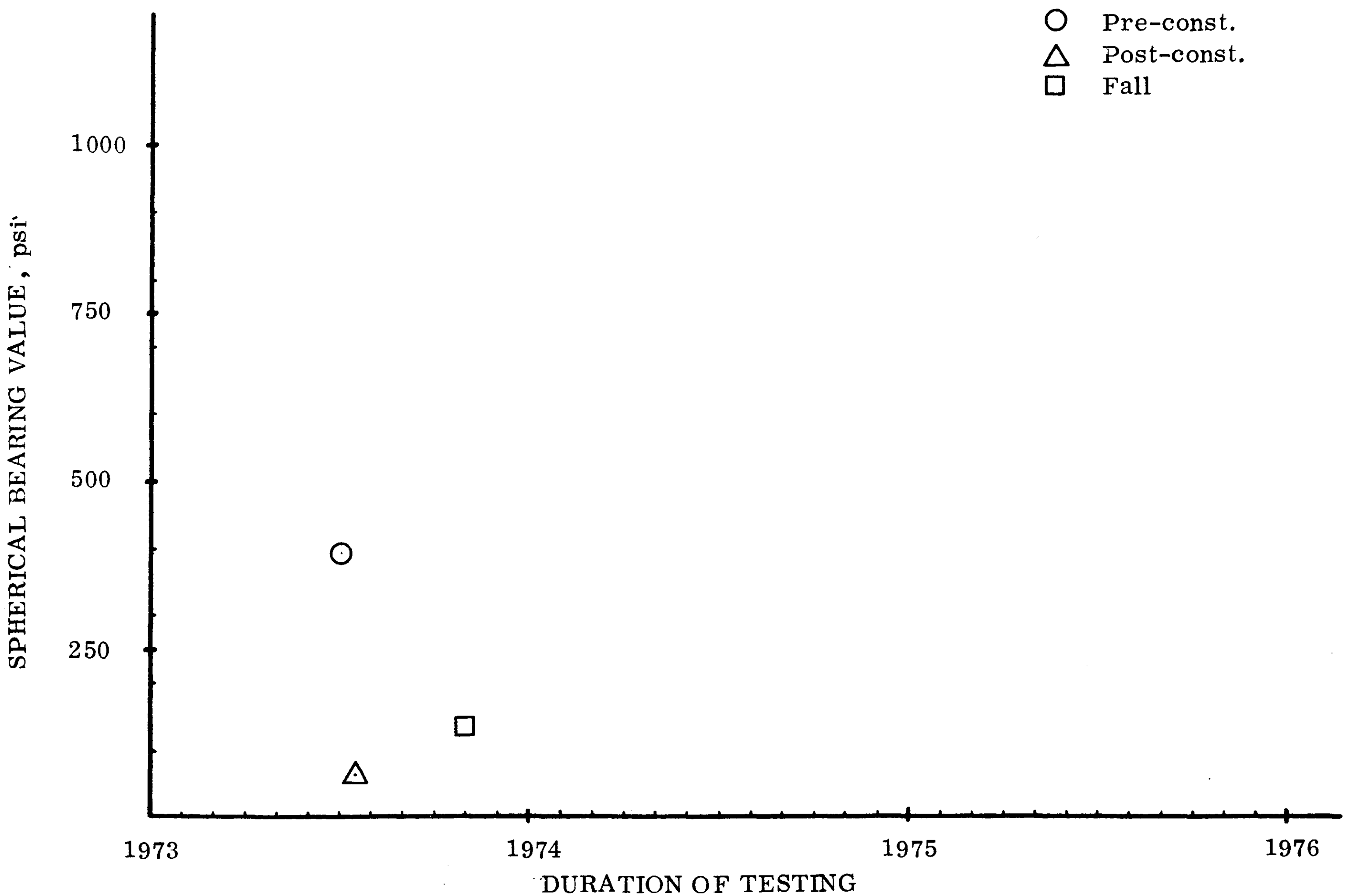
Test Section No. T-12

Figure 27. Spherical bearing value vs. time of year



Test Section No. T-13

Figure 28. Spherical bearing value vs. time of year



Test Section No. T-14

Figure 29. Spherical bearing value vs. time of year

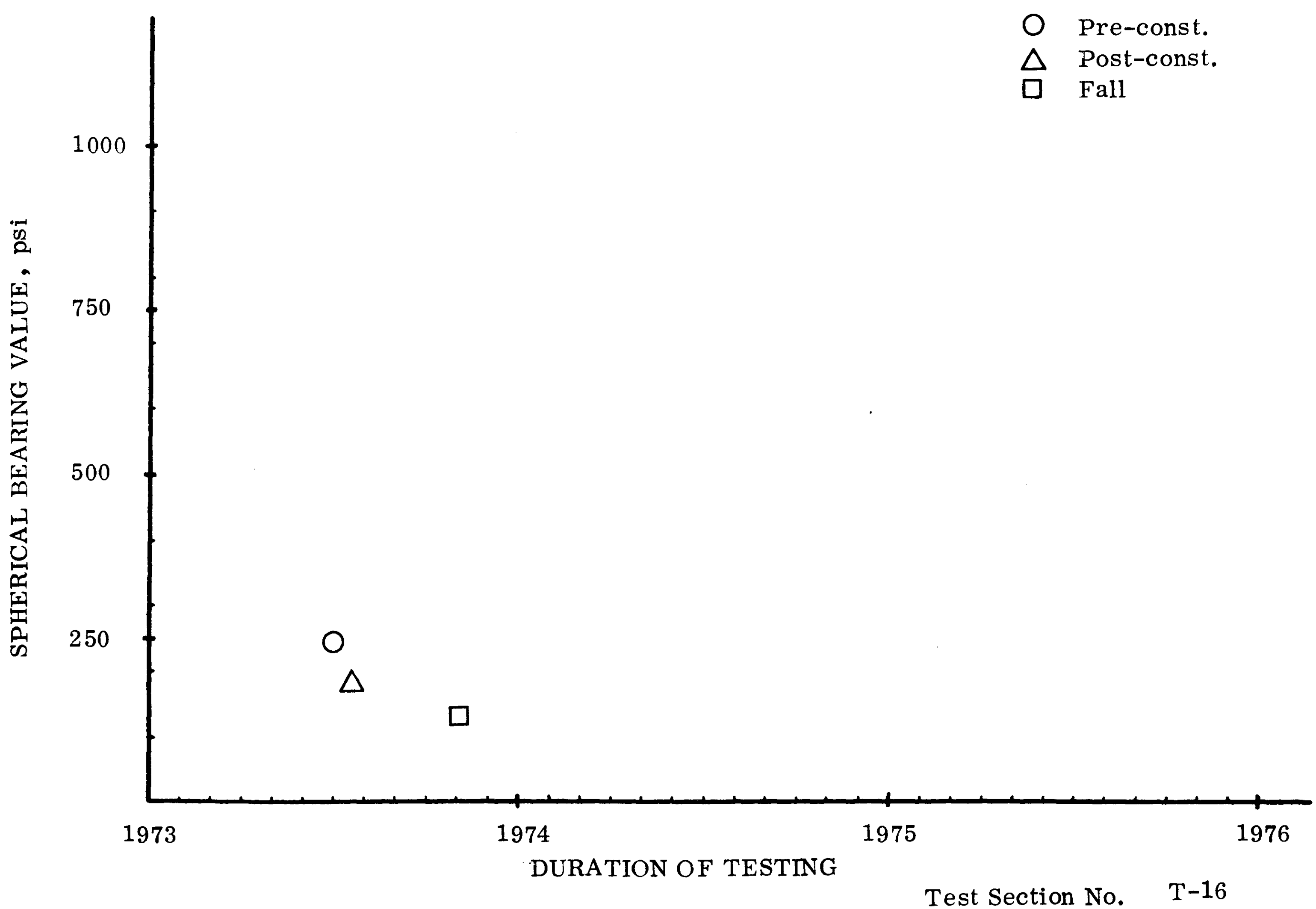
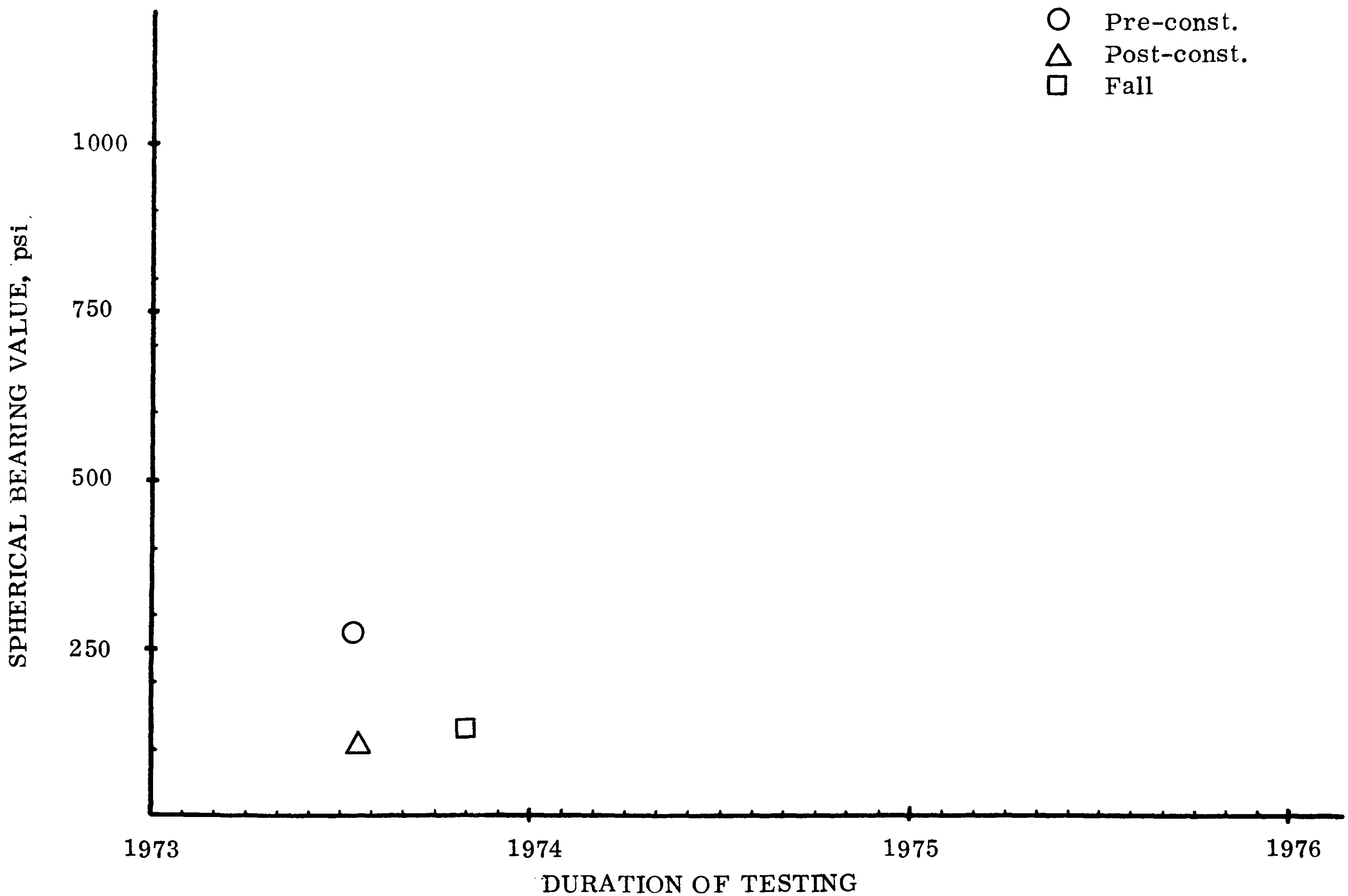


Figure 30. Spherical bearing value vs. time of year



Test Section No. T-19

Figure 31. Spherical bearing value vs. time of year

three series of tests conducted in 1973. Of the 14 test sections, comparison of post- and pre-construction values indicate a definite lowering of eight of the test sections, while one section remained relatively unaffected, and five sections showed immediate bearing improvement due to construction and addition of the stabilization additives. Those sections showing a definite post-construction reduction in bearing were T-2, T-2A, T-5, T-8BB, T-10, T-14, T-16, and T-19.

Comparison of Fall 1973 with pre-construction SBV's indicate an identical number of sections showing lower, relatively unaffected, and increased values. Those sections showing a reduction from pre-construction bearing were T-2, T-2A, T-5, T-6, T-10, T-14, T-16, and T-19.

Comparison of Fall 1973 with post-construction SBV's indicate a lowering of four of the test sections, while one remained relatively the same, and nine sections showed bearing improvement. Those sections showing reduced bearing were T-6, T-8AB, T-9, and T-16. During the approximate three months following construction, sections T-2, T-2A, T-5, T-8BB, T-10, T-11, T-12, T-14, and T-19 showed definite increases in bearing capacity.

The bearing of Section T-13 was relatively unaffected between post- and pre-construction testing and remained at the same bearing approximately three months after construction. Section T-16 showed a continued decrease in bearing from pre- to post-construction to Fall 1973 values.

As of the period of testing presented in this report, the reasons for the variation in SBV bearing values is not yet apparent. Spring and Fall 1974 values should assist in analyzing the obvious variances.

Spherical Bearing Values have been compared to unsoaked CBR and unconfined compressive strengths of soil by Butt, Demirel, and Handy. For example, an SBV of 100 is approximately equal to an unsoaked CBR of 5, while an SBV of 200 is about a CBR of 18 and an SBV of 300 is equivalent to CBR 32. Unconfined compressive strengths for the same three SBV's noted above are respectively 27, 54, and 80 psi.

Continuing Studies

In addition to the various field tests discussed in the preceeding sections, numerous laboratory investigations were begun in June 1973, and will continue until completion sometime during the summer of 1974*. Each of the laboratory investigations is designed to contribute correlative information relative to analysis of the in-situ field tests and observations. In addition, correlation of laboratory and field tests should contribute to mix and thickness designs of surface/base roadways stabilized with the various products used in the test sections. Each lab test is conducted on soil obtained from the test sections prior to construction and utilized in: (a) an untreated condition, without additive other than water; (b) treated at the same percentage of additive concentration used in the field construction and as recommended by the participant; (c) treated at less percentage of additive than used during construction; and (d) treated at greater percentage of additive than used during construction.

Laboratory tests completed or being conducted on treated and untreated section soils include:

1. Moisture-density under standard compaction.
2. Particle size distribution.
3. Atterberg Limits.
4. Unconfined compressive strength following 24-hour air cure.
5. Erosibility following 24-hour air cure.
6. Freeze-thaw following 24-hour air cure.
7. Trafficability following 24-hour air cure.

In these particular lab studies it should be noted that we have used only the 24-hour air cure period. Previous studies have indicated that 0 day air cure tests yield very little data relating to chemical reactions taking place. In prior studies that have utilized the 0-, 1-, 3-, and 7-day cures it was shown that

*Limited laboratory tests thereafter will probably continue until 1976 coupled with core drilling annually, and annual Spring and Fall beam, SBV, and other tests.

a 24-hour cure period was sufficient to demonstrate a qualitative and somewhat quantitative evaluation of the strength properties. It was decided that a 24-hour cure would best expedite the laboratory phase by cutting down on soil expenditure as well as saving time.

In addition to the above tests, traffic data is to be accumulated for all test section roads in order to correlate lab and field tests, field observations, and performance to traffic.

It should thus be obvious that the data and observations presented in this report constitute but a very limited portion of the data which will ultimately be accumulated in the research project. Therefore, the authors have drawn no conclusions in this report, since conclusions of any type would be extremely premature and without benefit of total objective results of all tests. Though the data presented in this report may indicate that some test sections show superior or inferior results when compared with other sections, the authors suggest that each participant in this project refrain from such conjecture.

Progress Report No. 2 will be issued on or about March 1975 and will cover all activities for the period January 1 to December 31, 1974.