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A THERMOVISCOELASTIC CHARACTERIZATION OF AN ASPHALTIC CONCRETE

in cooperation with the
Department of Transportation
Federal Highway Administration
Bureau of Public Roads

RESEARCH REPORT 127-2
STUDY 2-8-69-127
BINDER REQUIREMENTS FOR FLEXIBLE PAVEMENTS

A THERMOVISCOELASTIC CHARACTERIZATION OF AN ASPHALTIC CONCRETE

by

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Research Report 127-2
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PREFACE

A preliminary draft of this report was reviewed by members of the staff of the Texas Transportation Institute, the Texas Highway Department, and the U. S. Bureau of Public Roads. This final draft has been revised in cognizance of the suggestions made in the reviews by these three organizations.

The opinions, findings and conclusions expressed in this publication are those of the author and not necessarily those of the Bureau of Public Roads.

ABSTRACT

Material properties in the uniaxial tension and compression modes were obtained for two compacted asphaltic concrete mixtures for various temperatures and rates of loading. The first mixture was preheated and compacted at 300°F whereas the second mixture was processed at 450°F; thus, the embrittlement due to overheating with the compactive effort constant was quantified. The results show that stress should not be used and strain could be used for assessing material performance.

OBJECTIVES OF STUDY

This reported work is a part of a continuing research study at the Texas Transportation Institute, sponsored by the Texas Highway Department and the U. S. Bureau of Public Roads. The objectives of the overall study are to:

1. Determine the performance requirements of the material needed to serve as the cohesive-adhesive waterproof binder for a first-class, long-life flexible pavement surface course,

2. Develop control tests for use in a specification for a material that will meet the performance requirements in objective one, and,

3. Concurrently with objective number two, search for a cohesive-adhesive material that will meet the requirements of objective number one.

SCOPE OF THE REPORTED RESEARCH

The scope of this report covers a thermoviscoelastic characterization of two types of asphaltic concrete tested in uniaxial compression and tension at various strain rates and temperatures. The first mixture was preheated, mixed, and compacted at 300°F. The second mixture was preheated, mixed, and compacted at 450°F. The data reduction and results for 55 tests are given in this report.

The test conditions were constant strain rate loading over 5 decades of rate and a temperature range of -50 to 150°F. Fundamental material properties (ultimate strain, ultimate stress, ultimate secant modulus, and initial tangent modulus) were obtained in two of the fundamental loading modes, namely, uniaxial tension and uniaxial compression.

The objectives of the reported work were to:

1. Determine uniaxial properties of asphaltic concrete since this type of data is so limited.

2. Compare uniaxial tension and uniaxial compression properties of flexible concrete.

3. Investigate suitable basis of failure that could be used in formulating performance requirements to reduce pavement cracking.

4. Investigate trends in variations of properties as a function of rate of loading and temperature.

5. Summarize the experimental work for use in design procedures.

6. Determine porperties of the concrete for two mixing temperatures with all other direct controlled variables and conditions the same.

SUMMARY OF THE REPORTED RESEARCH

1. Constitutive relations were obtained for the thermoviscoelastic properties in uniaxial compression of a particular asphalt concrete that could be used for preliminary design purposes.

2. All of the pertinent results concerning the thermoviscoelastic characterization were portrayed on a single graph for each set of tests to show the variation of the four fundamental properties as functions of time and temperature.

3. For an asphaltic concrete mixed at 300°F and tested in uniaxial compression, the ultimate stress varied between about 14 and 1400 psi and the average ultimate strain varied between 1 and 3 percent, depending upon strain rate and test temperature (Figure 5).

4. For specimens from the regular mix tested in uniaxial tension, the ultimate stress varied between about 3 and 400 psi and the ultimate strain varied between 0.2 and 1.3 percent (Figure 5).

5. For an asphaltic concrete mixed at 450°F and tested in uniaxial compression, the ultimate stress varied between about 32 and 3500 psi and the ultimate strain varied between 1.1 and 1.7 percent (Figure 6).

6. For specimens from the asphaltic concrete mixed at 450°F and tested in uniaxial tension, the ultimate stress varied between about 6 and 500 psi and the ultimate strain varied between 0.1 and 1.1 percent (Figure 6).

7. The Smith failure envelope shows how increasing the mix and compaction temperature increases the maximum ultimate stress and decreases the maximum ultimate strain.

8. The average percent air voids in the specimens mixed and compacted at 300°F and 450°F were 8.8 and 3.0, respectively.

9. Since the ultimate stress varied 2 orders of magnitude while the ultimate strain varied by a factor of about 3 for the extremes of test temperature and strain rates, ultimate normal strain could be used, but ultimate normal stress should not be used as a failure criteria.

IMPLEMENTATION STATEMENT

The practical application of the research findings is that quality control on flexible pavement could be established on the basis of ultimate uniaxial strain allowable, and a pavement system could be designed theoretically rather than empirically. However, as the results are interim, no implementation is recommended at this time, pending results of research in progress.

INTRODUCTION

Background on the Project

It may be useful to succinctly summarize some of the discussions at the various project conferences and previous documentations. In general, a theoretical approach is being used so far as possible, with experimental work performed as necessary. The theoretical approach, in contradistinction to an empirical approach, enables guidelines to be determined for applicability, assessment of limitations, and offers the possibility of a priori design.

Several points must be understood before performance requirements can be stated and quantified. First, the problem or problems must be defined. It appears that rutting and shoving problems have been almost eliminated due to the use of stability tests (Marshall or Hveem). However, in satisfying stability requirements highway design has gravitated toward leaner mixtures which are more susceptible to cracking. It is therefore assumed in this project that the primary performance requirement presently needed is something that can be used to decrease the occurrence and/or propensity of a surface course to crack due to the various causes.

To be of any practical utility, a stated performance requirement must be given in terms of readily measurable fundamental quantities, i.e. mechanical, physical, and/or chemical properties. Whatever is measured should be invariant, so far as possible, with source and other factors. To determine these conditions implies a knowledge of mechanical properties, i.e. failure criteria such as some form of stress, strain, or energy. But the failure criteria of a material is usually, if not always, stress state dependent. This means that a uniaxial test on a binder alone may be necessary but is insufficient.

Since the mechanical characterization of any conceivable binders for flexible concrete will be both temperature and secularly* dependent, this implies the need for thermoviscoelastic characterizations to determine the critical condition or conditions. But, it is not presently possible to theoretically predict the mechanical properties of an asphaltic concrete in a quantitative manner, knowing only the mechanical properties of the constituents (because of the multitude of interactions between the binder and the aggregate). It is therefore necessary that experimental material characterizations be performed for purposes of design. If a test is devised for purposes of evaluating the binder only, then the binder must be tested in a suitable triaxial stress state because of the stress state dependency of the failure mechanism. Further, without knowing something about the correlation of the mechanical properties for a composite with real aggregate and with an artificial and/or standardized aggregate to produce hydrostatic stress and to reduce the variables, the latter could only be useful for relative ranking.

Having a problem definition, knowing what to measure and the measurement that should be obtained, the next step is to determine if present binders for first class pavements are generally inadequate or if they give a structural integrity that is marginal. The answer to this dictates the type of measurement of performance requirement that should be considered, how the control test

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*Varying with time while exercising a memory of previous conditions.

should be performed, and gives some insight for the selection of more suitable binders, e.g. modified asphalt, epoxy, polyethylene, etc.

If the thermoviscoelastic moduli are known, theoretical procedures can then be appropriately used for calculation of the required measure of performance, which can then be compared with the corresponding material allowable. Therefore, both theoretical analysis and experimental characterizations must be considered in this project if the stated objectives are to be accomplished.

In view of the preceding assessments, the necessary approach includes experimental characterizations, knowledge of failure mechanisms, and theoretical analysis.

Background on the Reported Research

The primary purpose of this work was to perform limited thermoviscoelastic characterizations of asphaltic concrete in order that problems might be better anticipated in further work. In the process of doing this, two compaction procedures were used for possible utility, i.e., to quantify "embrittlement" in terms of fundamental properties, to a first approximation.

It may be useful to define some terms being used here. After mixing a binder with aggregate, the resulting material is called a "mixture." After the mixture is compacted (either in the laboratory or in the field), the resulting material is called "flexible concrete." If the concrete is placed in the field over supporting structure, the material is called "pavement" or "surface course." If the pavement is placed over a supporting structure in the laboratory, the resulting configuration is called a "highway model analog."

This work was necessitated by an apparent lack of comprehensive fundamental thermoviscoelastic properties for flexible pavement over the range of temperatures used, as shown by a literature review. An excellent summary of existing material data on asphaltic concrete has been given by Finn (1).* The most comprehensive study on linear viscoelasticity found in the literature was the work by Alexander (2). Fundamental properties needed for purposes of design or for setting performance requirements (3) are the time-temperature dependent initial tangent modulus, ultimate secant modulus, and the Smith failure envelope (4). Monismith, Secor, and Secor (5) included the Smith failure envelope for the materials tested in their study on temperature distress in flexible pavement.

In general, for conservative design to preclude pavement cracking, the initial tangent modulus can be used to calculate stress when given the strain or displacement, and the ultimate secant modulus can be used to calculate strain when given the stress load. This procedure is not appropriate when the initial tangent modulus is greatly different from the ultimate secant modulus since the results would then be too conservative. Further, large differences in these two moduli would indicate stability problems (shoving, rutting) and not pavement cracking.

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*References given at the conclusion of the text.

EXPERIMENTAL PROCEDURES

The mixtures consisted of AC-10 asphalt and Georgetown limestone with Asphalt Institute Gradation IV-B where the maximum size was 3/4 in. Asphalt content was six percent of the aggregate weight, or 5.66 percent of the mixture weight. The components of Mix 1, the regular mixture, were preheated, mixed, and compacted at 300°F. The components of Mix 2, the high temperature mixture, were preheated, mixed, and compacted at 450°F. The experimentally determined SG_e* of the asphalt and aggregate were 1.002 and 2.68, respectively, which results in a theoretical SG_t of 2.45, neglecting the aggregate absorption.

The cakes of flexible concrete measuring 17½ in. diameter by 2 in. thick were prepared using the kneading compactor developed by Jimenz (6) and mentioned briefly in his paper (7). Details concerning operation of this compactor were also given by Layman (8). Briefly stated, the mixture components and the mold were preheated to the selected compaction temperature. The components were mixed for approximately two minutes, tilt compacted for two minutes, and then level compacted for thirty seconds.

The cakes were sawed with a diamond bit blade into test specimens having a nominal cross section of 1.5 in. x 1.5 in. The nominal lengths were 4 in. for compression and 5 in. for tension. The air void content of the test specimens ranged from 2.9 to 12.3 percent for the regular mixture, averaging 8.8 for 23 samples, and ranged from 0 to 6.6 percent for the high temperature

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*See Nomenclature at the conclusion of the text (following the references).

mixture, averaging 3.0 for 32 samples. The percent air voids were calculated from the indicated relations given in the nomenclature, where the specimen volume was calculated from an average of the measurements of the four edges for each dimension.

An Instron Testing Machine with an environmental control chamber was used to perform the constant strain rate tests. The compression samples were placed between platens fixed on the crosshead and on the compression bench.

The tension configuration was prepared by bonding each end of the samples to a 2 in. diameter, 2½ in. long aluminum cylinder using 1 part hardener[1] per seven parts epoxy resin.[2] Tape was wrapped around the end of the cylinder to prevent run-off of the epoxy. The cure time was accelerated by placing the samples in an oven for 30 minutes at 200°F with specimen stability achieved using a wooden fixture. To minimize bending moments during testing, a universal joint was attached to each end of the test configuration.

The test temperatures were -50 (well below the glass point of the binder), 0, 75, and 150°F for uniaxial compression; and 25, 75, and 125°F for the uniaxial tension test. The upper temperature extreme for the tension tests was limited due to the low strength and fragility of the specimens. An attempt was made to obtain 150°F tension data, and over 20 specimens failed during placement of the samples in the fixtures or before the test could be initiated. The nominal strain rates used at each temperature were

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[1] Diethylene Triamine, Ring Chemical Company, Houston, Texas.

[2] Epon Resin 828, Shell Chemical Company, New York, New York.

0.0005, 0.05, and 5 in./(in.-min.), which are the slowest, medium, and fastest strain rates that can be obtained with the given machine when using a test specimen having a 4 in. gage length.

EXAMPLE CONSTITUTIVE RELATIONS

The constitutive relations for the compression in the 300°F mixture tests were determined in an exercise to explore suitable functional forms for use in pavement design.

A best fit straight line was drawn through the ultimate stress data points for each temperature (Figure E1)* and the mean center of the intersections of these projected lines occurred at R = 2000 in./in. - min., σ_u = 380 psi (Figure E2). The best fit of straight lines from this center and running through the data points is given in Figure 1. The equation of these lines is,

$$\sigma_u = a(T)R^{b(T)}$$

and the constants were determined to be,

a(T) = 787, 685, 312, 77.5

b(T) = 0.0957, -0.0775, 0.0262, 0.210

for T = -50, 0, 75, 150°F, respectively.

From plots of log a vs T (Figure E3) and b vs T (Figure E4) it was found that straight lines on these plots could be used for a close approximation in the temperature range of 0 to 150°F. The equations for the straight line approximations of a and b were,

$$b(T) = -0.104 + 0.00189T$$

$$a(T) = 835\ (0.985)^T$$

so that the final equation for the ultimate compressive stress was determined to be,

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*Figures E1 through E7 comprise Appendix E of this report.

$$\sigma_u(-,R,T)^* = 835(0.985)^T R^{-0.104 + 0.00189T} \text{ psi.}$$

It was noted that very small changes occurred in the ultimate compressive strain relative to the variation of the ultimate stress as a function of temperature. Therefore, the ultimate strain data were averaged over the temperatures and the variation of this average strain versus R (Figure E5) plotted the straight line shown in Figure 1, with the resultant equation,

$$\varepsilon_u(-,R) = 2.495R^{0.101} \text{percent.}$$

Using the above equations, the thermoviscoelastic equation for the ultimate secant modulus in compression can be approximated by the relation,

$$E_s(-,R,T) = 33.4(0.985)^T R^{-0.205 + 0.00189T} \text{ ksi.}$$

If straight line approximations are drawn through the compression data for tangent modulus, the equation form will be,

$$E_t(-,R,T) = a(T)R^{b(T)}.$$

Plots of a vs T (Figure E6) and b vs T (Figure E7) gave a straight line and a parabola, respectively. The equations for these curves were then used for an approximate expression for the initial tangent modulus, to wit,

$$E_t(-,R,T) = (29.0 - 0.163T)R^{(6.50 \times 10^{-6}T^2 - 0.265)} \text{ ksi.}$$

The above equations illustrate the way in which fundamental data can be used to develop constitutive equations useful in the design of flexible pavement.

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*Negative sign indicates compression data.

DISCUSSION OF RESULTS

The test data were reduced using the indicated relations included in the nomenclature and the tabulated results are given in Appendix A. The results for the uniaxial compression and tension properties of both mixes considered here are graphically portrayed in Figures 1 through 4, where the property functions are given in terms of temperature and strain rate.

As shown in Figures 1-4, the ultimate strain varied in a relatively minor fashion while the ultimate stress varied drastically as a function of rate of loading and temperature. These trends as well as the numerical values compare favorably with the previous work by Tons and Krokosky (9) even though different procedures for mixture compaction and specimen preparation were used here.

The Smith failure envelopes for uniaxial tension and compression are shown in Figures 5 and 6 for the regular mix and the overheated mix, respectively, and a composite of the four envelopes obtained are given in Figure 7. The envelope indicates the permissible bounds of ultimate stress and ultimate strain for a given set of parameters (in this case the type of loading, type of mix and temperature). Thus an examination of Figure 7 indicates that increasing the mixing temperature reduced the maximum strain the mix may be capable of withstanding without failure. Another way of saying it is that the domain to the left of the envelope contains permissible values of maximum stress and maximum strain, whereas the domain to the right of the envelope contains values which will not occur. The <u>net result</u> of overheating the mix with constant compaction time was that the ultimate stress increased, the ultimate strain decreased, and the air voids decreased.

This shift in failure envelope shows that overheating of the mixture is very undesirable for thin (say less than 2 in.) flexible pavement, as thin pavement essentially follows the deflection of the base for imposed wheel loads, and the resultant strains are usually large.

The average air voids decreased from 8.8 to 3.0 percent due to increasing the mixing and compaction temperature from 300 to 450°F, since all of the conditions were otherwise constant. Manifestations of increased mixing and compaction temperatures are decreased binder film thickness on the aggregate due to decreased viscosity of the aggregate, higher density due to less flow resistance in the mixture, increased absorption of the binder into the aggregate, and oxidation of the binder causing a stiffer pavement. The idea was to crudely quantify "embrittlement" in terms of net fundamental mechanical properties and therefore the individual physical changes were not of interest here.

As stated in objective 6 of the scope of the reported research, two mix temperatures were considered while all other directly controlled variables and conditions were kept the same. It was also stated in the background on the reported research that the primary purpose of this work was to perform limited studies in order that experimental difficulties might be better anticipated in future work.

The change in the fundamental material properties were attributed here directly to mixing and compacting temperatures and not air voids, since all other directly controlled conditions and variables were held constant. The change in air voids was undoubtedly a result of the increased temperature and definitely affected the measured material properties. However, the causative factor was the mixing and compacting temperature and thus the results are discussed in terms of the temperatures involved.

A few 10 in. length specimens were tested. For instance, of the two data points for T = 75, R = 0.02, Figure 2, one is for a 10 in. length and the other is for a 5 in. length, and the difference in properties (σ_u, ε_u) is only about 10 percent. The error due to end effects is then negligible relative to other sources of error. The major source of error is believed to be due to the angularity and orientation of the largest size aggregate in the mix since it was noted that all of the tensile specimens failed in a plane at a large stone. The orientation of the large stones also affects the mode of rock locking in compression, which was observed from the unusual nature of a few of the force-deformation traces.

Using a 4 in. length sample in a compression control test, a calibrated LVDT having a 1 in. gage length was placed on each side of the specimen and the average of the LVDT readings showed that the indicated strain from the Instron chart amounted to 85 percent of the strain in the central section measured by LVDT. The strain calculated from the Instron chart should be less than the strain in the central section since the end effects induce a biaxial stress state at and about the loading platens, thereby decreasing the local strain.

Machine deformations were not accounted for in this study, but it is known that the machine deformation amounted to about 5 percent of the strain indicated on the Instron chart, so the true specimen strain is about 95 percent of the values reported here. The difference between the average strain over the specimen length and the strain in the central section was then about 10 percent. The influence of end effects are decreased with increased specimen length. The true average strain could therefore be expected to agree with the strain in the central section within about 5 percent if the specimen length were about 6 in.

It is clear that the experimental variation of the results would be considerably less if the maximum aggregate size is small compared to the thickness of the specimen cross section. However, the given aggregate gradation and the specimen thickness corresponds to the gradation and pavement thickness often used on Texas highways. Statistical analysis of the data was not warranted for the small number of tests reported here.

CONCLUSIONS

1. Ultimate stress does not seem to be a suitable failure criteria for flexible concrete due to the evidence and facts presented in this report.
2. For practical purposes, ultimate strain appears to be a suitable failure criteria for flexible concrete.

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NOMENCLATURE

Symbol	Variable	Unit
A	Cross sectional area	in.^{2}
a,b,c,d	Constants	
CH	Crosshead rate	in./min.
CS	Chart speed	in./min.
E_s	Secant modulus at ultimate force, $\sigma_u/(10\varepsilon_u)$	ksi
E_t	Initial tangent modulus, $F_t CS/(1000\ ARG_t)$	ksi
FS	Final full scale sensitivity of load cell	lb.
F_t	Arbitrary force on initial slope of force-displacement trace	lb.
F_u	Ultimate force	lb.
G_t	Graph length from start of test to F_t	in.
G_u	Graph length from start of test to F_u	in.
H	Height (average of 4 edges)	in.
L	Length (average of 4 edges)	in.
P_i	Percentage of component in mixture	percent
R	Strain rate, CH/L	in./(in.-min.)
SG_e	Experimentally determined specific gravity	numeric
SG_i	Specific gravity of mix component	numeric
SG_t	Theoretical specific gravity $100/\Sigma(p_i/SG_i)$	numeric
T	Temperature	°F
V	Volume	in.^{3}
v	Air voids, $100(1-SG_e/SG_t)$	percent
W	Width (average of 4 edges)	in.
w	Weight of specimen	grams
ρ	Density, 3.81 w/V	lb./ft.^{3}
ε_u	Ultimate strain, $100R\ G_u/CS$	percent
σ_u	Ultimate stress, F_u/A	psi

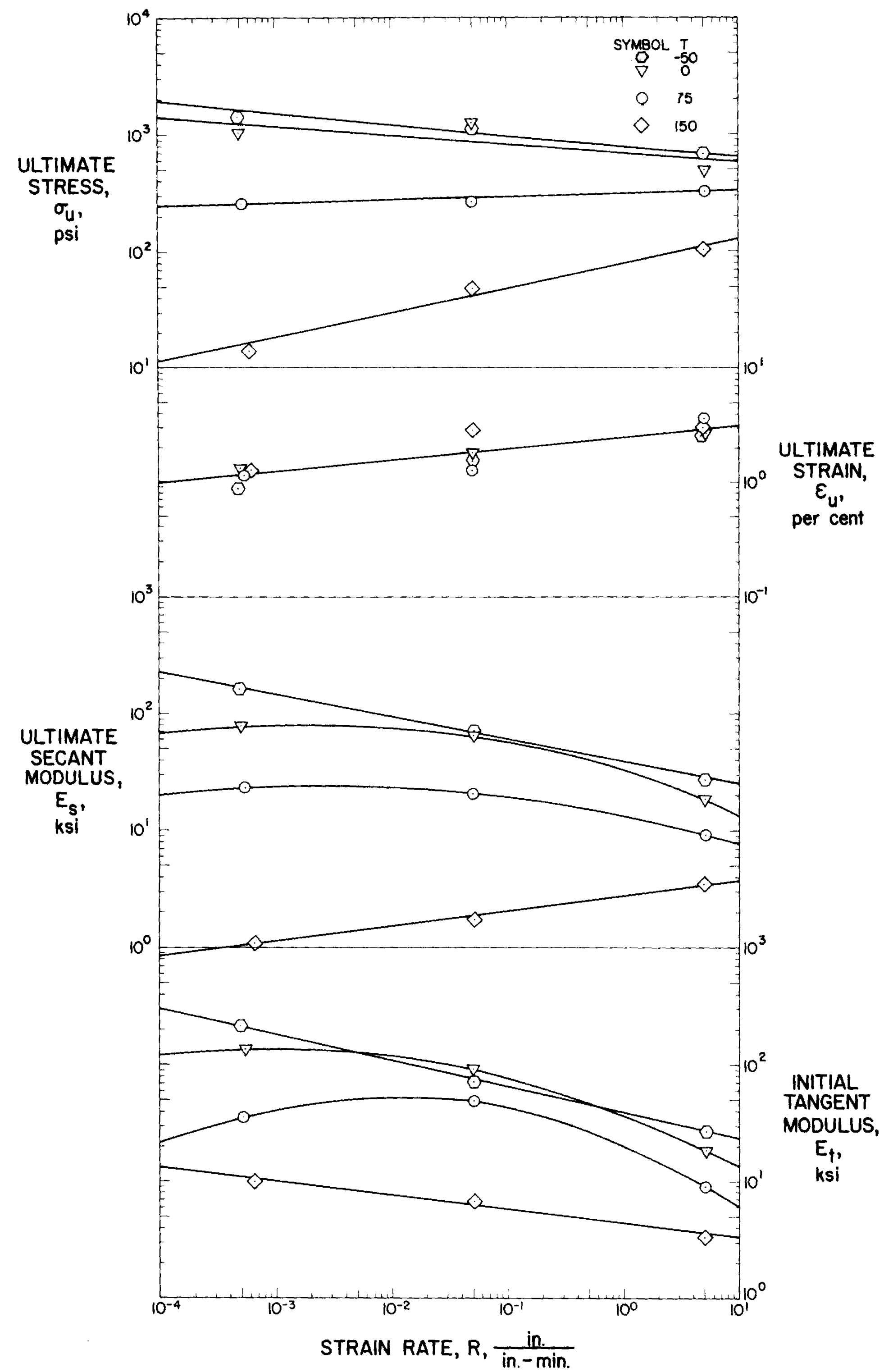

Fig. 1. Thermoviscoelastic Properties of a Regular Mix in Uniaxial Compression.

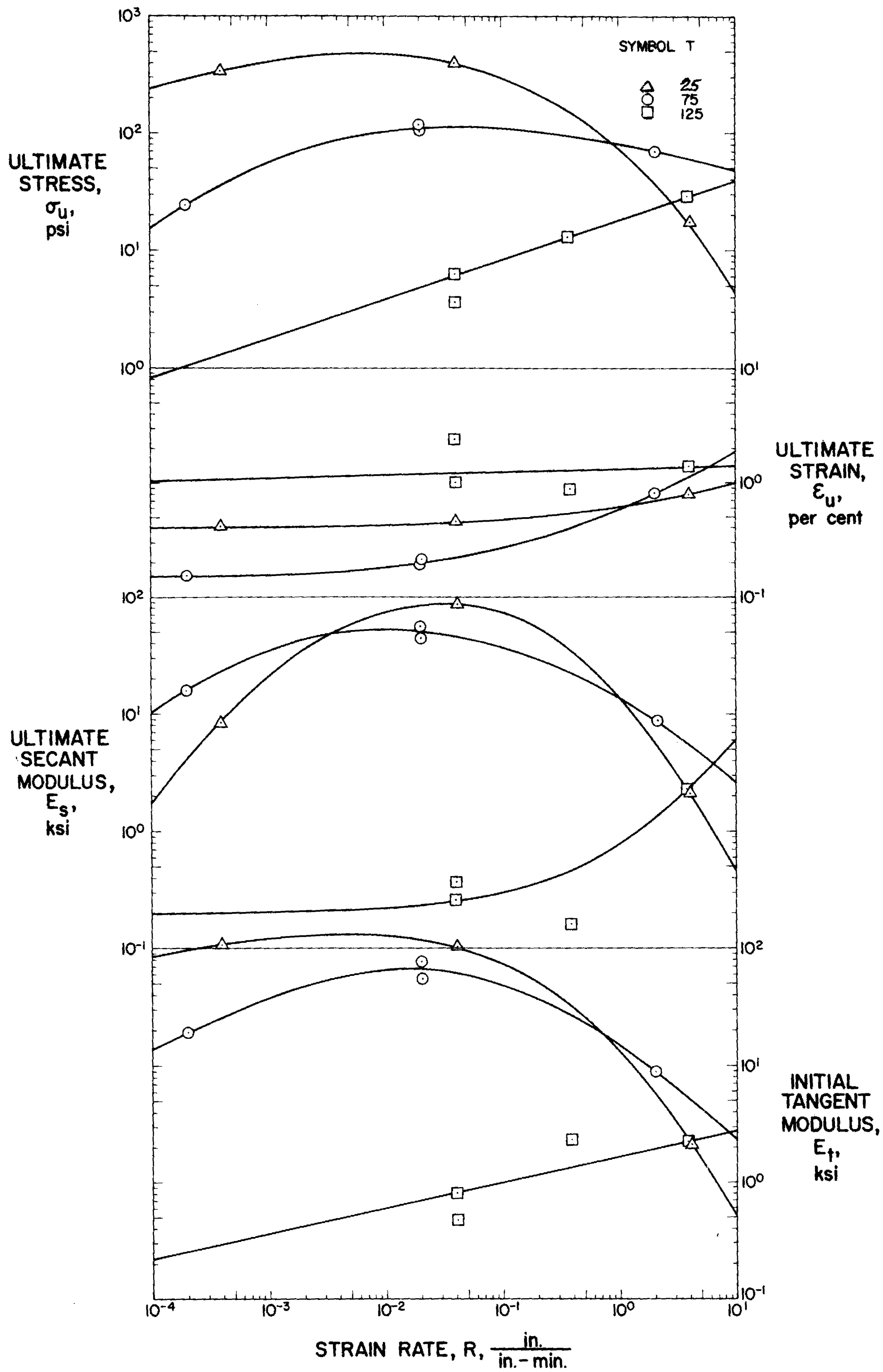

Fig. 2. Thermoviscoelastic Properties of a Regular Mix in Uniaxial Tension.

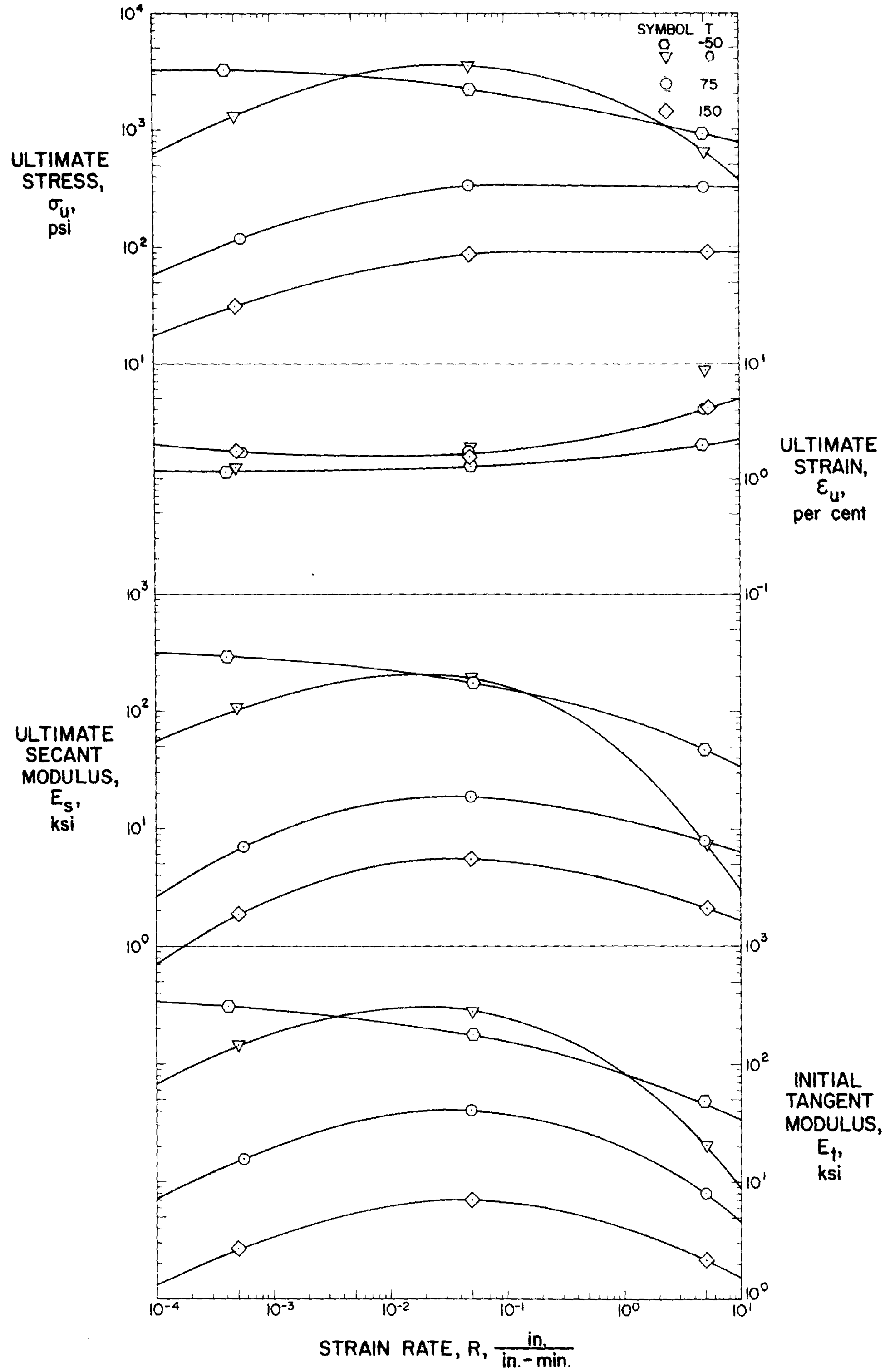

Fig. 3. Thermoviscoelastic Properties of an Overheated Mix in Uniaxial Compression.

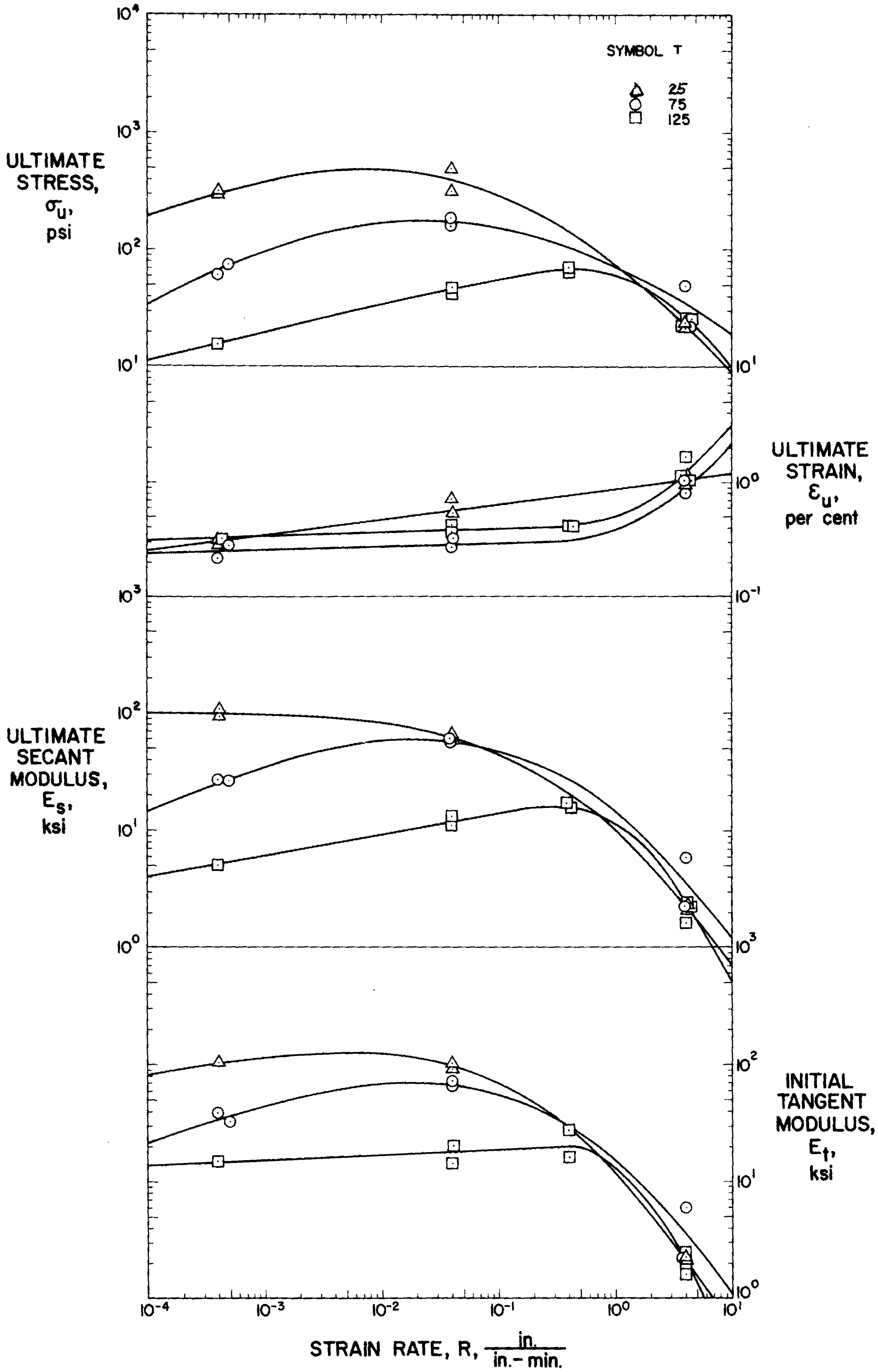

Fig. 4. Thermoviscoelastic Properties of an Overheated Mix in Uniaxial Tension.

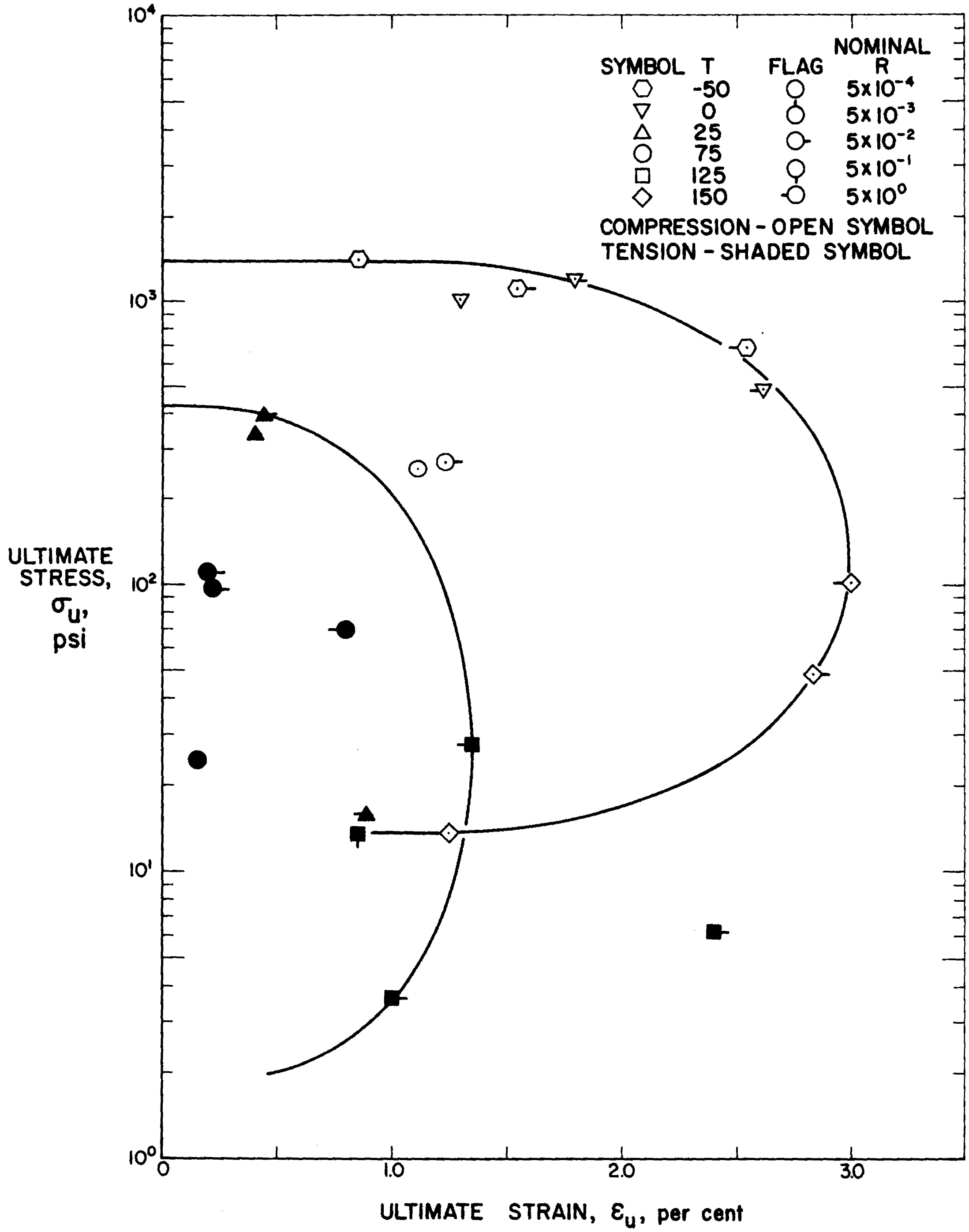

Fig. 5. Smith Failure Envelopes for a Regular Mix.

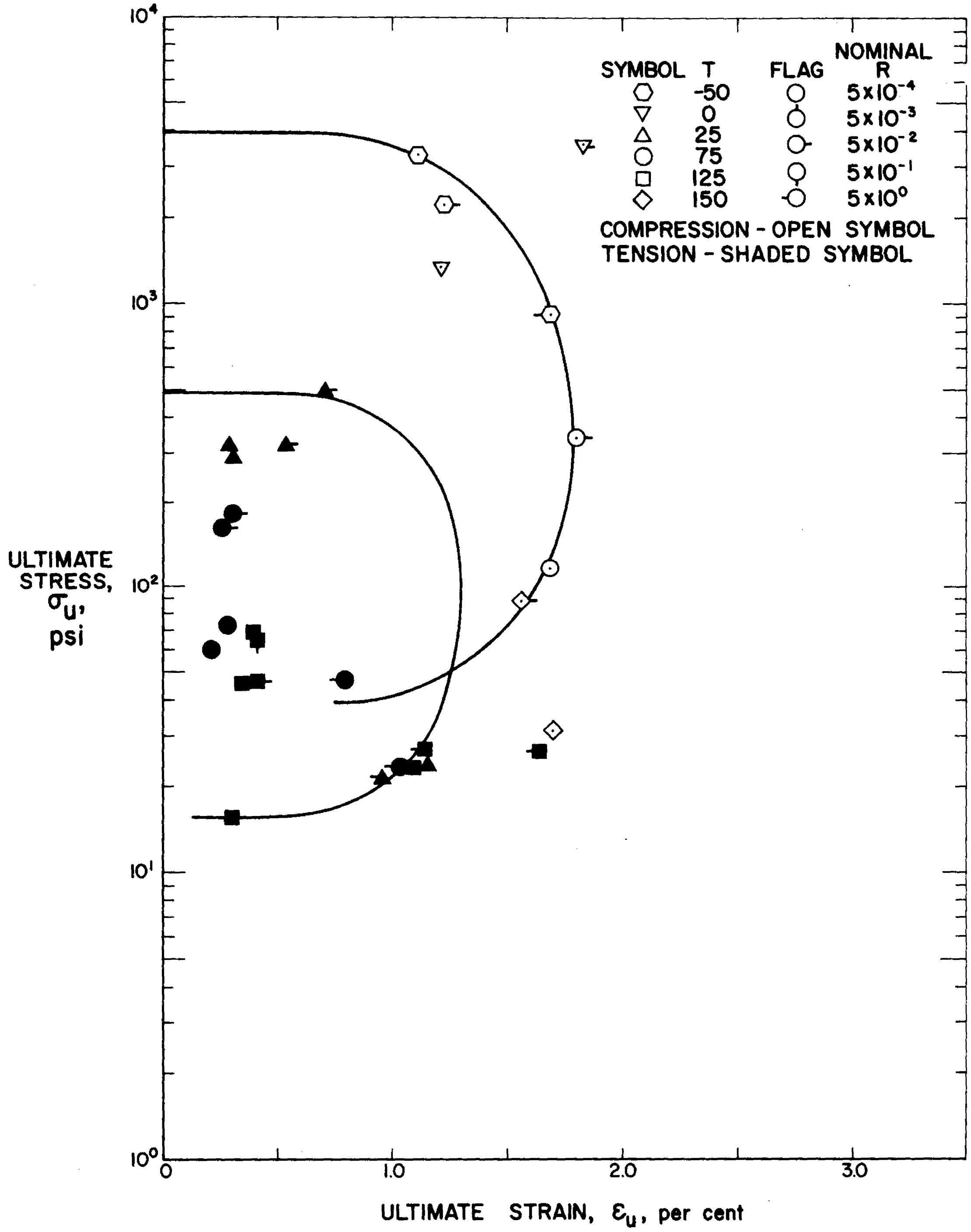

Fig. 6. Smith Failure Envelopes for an Overheated Mix.

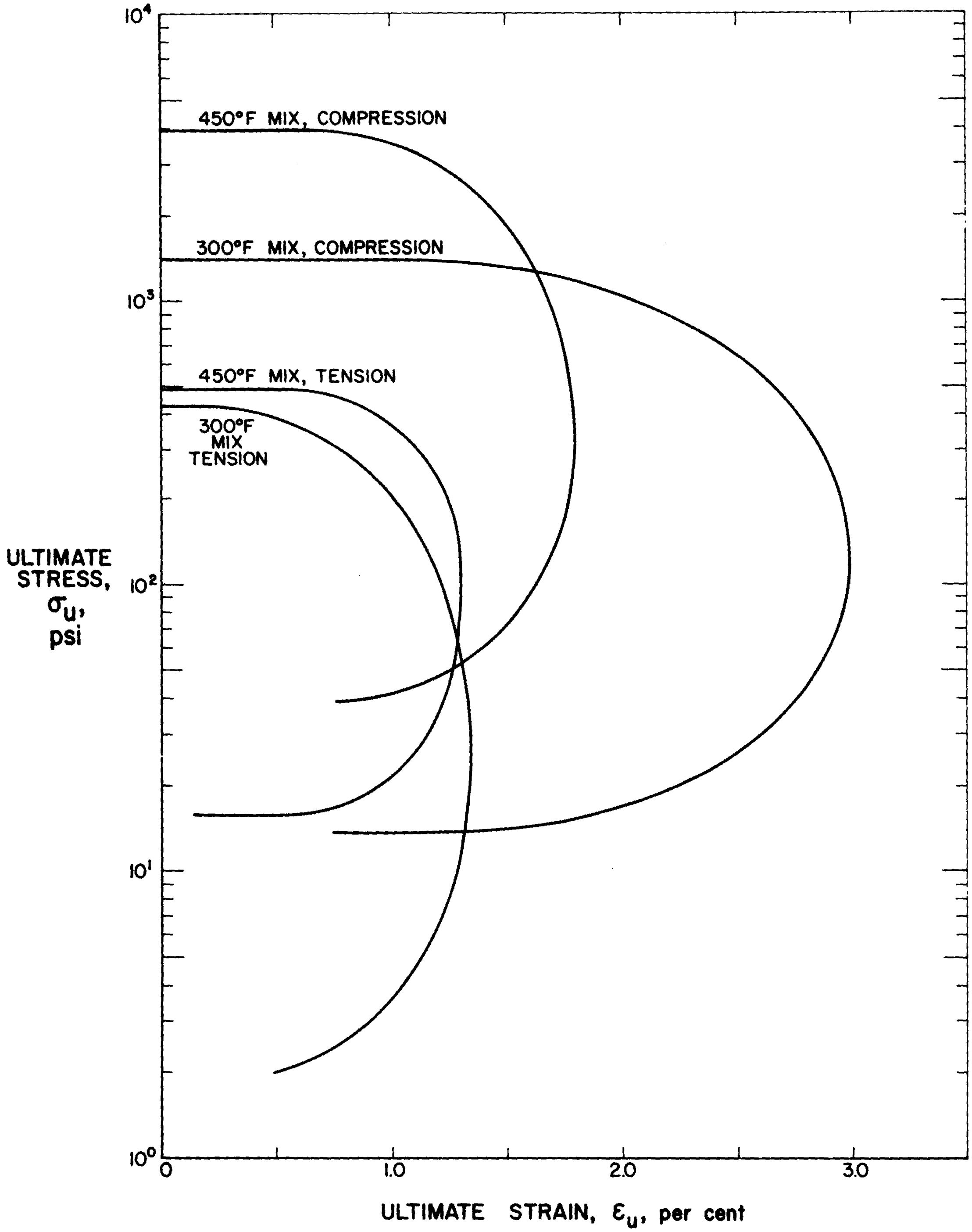

Fig. 7. Composite Failure Envelopes.

APPENDIX A: Data on Materials and Specimens and Tabulated Results

Table A1. **DATA ON ASPHALT***

AC 10, Grade, Received 16 June 1968, Asphalt No. 11, Order ADH-2536, Barrel No. 34B

Specific Gravity (@ 77°F)	1.004
Melting Point (Ring & Ball, °F)	116
Penetration (@ 77°F)	106
Ductility (@ 77°F)	204
Solubility (CCl_4, %)	99.92
Flash Point (Cleveland Open Cup, °F)	565
Oliensis Spot Test	Negative
Viscosity (@ 275°F, Furol sec.)	162
Viscosity (@ 275°F, Strokes)	3,20
Viscosity (@ 140°F, Strokes)	1392
Relative Viscosity of Asphalt after oven aging 15 u film 2 hrs. @ 225°F, Viscosities @ 77°F	2.7

Thin Film Tests, 1/8" @ 325°F for 5 hours:

Loss of Heating (%)	0.03
Penetration (@ 77°F on residue, % of original)	66.0
Ductility (@ 77°F on residue)	200

*Manufacturer's data.

Table A2. GRADATION OF AGGREGATE

TEXAS CRUSHED LIMESTONE — 12 NOVEMBER 1969

	Percent Retained						
	Modified Grade #3	5/16 F	Premium Screenings	2110 Georgetown Screenings	Combined Analysis		Asphalt Institute Grading IV B
Size	Mix.-Comp.	Mix.-Comp.	Mix.-Comp.	Mix.-Comp.	Retained	Passing	Specifications
3/4	0	0					0
1/2	55.3-17.7	0			17.7	82.3	0-20
3/8	93.2-29.8	0			29.8	70.2	10-30
4	96.9-31.0	13.2- 3.2	0	0	34.2	65.8	30-50
8	97.3-31.1	84.4-19.4	3.3- .8	1.9- .4	51.7	48.3	50-65
16	97.5-31.2	95.5-22.0	22.5- 5.2	48.6-10.7	69.1	30.9	
30	97.6-31.2	96.9-22.3	40.9- 9.4	67.3-14.8	77.7	22.3	71-82
50	97.7-31.3	97.2-22.4	54.0-12.4	79.4-17.5	83.6	16.4	77-87
100	97.8-31.3	97.4-22.4	62.8-14.4	89.4-19.7	87.8	12.2	84-92
200	98.0-31.4	97.6-22.5	68.9-15.8	96.1-21.1	90.9	9.2	90-96
PAN	100 -32	100 -23	100 -23	100 -22	100	0	

TABLE A3. DATA ON REGULAR MIXTURE, COMPRESSION TESTS

Spec. No.	Cake No.	FS	CH	CS	W	H	L	A	V	w	ρ	SG_e	v
5	1	5000	0.002	2	1.41	1.53	4.12	2.16	8.88	326	140	2.24	8.4
4	1	5000	0.2	1	1.50	1.48	3.87	2.22	8.60	311	138	2.21	9.7
11	2	10000	20	20	1.48	1.44	3.94	2.13	8.40	310	141	2.25	8.1
25	2	5000	0.002	0.2	1.50	1.59	4.00	2.39	9.56	358	143	2.29	7.9
16	2	5000	0.2	10	1.44	1.53	3.97	2.20	8.73	313	137	2.19	10.6
17	2	5000	20	50	1.59	1.50	3.81	2.39	9.10	343	144	2.30	5.1
14	2	1000	0.002	0.5	1.45	1.42	3.69	2.06	7.60	284	142	2.28	8.4
13	2	1000	0.2	5	1.38	1.42	3.95	1.96	7.74	282	139	2.22	9.3
12	2	2000	20	50	1.42	1.53	3.91	2.16	8.44	302	136	2.18	10.9
1	1	500	0.002	0.2	1.37	1.48	3.02	2.04	6.15	229	142	2.27	7.2
2	1	500	0.2	10	1.48	1.50	3.89	2.23	8.66	310	136	2.19	10.6
3	1	500	20	20	1.22	1.34	4.00	1.63	6.52	239	140	2.24	8.5
													8.7 Average

Table A4. RESULTS FOR REGULAR MIXTURE, COMPRESSION TESTS

Spec. No.	T	R	F_u	G_u	σ_u	ε_u	E_s	F_t	G_t	E_t
5	-50	0.000486	3050	35.4	1412.0	.860	164.2	1000	8.78	217
4	-50	0.0517	2480	0.3	1117.1	1.55	72.1	1000	8.78	72.1
11	-50	5.08	1480	0.1	694.8	2.54	27.4	1000	8.78	27.4
25	0	0.00050	2425	5.2	1014.6	1.30	78.04	2500	3.1	135.0
16	0	0.0504	2690	3.53	1222.7	1.78	68.7	3000	2.92	92.7
17	0	5.245	1170	0.25	489.5	2.62	18.68	3000	2.92	18.68
14	75	0.00054	527	10.35	255.8	1.11	23.04	500	6.25	35.9
13	75	0.0506	527	1.22	268.9	1.23	21.8	500	0.53	47.6
12	75	5.115	710	0.35	328.7	3.58	9.18	500	0.53	9.18
1	150	0.000662	28	3.76	13.7	1.24	1.10	300	4.55	9.78
2	150	0.0514	108	5.51	48.4	2.83	1.71	400	5.10	6.83
2	150	5.00	169	0.12	103.7	3.0	3.45	400	5.10	3.45

Table A5. DATA ON REGULAR MIXTURE, TENSION TESTS

Spec. No.	Cake No.	FS	CH	CS	W	H	L	A	V	w	ρ	SG_e	v
113	3	1000	0.002	0.2	1.50	1.50	5.06	2.25	11.38	417	140	2.24	8.5
114	3	500	0.2	20	1.50	1.47	5.00	2.21	11.05	401	138	2.22	9.3
115	3	500	20	50	1.50	1.50	5.00	2.25	11.25	409	139	2.22	9.3
18	2	500	0.002	0.5	1.47	1.53	9.97	2.25	22.41	790	134	2.15	12.3
79	2	500	0.1	5	1.44	1.44	4.94	2.07	10.21	363	136	2.17	11.4
10	1	500	0.2	5	1.48	1.45	10.00	2.16	21.60	843	149	2.38	2.9
7	1	1000	20	50	1.63	1.53	9.88	2.48	24.49	914	142	2.28	8.4
103	3	500	0.2	2	1.50	1.51	5.00	2.26	11.30	423	143	2.28	8.4
112	3	500	0.2	2	1.50	1.50	5.00	2.25	11.25	417	141	2.26	7.7
118	3	500	2	20	1.50	1.50	5.31	2.25	11.94	436	139	2.23	9.0
111	3	500	20	50	1.50	1.50	5.03	2.25	11.30	409	139	2.21	9.7
													8.8 Average

TABLE A6. RESULTS FOR REGULAR MIXTURE, TENSION TESTS

Spec. No.	T	R	F_u	G_u	σ_u	ε_u	E_s	F_t	G_t	E_t
113	25	0.00039	760	2.11	337.8	.411	8.2	500	1.08	105.5
114	25	0.04	880	2.26	398.2	.452	88.2	250	0.55	102.9
115	25	4.00	38	0.1	16.9	.800	2.11	250	0.55	2.11
18	75	0.0002	55	3.85	24.4	.154	15.8	100	5.82	19.1
79	75	0.0202	204	0.55	98.6	.222	44.4	250	0.55	54.3
10	75	0.02	239	0.5	110.6	.200	55.3	250	0.38	76.1
7	75	2.024	173	0.2	69.8	.810	8.62	250	0.38	8.62
103	125	0.04	14	1.2	6.2	2.40	0.258	50	2.37	0.466
112	125	0.04	8	0.5	3.6	1.00	0.360	25	0.7	0.794
118	125	0.377	30	0.45	13.3	0.85	0.157	150	1.56	2.26
111	125	3.976	63	0.17	28	1.35	2.07	150	1.56	2.07

TABLE A7. DATA ON AN OVERHEATED MIXTURE, COMPRESSION TESTS

Spec. No.	Cake No.	FS	CH	CS	W	H	L	A	V	w	ρ	SG_e	v
6	1	10000	0.002	0.2	1.53	1.47	4.87	2.23	10.8	422	149	2.39	2.6
65	2	10000	0.2	2	1.50	1.42	3.89	2.13	8.3	340	156	2.50	---
64	2	10000	20	50	1.45	1.42	4.13	2.05	8.5	324	145	2.33	4.9
68	2	5000	0.002	0.2	1.50	1.50	4.06	2.25	9.1	359	151	2.41	1.6
37	1	10000	0.2	10	1.52	1.55	4.00	2.32	9.3	365	150	2.40	2.0
66	2	5000	20	50	1.56	1.61	3.94	2.49	9.8	386	150	2.40	2.0
36	1	500	0.002	0.2	1.50	1.58	3.63	2.35	8.5	319	143	2.29	6.6
34	1	1000	0.2	5	1.59	1.58	4.10	2.49	10.2	384	144	2.30	6.1
35	1	2000	20	50	1.59	1.53	4.06	2.43	9.8	380	148	2.37	3.2
38	1	500	0.002	0.2	1.45	1.48	4.00	2.14	8.6	339	150	2.40	2.0
67	2	500	0.2	2	1.58	1.50	4.13	2.35	9.7	375	147	2.36	3.7
69	2	500	20	50	1.47	1.52	3.88	2.19	8.5	326	146	2.34	4.5
													3.6 Average

TABLE A8. RESULTS FOR OVERHEATED MIXTURE, COMPRESSION TESTS

Spec. No.	T	R	F_u	G_u	σ_u	ε_u	E_s	F_t	G_t	E_t
6	-50	0.00041	7330	5.47	3287.0	1.12	293.5	3000	2.1	312.5
65	-50	0.0514	4690	0.48	2201.9	1.23	179.0	3000	2.1	179.0
64	-50	4.84	1890	0.2	922.0	1.94	47.5	3000	2.1	47.5
68	0	0.00049	2980	4.98	1324.4	1.22	108.6	3000	3.7	147.1
37	0	0.050	8220	3.66	3543.1	1.83	193.6	2500	0.76	284.1
66	0	5.08	1640	0.88	658.6	8.94	7.4	2500	0.48	20.6
36	75	0.00055	277	6.1	117.9	1.68	7.0	250	2.45	15.8
34	75	0.0488	840	1.83	337.3	1.79	18.8	300	0.3	41.4
35	75	4.93	800	0.42	329.2	4.14	8.0	300	0.3	8.0
38	150	0.0005	68	6.85	31.8	1.71	1.9	90	6.2	2.7
67	150	0.0484	208	0.65	88.5	1.57	5.6	263	0.65	7.11
69	150	5.16	201	0.4	91.8	4.13	2.2	263	0.65	2.2

TABLE A9. DATA ON AN OVERHEATED MIXTURE, TENSION TESTS

Spec. No.	Cake No.	FS	CH	CS	W	H	L	A	V	w	ρ	SG_e	v
27	1	500	0.002	0.2	1.53	1.50	4.86	2.29	11.1	443	152	2.43	1.0
31	1	500	0.002	0.2	1.61	1.52	4.87	2.42	11.7	478	156	2.49	---
28	1	2000	0.2	20	1.50	1.52	4.88	2.27	11.0	429	149	2.38	2.9
32	1	1000	0.2	20	1.50	1.63	4.81	2.43	11.6	452	149	2.38	2.9
29	1	500	20	50	1.56	1.50	4.83	2.34	11.3	440	148	2.38	2.9
82	2	500	20	50	1.56	1.55	4.94	2.41	11.9	458	147	2.35	4.1
30	1	500	0.002	0.2	1.50	1.53	4.00	2.29	9.16	367	153	2.45	0
15	1	500	0.002	0.2	1.50	1.50	4.94	2.25	11.1	447	153	2.46	---
19	1	500	0.2	20	1.55	1.56	4.94	2.41	11.8	449	145	2.32	5.2
33	1	500	0.2	20	1.56	1.56	4.87	2.43	11.8	465	150	2.40	2.0
70	2	500	20	50	1.56	1.53	5.00	2.38	11.9	452	145	2.32	5.2
71	2	1000	20	50	1.55	1.53	5.00	2.36	11.8	445	144	2.30	6.1
21	1	500	0.002	0.2	1.63	1.52	4.94	2.47	12.2	475	148	2.38	2.9
8	1	500	0.2	2	1.50	1.52	4.94	2.26	11.2	438	150	2.41	1.6
22	1	500	0.2	20	1.56	1.58	4.86	2.46	11.9	447	143	2.29	6.6
9	1	500	2	20	1.59	1.56	4.87	2.48	12.0	451	143	2.29	6.6
83	2	500	2	20	1.52	1.52	5.00	2.28	11.4	452	151	2.42	1.2
20	1	500	20	50	1.50	1.61	4.88	2.40	11.6	471	155	2.48	---
24	2	500	20	50	1.48	1.50	4.88	2.22	10.8	412	145	2.33	4.9
26	1	500	20	50	1.48	1.50	4.88	2.22	10.8	435	154	2.46	---
													3.7 Average

TABLE A10. RESULTS FOR AN OVERHEATED MIXTURE, TENSION TESTS

Spec. No.	T	R	F_u	G_u	σ_u	ε_u	E_s	F_t	G_t	E_t
27	25	0.00041	727	1.4	317.5	0.29	109.5	175	0.35	106.5
31	25	0.00041	704	1.5	290.9	0.31	93.8	250	0.52	96.9
28	25	0.0410	1115	3.52	491.2	0.72	68.2	250	0.6	89.5
32	25	0.0416	773	2.58	318.1	0.54	58.9	250	0.48	103.1
29	25	4.14	56	0.14	23.9	1.16	2.1	250	0.48	2.1
82	25	4.05	51	0.12	21.2	0.97	2.2	250	0.48	2.2
30	75	0.00050	168	1.1	73.4	0.28	26.2	207	1.1	32.9
15	75	0.00040	135	1.1	60.0	0.22	27.3	200	1.15	38.6
19	75	0.0405	440	1.57	182.6	0.32	57.1	495	1.57	64.6
33	75	0.0410	393	1.3	161.7	0.27	59.9	450	1.26	71.7
70	75	4.00	56	0.13	23.5	1.04	2.3	450	1.26	2.3
71	75	4.00	112	0.1	47.5	0.80	5.9	450	1.26	5.9
21	125	0.00040	38	1.5	15.4	0.30	5.1	110	1.5	14.8
8	125	0.0404	105	0.21	46.5	0.42	11.1	140	0.21	14.6
22	125	0.0411	113	1.7	45.9	0.35	13.1	172	1.7	20.0
9	125	0.4106	163	0.2	65.7	0.41	16.0	172	1.7	16.0
83	125	4.40	158	0.2	69.3	0.40	17.3	250	0.2	27.4
20	125	4.10	57	0.13	23.8	1.07	2.2	250	0.2	2.2
24	125	4.10	58	0.2	26.1	1.64	1.6	250	0.2	1.6
26	125	4.10	60	0.14	27.0	1.15	2.3	250	0.2	2.3

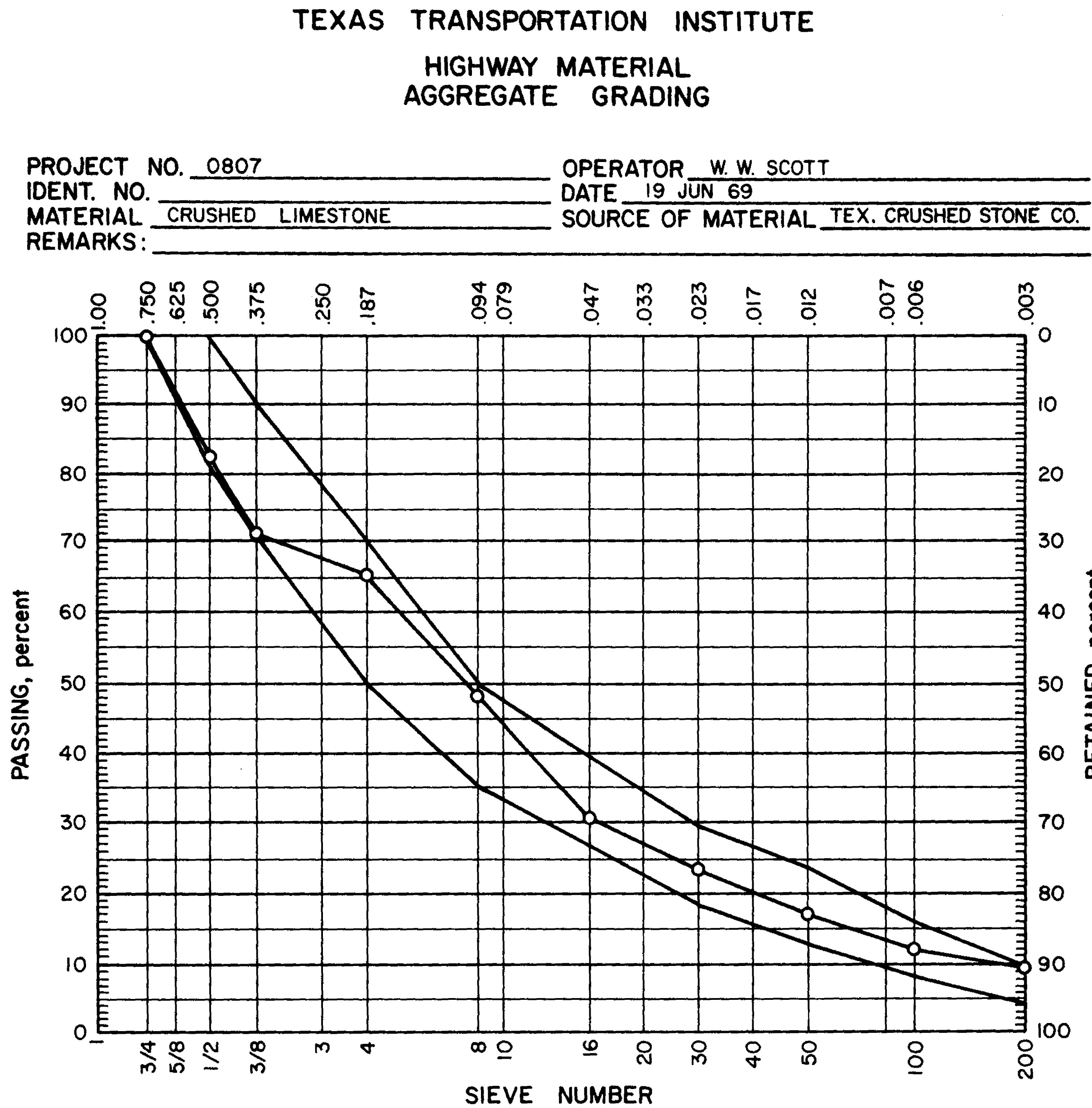

Figure A1. Aggregate Grading Curve.

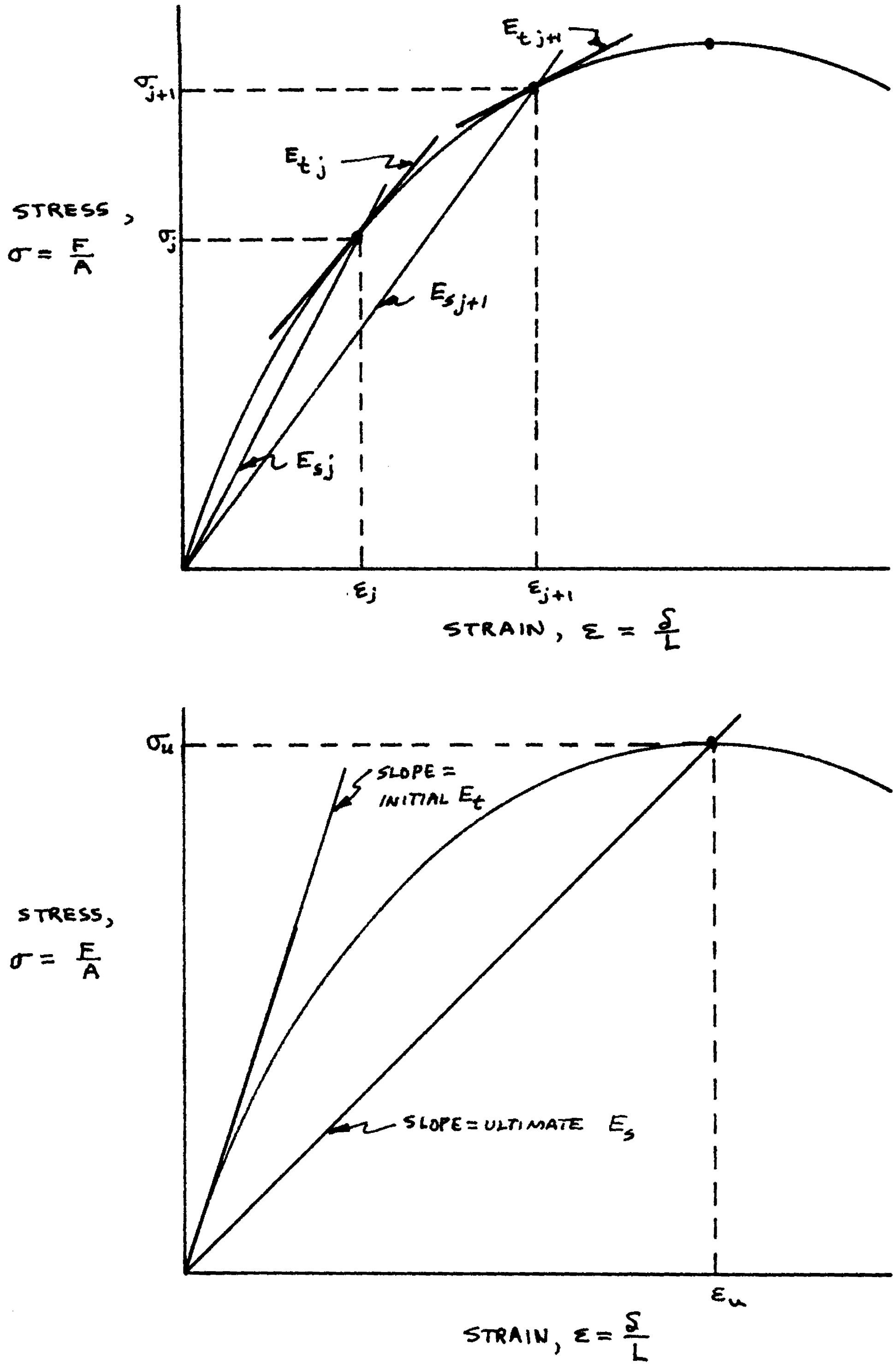

Figure A2. Definitions of Moduli.

APPENDIX B: Procedure for Mixing Asphalt

First select aggregate and asphalt to be used. Weigh out the desired amount of aggregate and place in oven at desired temperature. Weigh out more than the desired amount of asphalt and put in the same oven, separate from the aggregate. Heat overnight at temperatures which will be used in compaction. Re-weigh the aggregate and add to it 6.0% asphalt. Place this in the food mixer and mix it thoroughly while heating the mixer pot with a bunsen burner. A preheated steel plate covered with an asbestos disk and then a foil disk is put at the bottom of the mold. Take the mix and put it in the preheated mold. Smooth the mixture out in the mold with a spoon and then pound it with a preheated steel tamper. Place a brown paper disk and then a preheated steel disk on top and carry the mold (on a large wooden disk) to the compaction machine. Remove the wooden carrier disk when positioning the mold, bolt it in place, and run the compaction machine for two minutes at right tilt and then thirty seconds at left tilt, moving the mold at all times. Take the mold off of the formed cake and slide the asbestos disk with the cake on it onto a table. Allow it to cool several hours before handling again.

APPENDIX C: Method for Cutting Asphalt Samples

After the prepared asphalt cake has sufficiently cooled for handling, mark off the size of the samples desired. When marking be sure to allow for the width of the saw blade. When the cake has been correctly marked and checked, it is now ready to be cut. Before turning on the saw make sure that the water is playing on the blade. Line the sample up meticulously to insure that the saw blade does not go off the desired cutting line. Turn the blade on and slowly push the sample towards the blade. Do not apply enough pressure to make the saw motor pull down. While sawing, keep the saw table clean, fingers out of the way, and the blade off when not in use.

APPENDIX D: Procedure for Bonding and Cleaning Samples

When bonding asphalt samples to the aluminum cylinders, mix fourteen grams of epoxy[1] and two grams of hardener[2] for each pair of cylinders. The cylinder surface must be level and clean. Put tape around the top to hold the resin in. Only one end of a sample can be bonded at a time. Place the sample in the center of the resin and cylinder and heat for thirty minutes at 200°F. Remove the partially prepared samples and mix the resin for the cylinders to be bonded to the other side. The new cylinders with resin are placed in the wooden holders. When the samples already bonded are cool enough to handle, lower them from the top of the holders onto the cylinders with resin. Place these in the oven for thirty minutes at 200°F. They should then be ready for tension testing when they are cooled to the desired temperature. To clean the cylinders take hammer and chisel and knock away most of the epoxy and asphalt clinging to the cylinder. Take care not to damage the cylinder surface. Remove the remaining material with the circular sander. Make sure that the cylinder surface is level after sanding and that it is turned slightly while sanding.

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[1]Type used was Epon Resin 828, manufactured by the Shell Chemical Company, a division of Shell Oil Company, New York, New York.

[2]Type used was Diethylene Triamine, manufactured by the Ring Chemical Company, Houston, Texas.

APPENDIX E: Graphs of Functions for Constitutive Relations

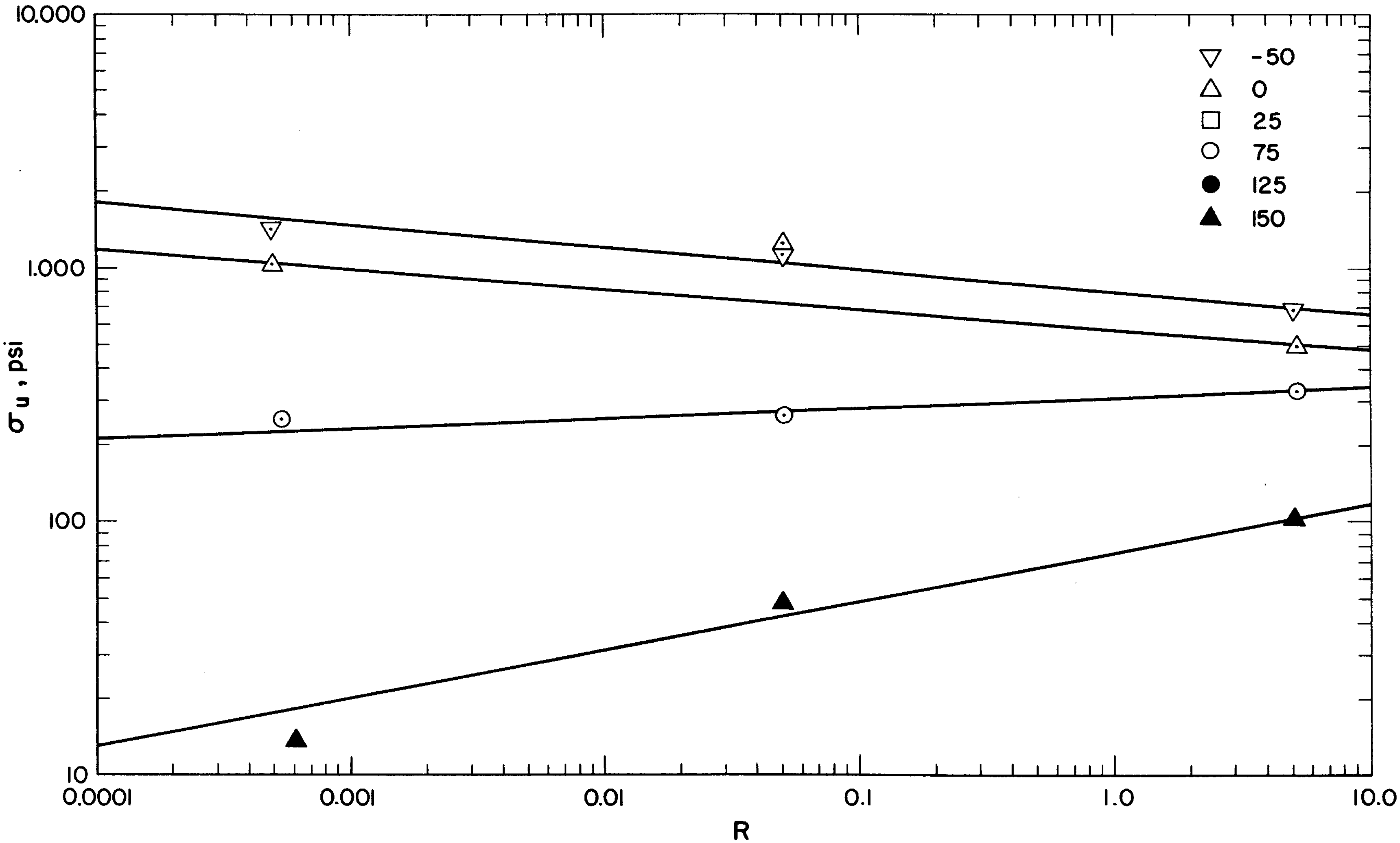

Fig. E1. Ultimate Stress vs Strain Rate.

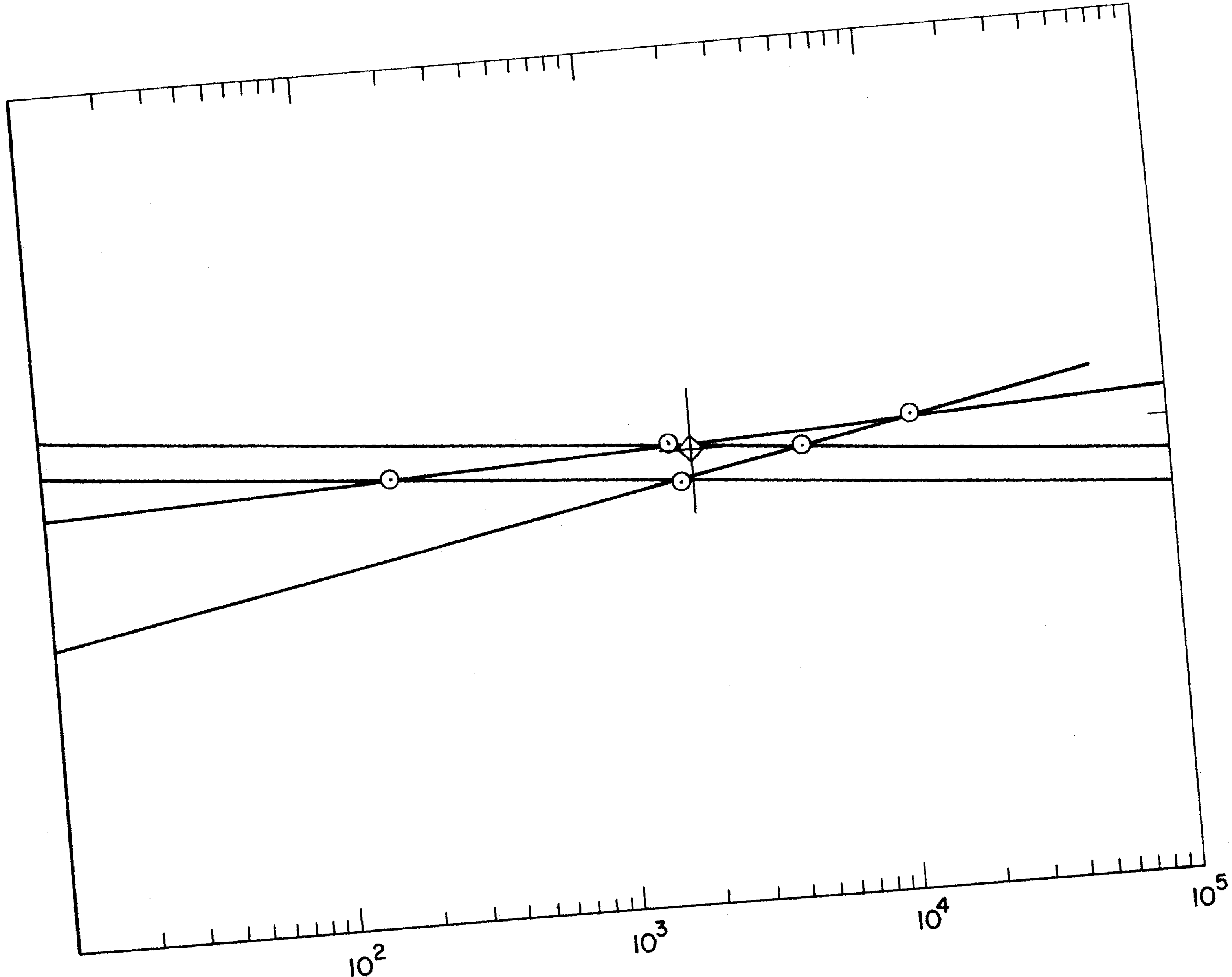

Fig. E2. Extension of Fig. E1.

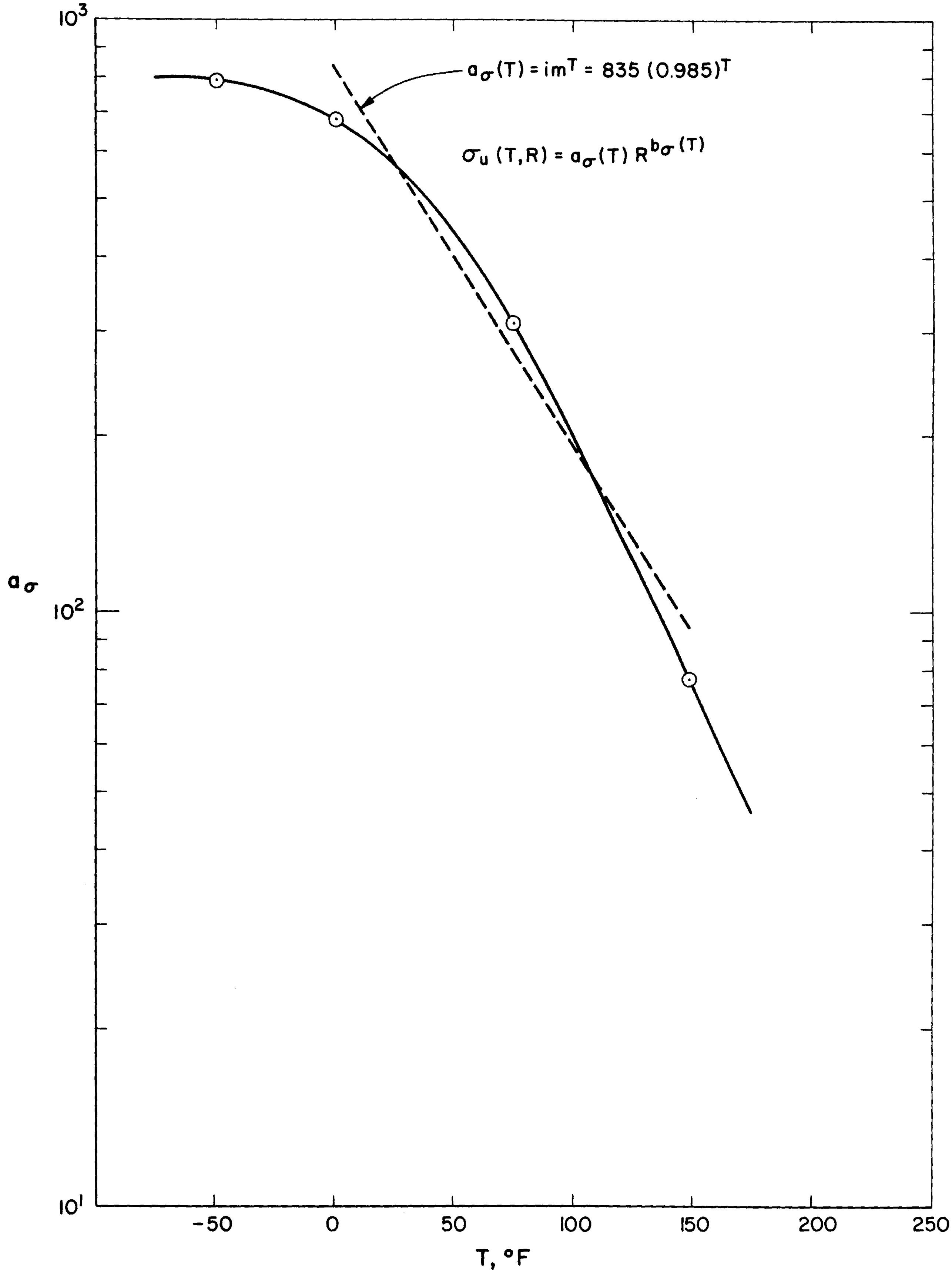

Fig. E3. Variation of a_σ with Temperature.

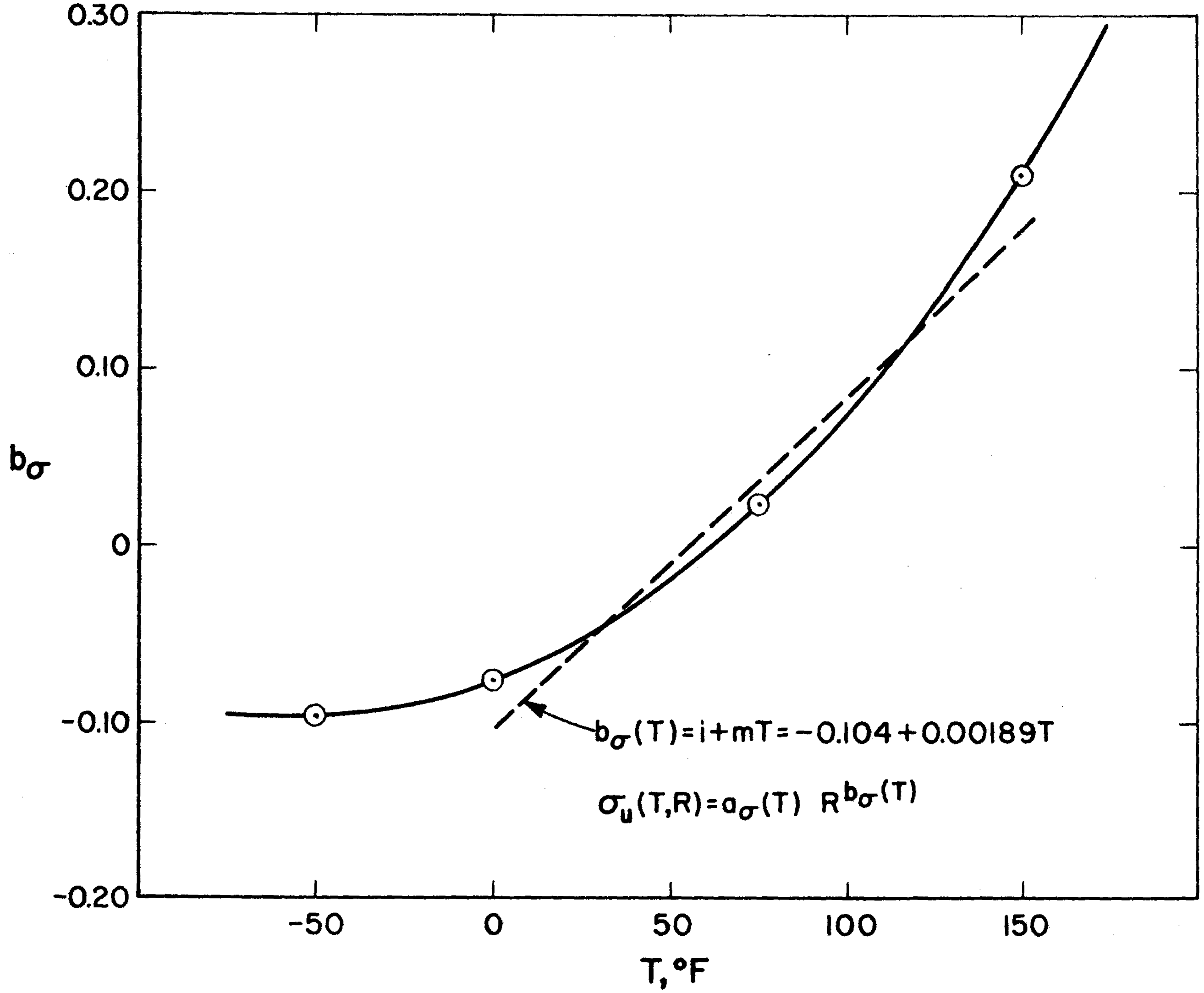

Fig. E4. Variation of b_σ with Temperature.

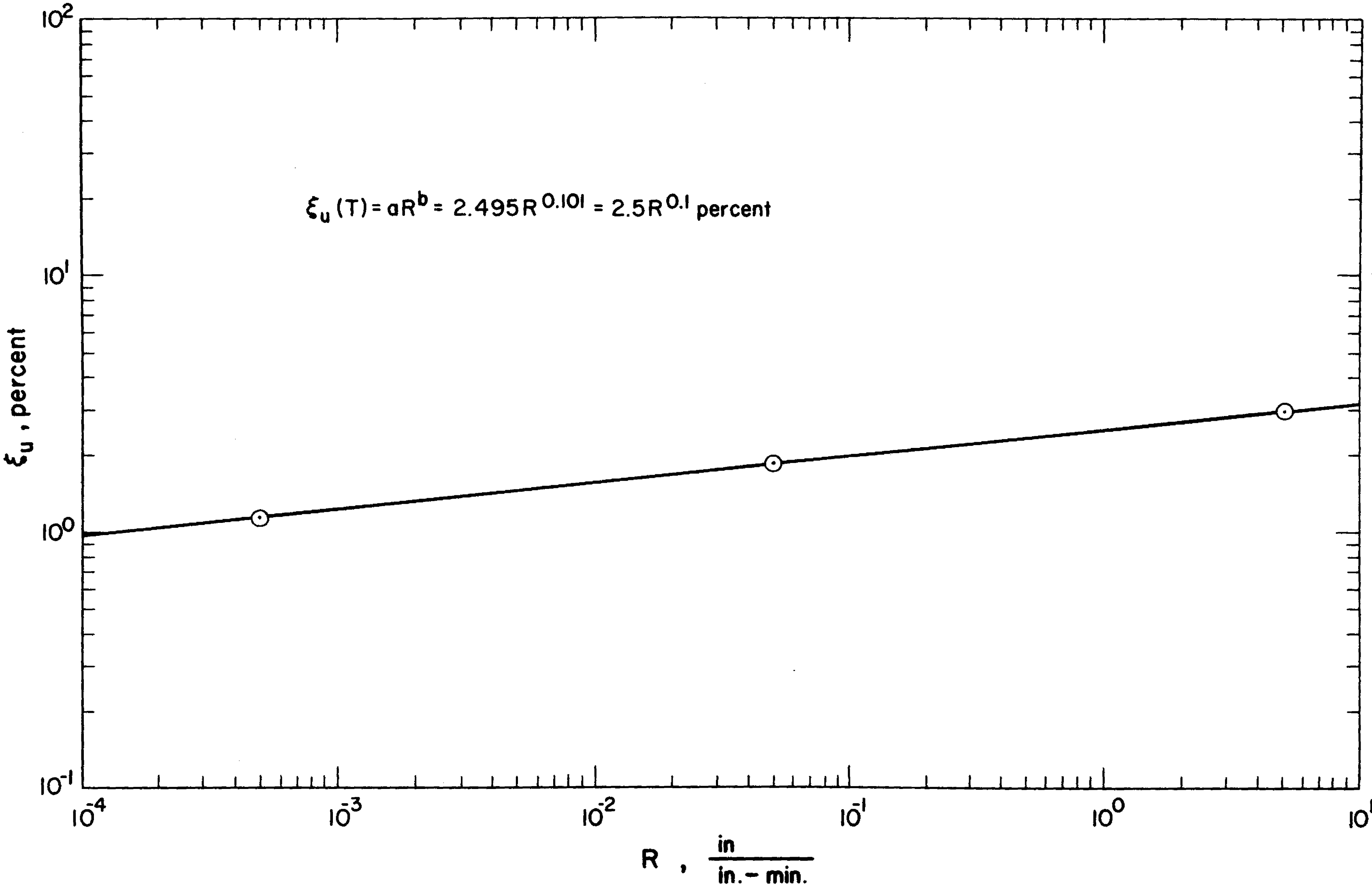

Fig. E5. Ultimate Strain vs Strain Rate.

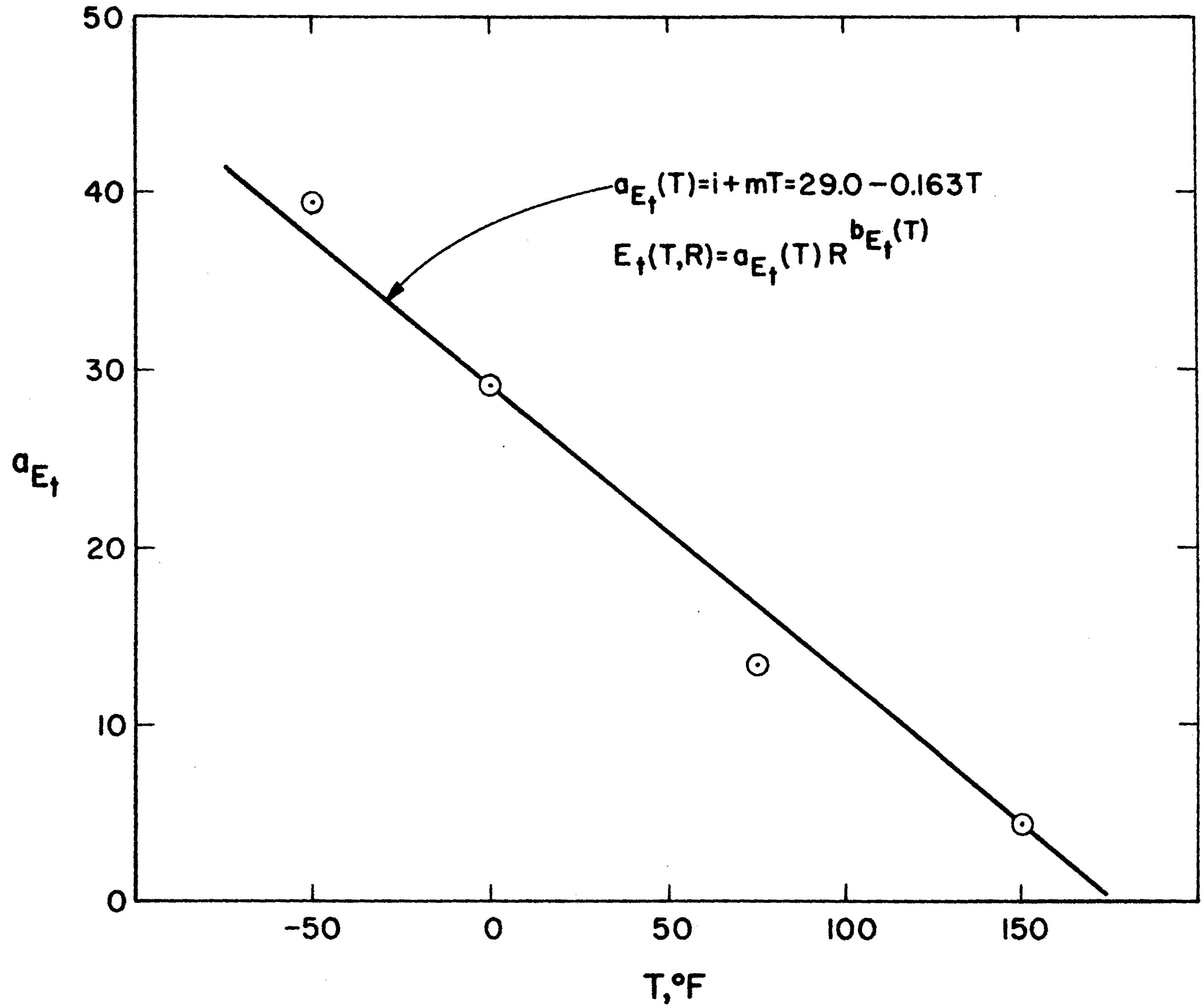

Fig. E6. Variation of a_{E_t} with Temperature.

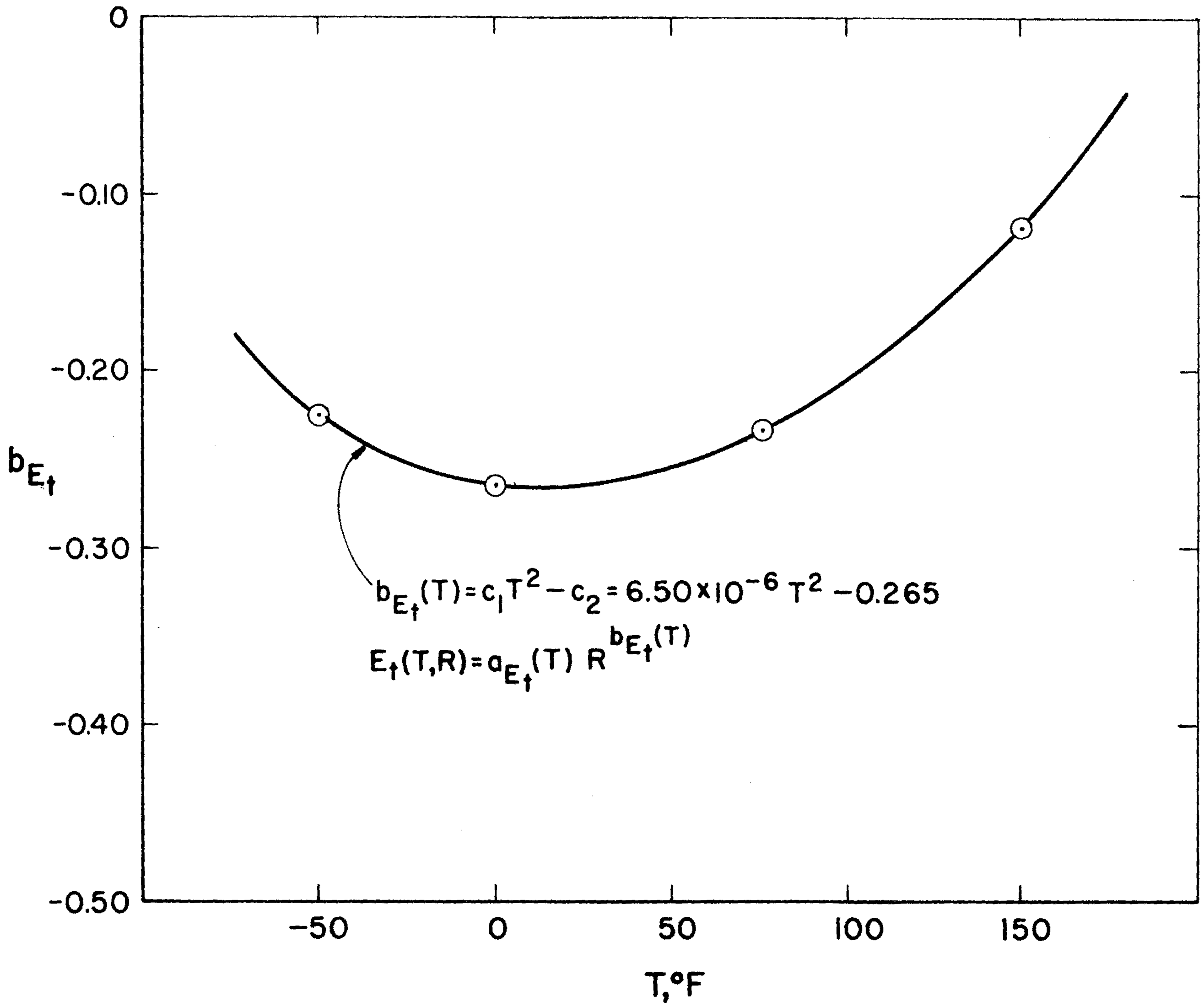

Fig. E7. Variation of b_{E_t} with Temperature.