

THE CREEP OF TRIAXIALLY STRESSED CONCRETE
AT HIGH TEMPERATURE

Fouad Dibsi, B.Sc. (C.E), DIC, M.Sc.

Thesis submitted for the Degree of Doctor of
Philosophy in the Faculty of Engineering of the
University of London

Imperial College of Science and Technology
London



ABSTRACT

The work presented in this thesis is aimed to carry out multiaxial creep tests at temperatures above 100°C for the first time, in order to do so, apparatus has been developed mainly to enable the long term axial and radial deformations of concrete cylinders to be measured when load at high temperature is applied. The apparatus consists of the following;

- 1- pressure cell,
- 2- strain measurement system,
- 3- heating system,
- 4- hydraulic loading system

The cell has been designed to operate over a range of stresses and temperatures which may occur in typical concrete pressure vessels. It is unlike conventional soil mechanics pressure cells which have been used widely at normal temperatures as well as temperatures up to 100°C . The heating system incorporates safety devices which are designed to maintain the specimen at a constant temperature in the presence of lateral pressure to close limits, in order to avoid toxic vapors or flammability hazards.

Two series of test were carried out: firstly, triaxial thermal creep tests under steady state temperatures and stresses, during a period of ten days, secondly, thermal shrinkage and expansion tests under similar conditions to the thermal creep tests, age, moisture content, size, rate of heating and temperatures but with no stresses.

Test results obtained were incorporated in mathematical equations, which were used in the analysis of a hot spot in a cylindrical pressure vessel under hypothetical conditions and in the absence of a cooling system.

The solution of the problem set out in this thesis is achieved by the development of computer programmes in which the thermal creep strain and the thermoelastic or elastic stresses arising are determined by finite element techniques.

ACKNOWLEDGEMENT

The research work described here was carried out at Imperial College of Science and Technology, London, under the general supervision of professor A.L.L. Baker and professor J.M. Alexander to both of whom the author feels deeply indebted. Their constant concern and invaluable guidance are gratefully acknowledged.

Thanks are due to Dr. D. J. Hannant, of Surrey University, Dr. J. B. Newman, and Dr. P.J.E. Sullivan, of Imperial College, contributed by giving helpful advice and engaging in discussion.

Sincere thanks are due to the member of staff of the Imperial College Computer Centre for their advice, and the members of staff of the Civil, Mechanical Engineering, and Concrete Laboratory for their help in constructing the testing apparatus and particular mention should be made of R. Baxter for his skilled work.

The author would like to take the opportunity to thank his parents and his brother Dr. Faysal for their unfailing encouragement and understanding. The author would be failing in his duty if he did not thank his wife who had taken great pains in typing the endless changing of the manuscript beside her domestic duties and her active encouragement and infinite patience.

Table of Contents

ABSTRACT	ii
ACKNOWLEDGEMENT	iv
Chapter 1	
INTRODUCTION	
1.1 General Remarks	1
1.2 Scope of the Research	2
Chapter 2	
SURVEY OF LITERATURE RELATING TO CREEP, MOISTURE AND SHRINKAGE AT ELEVATED TEMPERATURE	
2.1 Introduction	6
2.2 Creep	6
2.2.1 The Principal Theories of Concrete Creep	8
2.2.2 Creep Curves, Specific Creep and Creep Coefficient	12
2.2.3 The Effects of Temperature on Creep	13
2.2.4 Other Factors Affecting Creep	23
2.2.5 Creep Recovery	26
2.2.6 Poisson's Ratio	26
2.3 Moisture, Shrinkage and Thermal Expansion	27

2.3.1	Classification of Water Present in Concrete	27
2.3.2	Effect of Moisture on Concrete Properties at Room Temperature	30
2.3.3	Effect of Moisture on Concrete Properties at Elevated Temperature	32
2.3.4	Shrinkage	37
2.3.5	Theories of Shrinkage	38
2.3.6	Moisture Movement and Shrinkage at Elevated Temperature in PCPV	39
2.3.7	Thermal Expansion of Concrete	43
2.4	The Separation of the Strain Components	47
2.5	Concluding Remarks	48

Chapter 3

APPARATUS FOR LONG TERM MULTIAXIAL TESTING OF CONCRETE AT ELEVATED TEMPERATURES

3.1	General Remarks	62
3.2	Load Maintenance Technique	63
3.2.1	Low-Stiffness Loading Frame	64
3.2.2	Hydraulic Loading	64
3.2.3	Spring Loading	65
3.2.4	Gas-Operated Loading	65
3.2.5	Regulated Hydraulic Loading	66
3.3	Factors Affecting Materials Tests	67
3.3.1	Pinned and Fixed Loading Conditions	68
3.3.2	Operation of Spherical Seating	69
3.3.3	Longitudinal Stiffness of Testing Apparatus	70
3.3.4	Stability of Testing Apparatus	71
3.3.5	Rate of Loading	72

2.3.6	Effect of Apparatus Platens, Packing Materials and Specimen Shape on The State of Stress in Specimens	73
3.4	The Proposed Apparatus for Long Term Multi-axial States of Stress	77
3.4.1	Longitudinal Compressive Loading System	81
3.4.2	Radial Loading System	83
3.4.3	Strain Measurement	87
3.4.3.1	The Axial Strain Measuring System	93
3.4.3.2	The Radial Measuring System	93
3.4.4	Pressure Circuit	97
3.4.5	Thermal System	98
3.5	Operation of Pressure Cell	99
3.5.1	Safety	102
3.5.2	Significant Correction to Be Made to Strain Recordings	102
3.5.3	Less Significant Corrections (Normally Negligible)	103

Chapter 4

INSTRUMENTATION

4.1	Introduction	126
4.2	Measurement of Temperature	126
4.3	The Measurements of the Deformations	128
4.3.1	Vertical Displacement	128
4.3.2	Radial Displacement	130
4.4	Load Measurement	130
4.5	Measurement of the Thermal Expansion	132
4.6	U.V. SE 2100 Electronic Recorder	133
4.7	Galvanometers	134

4.8	Power Packs	134
-----	-------------	-----

Chapter 5

TEST SPECIMENS

5.1	General Remarks	141
5.2	The Concrete Mix	141
5.3	Mixing Procedure	142
5.4	Casting of Specimens	143
5.5	Curing of the Specimens	144
5.6	Surface Preparation	144

Chapter 6

DESCRIPTION OF TESTS

6.1	General Remarks	147
6.2	Preliminary Tests	147
6.3	Calibration Tests	148
6.3.1	Calibration of the Vertical Transducer	148
6.3.2	Calibration of Horizontal Transducers	149
6.3.3	Calibration of Load Cell	149
6.3.4	Thermocouples Calibration	150
6.4	The Main Tests	151
6.4.1	Triaxial Thermal Creep Tests	151
6.4.2	The Residual Strength	152
6.4.3	Thermal Shrinkage and Expansion	153

Chapter 7

TEST RESULTS AND DISCUSSION

7.1 Series 1	162
7.2 Series 2	166
7.3 Concluding Remarks	168

Chapter 8

FINITE ELEMENT ANALYSIS OF THERMAL CREEP

8.1 Introduction	182
8.2 Transient Heat Conduction	183
8.3 Thermal Creep of Concrete	187
8.4 Solution Procedure	189
8.5 Prestressed Concrete Reactor Pressure Vessel	190
8.6 Recommendation for Future Research	192

Appendices

APPENDIX 1

A1.1 Analysis of the Cylindrical Jacket as a Thin Shell	208
---	-----

APPENDIX 2

A2.1 The Finite-Element Procedure	216
A2.2 Development of the Governing Equations	218

APPENDIX 3

A3.1 Program Thermo	225
A3.2 Program Vessel Flow Chart	234

TABLES AND FIGURES OF THE MAIN TEXT

Figure 2.1a	Typical Deformation V.S. Time Curve for Concrete under Constant Load (from Ref. 93)	51
Figure 2.1b	Effect of Moisture Condition of Storage on Creep (from Ref. 93)	51
Figure 2.2	Influence of Temperature on Creep Sealed or Water-Stored Specimens after 60, 80, 90, or 100 Days of Loading (from Ref. 43)	52
Figure 2.3	Influence of Temperature on Creep Unsealed Specimens after 60, 100 or 107 Days of Loading (from Ref. 43)	52
Figure 2.4	Influence of Temperature on Creep Rate $f/f_c \leq 0.50$ (from Ref. 43)	53
Figure 2.5	High Temperature Creep Normalized to a stress Level of 0.2 from ref. 39	54
Figure 2.6	Creep of Concrete under Transient Temperature Conditions (from Ref. 40)	55
Figure 2.7	Creep of Concrete under Transient Temperature Conditions (from Ref. 40)	55
Figure 2.8	Relation between Creep and Time under Load (from Ref. 41)	56
Figure 2.9	Relation between Creep and Time under Load (from Ref. 41)	56
Figure 2.10	Effect of Curing on Adsorption Isotherms of Neat Cement Paste (from Ref. 56)	57

Figure 2.11	Phase Diagrams for Water in Concrete at 887 Days in Specimens of 5 ft. and a Hot Face Temperature of 125°C from England and Ross (73).	58
Figure 2.12	Water Content, Pore Pressure, Transverse and Longitudinal Shrinkage Distributions for the Shrinkage Specimen after 100 and 495 Days of Heating (from Ref. 81).	58
Figure 2.13	Correction Factor for Moisture and for Age (from Ref. 89).	59
Figure 3.1	Triaxial Creep Rig for Temperature up to 65°C (from Ref. 2).	
Figure 3.2	Pressure Cell for Temperature up to 95°C (from Ref. 1).	105
Figure 3.3	Uniaxial Creep Rig Relies on Flexible Diaphragm Jack (from Ref. 103).	105
Figure 3.4	A Possibility for a Low-Stiffness Creep Frame	106
Figure 3.5	Regulated Pressure Supply	106
Figure 3.6	General Plan of the Loading Rig, Pressure Cell and Concrete Specimen.	107
Figure 3.7	General View of Creep Apparatus	108
Figure 3.8	Loading Frame and Pressure Cell	109
Figure 3.9	General Elevation of the Loading Rig, Pressure Cell, Concrete Specimen	110
Figure 3.10	Pressure Cell	111

Figure 3.11	Pressure Cell, Axial Measuring System and Insulation	112
Figure 3.12	Pressure Cell and Radial Measuring Device	113
Figure 3.13	Cooling Jacket	114
Figure 3.14	Details of Pressure Cell Ends	115
Figure 3.15	Pressure Cell Support at the Bottom End	116
Figure 3.16	P.T.F.E. Seals Probe Rod	117
Figure 3.17	Arrangement of Radial Measuring System	118
Figure 3.18	Radial Measuring Device	119
Figure 3.19a	The Radial Measuring Device	120
Figure 3.19b	Spring Retaining Cap	121
Figure 3.19c	Bellows Retainer	122
Figure 3.19d	Lock Nut	123
Figure 3.19e	Bellows Lock Nut	123
Figure 3.20	Hydraulic Loading System	124
Figure 3.21	Electrical Connections (Heating System)	125
Figure 4.1	Typical Electrical Connections for D.C. Transducer.	136
Figure 4.2	Flat-Jack Calibration Curves (Type 42)	137
Figure 4.3	Electrical Connection of Load Cell	138
Figure 4.4	Ether Furnace & Shrinkage Specimen	139
Figure 4.5	Control and Monitoring Units.	140
Figure 5.1	Arrangement of Fitting Discs	146

Figure 6.1	Specimen Temperature	156
Figure 6.2	Vertical Transducer Calibration	157
Figure 6.3	Horizontal Transducers Calibration	158
Figure 6.4	Load Cell Calibration Curve	159
Figure 6.5	Thermocouple Calibration Curve	160
Figure 6.6	Loading Path	161
Figure 6.7	Pressurising Procedure	161
Figure 7.1	Elastic and Creep Strains for Ten Days Period	171
Figure 7.2	Elastic and Creep Strains for Ten Days Period	172
Figure 7.3a	Modulus of Elasticity	173
Figure 7.3b	Elastic Poisson's Ratio	174
Figure 7.4	Creep Poisson's Ratio	175
Figure 7.5	Creep Poisson's Ratio	176
Figure 7.6	Uni-Axial Residual Strength	177
Figure 7.7	Thermal Specific Creep Strain	178
Figure 7.8	Thermal Shrinkage for Ten Days Period	179
Figure 7.9	Analytical & Experimental Results Comparison	180
Figure 7.10	Samples of Tested Specimens	181
Figure 8.1	Variable Temperature Conditions	195
Figure 8.2	Prestressing and Gas Pressure Idealization	195
Figure 8.3	Finite Element Mesh for Axisymmetric Analysis	196

Figure 8.4	Topological Mesh for Axisymmetric Analysis Using TRIAX 6 ELEMENTS	197
Figure 8.5	Temperature Distribution in Hot Spot of PCPV	198
Figure 8.6	Temperature Distribution	199
Figure 8.7	Contours of Radial Stresses at High Tempe- rature	200
Figure 8.8	Contours of Vertical Stresses at High Temperature	201
Figure 8.9	Contours of Tangential Stresses at High Temperature	202
Figure 8.10	Contours of Shear Stresses at High Temperature	203
Figure 8.11	Contours of Radial Stresses at Low Tempe- rature	204
Figure 8.12	Contours of Vertical Stresses at Low Temperature	205
Figure 8.13	Contours of Tangential Stresses at Low Temperature	206
Figure 8.14	Contours of Shear Stresses at Low Tempe- rature	207
Figure A1.1	Initial Buckling Process	214
Figure A1.2	Stress-Strain Diagram	214
Figure A1.3	Change of Steel Stress at Point of Convol- ution	215
Figure A2.1	Axisymmetric Triangular Element	224

Table 2.1	Specimen Failure (from Ref. 41)	53
Table 2.2	Rough Survey of the Properties of Concrete Which Are Influenced by the Various States of Water Present in the Concrete (from Ref. 58)	60
Table 2.3	Average Coefficients of Linear Thermal Expansion of Rocks for Normal Temperature Ranges (from Ref. 89)	61
Table 3.1	General Multiaxial State of Stress Conditions	79
Table 3.2	Cylindrical State of Stress Conditions	80
Table 3.3	Apparatus State of Stress Conditions	80
Table 5.1	Mix Proportions	142
Table 6.1	Test for Creep Strain under Multiaxial Stress	155
Table 6.2	Test for Thermal Shrinkage of Concrete	155
Table 7.1	Secant Modulus of Elasticity and Poisson's Ratio	169
Table 7.2		169
Table 7.3	Thermal Expansion on the First Day	170
Table 7.4	Transient Thermal Shrinkage	170
Table 7.5	Thermal Expansion after 10 Days	170

Table A1.1

213

Table A1.2

213

Chapter 1

INTRODUCTION

1.1 General Remarks

The adoption of the prestressed concrete pressure vessel design for nuclear reactors has permitted an integral design, with the entire gas circuit, including boilers, contained within a single pressure envelope. Recently attention has been refocussed on the development of nuclear power, with special reference to the efficiency of the present system in use, and where cost economies can be applied. System efficiency can be increased by raising the working temperatures and pressures inside the reactor. Cost economies can be affected by a reduction in the volume of the costly cooling and insulation systems. Both these actions would increase the maximum temperature of the concrete, which, in the present system, is generally in the 60°C to 80°C region. These actions would also increase creep and shrinkage.

In line with the improvements in design philosophy advances have been in the application of computer-based methods for the analysis of complex structures. If these computer-based techniques are to be used to analyse realistically the response of a concrete pressure vessel to the imposed temperature and loading conditions, information is required of the relevant material properties and the way in which these change through the range of stresses and temperature states likely to be encountered.

Unfortunately data on the properties of concrete under various loading and temperature regimes, necessary for such an approach, are lacking.

Creep at elevated temperature has been studied extensively and a great deal of information is now available in this field. Nevertheless, some problems still remain unresolved, this is due to the fact that the stress-strain behaviour of concrete at elevated temperatures is extremely complex and is not completely understood.

All multiaxial creep tests at elevated temperature (1, 2, 3, 4) were generally in the region below 100°C , whereas creep tests above 100°C were carried out uniaxially.

The work presented in this thesis is aimed to extend the previous work (1, 2, 3, 4), beyond 100°C .

1.2 Scope of the Research

In order to carry out triaxial creep tests at temperatures above 100°C , apparatus has been developed mainly to enable the long term axial and radial deformations of concrete cylinders to be measured when load at high temperature is applied.

The apparatus consists of the following;

- 1- pressure cell,
- 2- strain measurement system,
- 3- heating system,
- 4- hydraulic loading system.

The cell has been designed to operate over a range of stresses and temperatures which may occur in typical concrete nuclear pressure vessels. It is unlike conventional soil mechanics pressure cells which have been used widely at normal temperatures as well as temperatures up to 100°C .

The cell has two important and significant features, firstly that pressure is maintained by all-metal containment and so avoids the risk of using organic seals or reinforced seals which, at high temperatures in long term tests, may leak and so reduce pressure. Secondly, the cell is cheap to make and would allow power station laboratories to carry out several long term tests simultaneously at a variety of temperatures and stresses, so that concrete could be monitored.

Since commercially available strain gauges do not perform reliably at high temperatures, a new strain measuring system was developed and subjected to various testing conditions before being constructed within the apparatus.

Provision of the heating system to maintain the specimen at a constant temperature in the presence of lateral pressure posed several mechanical and thermal problems. First, that of maintaining the fluid at 200°C requires careful selection -- as some fluids produce toxic vapors or are flammable at this temperature. Second, change in temperature due to the surrounding environment might fluctuate the pressure and this is eliminated by facilities incorporated in the hydraulic loading system designed for this purpose.

It is clear that the multiaxial test facilities are greatly complicated by the requirement of exposure to 200°C temperature.

Details of the apparatus are fully documented in chapter 3. The preliminary experimental work carried out to check the performance of the apparatus under various state of stresses and temperatures for several months has not been reported in this thesis.

Since the tests were planned to deal with temperatures higher than 100°C , it seemed neither useful nor necessary in the preliminary programme to seal the concrete specimens. To this extent, there is a limit set to the applicability of the test results to conditions in solid reactor walls. It should, however, be pointed out that in the area of the liner and of tube penetrations in reactor vessels temperatures above 100°C must be expected to occur which in turn may result in intensive moisture migration in some areas and, in the long run, in substantial drying out of the concrete.

Two series of test were carried out:

Firstly, triaxial thermal creep tests under steady state temperatures and stresses, during a period of ten days.

Secondly, thermal shrinkage and expansion tests under similar conditions to the thermal creep tests, age, moisture content, size, rate of heating and temperatures but with no stresses.

Test results obtained were incorporated in mathematical equations, which were used in the analysis of a hot spot in a cylindrical pressure vessel under hypothetical conditions and in the absence of a cooling system.

The solution of the problem set out in this thesis is achieved by the development of computer programmes in which the thermal creep strain and the thermoelastic or elastic stresses arising are determined by finite element techniques.

Chapter 2

SURVEY OF LITERATURE RELATING TO CREEP, MOISTURE AND SHRINKAGE AT ELEVATED TEMPERATURE

2.1 Introduction

The general review of the existing literature has been divided into two sections, namely,

- i) Creep of concrete at elevated temperature,
- ii) Moisture and shrinkage at elevated temperature.

Under the first, those works dealing with the more fundamental aspects of creep at elevated temperature up to 450°C for steady state as well as for the transient temperature, and also a comparison of the results have been made which show that the dispersion of these results increases significantly with increasing temperature and differences of more than 100% can be observed.

Under the second section, a general review is given of the moisture influence on the properties of concrete, shrinkage and the effects of moisture migration in thick-wall sections. The general review is particularly important in regard to nuclear pressure vessel construction.

2.2 Creep

Creep in concrete reveals itself as a dimensional change in the material under the influence of sustained mechanical

loading (5), (6), (7), (8), (9), (10). Quite small loads will cause the concrete to deform. Even the self-weight of a concrete member is sufficient to initiate creep.

The phenomenon of creep occurs at elevated as well as at ambient temperatures. Elevated temperatures, in fact, increase the rate of creep.

Creep causes, in a loaded member, a stress redistribution during the transient period before steady state stress conditions are reached after several months. In some circumstances, stress redistribution can be beneficial, for example in the relief of locked-in stress, but in others disadvantageous, as in the relaxation of steel prestress.

Concrete is a material of such variability that elastic and creep data obtained from observations on a concrete structure under particular loadings and circumstances can be considered characteristic only of that concrete and those circumstances.

There is a general inter-relationship between concrete strength and creep (11), (12); the stronger the concrete, the smaller the creep.

Creep strain, by definition, is time-dependent, and the total amount of creep at any time increases at a decreasing rate.

Creep of concrete is more a property of the cement paste

than of the aggregates, although crystalline mineralogical creep deformations contribute to the total dimensional changes involved.

2.2.1 The Principal Theories of Concrete Creep

The first hypothesis to explain concrete creep was due to Freyssinet (13) who published it after his famous observations of the deformations of Plougastel bridge in 1926-1929.

The explanation of creep was similar to that of shrinkage; The depression caused by capillary water inside the voids - which is a function of the curvature radii of the water-vapour separation meniscus (Laplace law) - was assumed to be responsible for strain due to duration of loading. The curvature of the meniscus, which depends on the relative humidity of the environment (Lord Kelvin law), was also assumed to depend on the applied stress, which would change the radii of the voids and therefore also the curvature of the water-vapour separation meniscus. For Freyssinet, creep was thus a shrinkage amplified by the applied stresses.

This theory implies that the loss of water through drying is greater in loaded concrete than in free concrete; it would also imply that concrete submitted to a pure shear would not creep. In practice, however, it has been well demonstrated that none of these facts are true. Loss of water and humidity distribution inside concrete are the same no matter whether the concrete specimen is loaded or not.

Hydrated cement consists of crystallized substances,

most of which are hydrated silicates, composed of stacks of superimposed single crystal plates, separated by water molecules (interlayer water). It also has a remarkable surface area which is the source of important phenomena of water molecules adsorption.

It is thus natural to seek the causes of creep in this structure. Lynam (14), in 1934, suggested three possible fundamental causes for creep: crystalline sliding, movement of the adsorbed water in the cement hydration products, and viscous flow. All the causes suggested to this day can be included in these three. Recently investigations by Coutinho (20), suggested that there is another cause which is not included in this group: a cause of a chemical nature, which consists in the hydration of the cement particle nuclei due to the applied load.

In the first place one can think that the crystals of the hydrated components of the cement themselves undergo deformation due to creep, through a mechanism similar to that of metal crystals: strain as a function of time would be due to a process of diffusion of the dislocation.

This process can be applied to the hydrated cement crystals, to the solid and epitaxial bonds between them and the aggregate, to the crystals of unhydrated cement, and also to the crystals of the aggregate.

Within this movement of crystalline deformation another movement can be suggested, due to a shear process taking place

where multi-face crystal surfaces slide one over the other (15).

But the contribution of these causes must be rather small as the movement or dislocations, which is important between atoms with metallic bonds, is probably difficult with ionic and covalent bonds.

Another cause could be the movement of adsorbed water on the surface of the porous crystalline material which constitutes hydrated cement: under the action of a load the adsorbed water would be expelled, with difficulty caused by friction inside the pores. T.C. Powers (16), in 1966, developed this idea.

In the wedge-shaped voids inside hydrated cement paste, in the vicinity of the tip, the gap is narrower than the thickness of the water adsorbed film. This region is then an area of hindered adsorption. Powers supposes that in these places, adsorbed water has a load-bearing capacity. If a load is applied to a body with micropores partially filled with water, some load-bearing water will be squeezed out to restore equilibrium. The expulsion, from the narrower places between particle-areas of high loading- of the thin films of adsorbed water, takes place through diffusion and results in creep strain. Powers also states that creep and shrinkage are merely two branches of the same process.

For Sereda and Feldmann (17), creep, just as shrinkage,

is due to the exit of the interlayer water found between the plates of the crystals of cement hydration products. The entrance and exit of interlayer water depends on the relative humidity of the environment and on the applied load. These causes make the interplate distance vary, thus leading to variations in the concrete volume. As the molecules are caught between the plates and the remaining crystals, their exit is rendered very difficult, which accounts for the slow nature of the strain.

In the third mechanism suggested by Lynam, hardened cement is considered to be a definitely viscous liquid, whose viscosity increases with time. Viscous deformation is due to a sliding or very slow movement of the particles over one another when the van der Waals forces acting on their surfaces are small. This mechanism could explain the irreversible part of creep if it were not in contradiction with what is known about the crystalline structure of hydrated cement: grains are not bonded to one another by van der Waals forces, but they are bonded by the solid bonds of ionic or covalent nature.

The deformation process may certainly be applied to particles foreign to the hydrated cement structure, with non-solid bonds to this structure.

Nevertheless, Hansen (18) suggests that the viscous flow in hydrated cement paste could take place in the grain boundaries, that is to say, in their "welding points". Neville (19), supporting this idea but expressing some doubt whether

solid bonds, like chemical or crystalline bonds, can behave in a viscous manner, says that this can be the case in grain boundaries in poly-crystalline metals when the boundaries represent a compromise between the crystalline arrangements in two adjacent grains. Thus hydrated cement, a polycrystalline porous solid, would not be unique in showing viscous flow in grain or particle boundaries. This would be a process involving bond breaking and reforming (15).

2.2.2 Creep Curves, Specific Creep and Creep Coefficient

Representative curves of creep strain plotted against time are shown in fig. (2.1a). A concrete specimen subjected to uniaxial compression manifests an immediate elastic deformation. If the load is sustained, additional strains due to creep occur; the rate of which decreases with time in the manner shown. On removing the load there is an immediate elastic recovery followed by a recovery of creep strain the rate of which rapidly decreases with time. A residual strain is left greater in magnitude than the initial elastic strain. It must be realized that during the period of such a test, other important concrete constants will have been changing. Young's Modulus for instance will have been increasing as the concrete strength increases with time and thus the material will show increased resistance to effects such as creep and load changes, towards the end of the test.

At relatively low levels of stress, creep is roughly proportional to stress and it has been found convenient to introduce the concept of specific creep or creep strain per unit stress.

This is a useful comparative indicator of creep effects, since references to the magnitude of creep strain, without an indication of the stress causing it, are without much value.

In addition, the term creep coefficient is used to denote the ratio of the ultimate long-term creep strain to the instantaneous elastic strain under a given load.

2.2.3 The Effects of Temperature on Creep

Perhaps the first indication that creep in concrete can drastically be affected by temperature was given by Theuer in 1937. (21). Additional evidence of a strong temperature dependency of creep was presented by Ruetz (22) in 1958. Describing tests on hermetically sealed, pure, cement-paste specimens, he reported that "the modulus of elasticity decreased as the moisture content decreased" and "the ability of the paste to creep and its inelastic deformations increased with increasing temperature". Within a temperature range of 50°C , the creep of the paste was found to increase as much as 10 to 20 times its value at the lowest test temperature.

Serafim and Querreiro (23) conducted tests on "mass Concrete" specimens at room temperature and at 45°C . They found that the creep of mass cured (i.e. sealed) specimens at 45°C , which were loaded at 3 and 8 days age, increased only moderately (about 10 percent after 100 days under load) compared with that of the specimens tested at "room temperature". Practically all of the increase occurred within the first few days under load, after 4 to 7 days under load, the creep rate became the same

for both temperatures.

In 1960, Hansen (24) reported the results of flexural tests on concrete beams at temperatures between 15°C and $+60^{\circ}\text{C}$. All specimens were cured under water at 35°C for 6 months before being loaded in a sealed condition. The results indicated that creep of specimens tested at 40°C and 60°C was approximately double and almost triple, respectively, the creep of specimens tested at 20°C . The reduction in modulus of elasticity with an increase in temperature was also reported.

Two years later, England and Ross (25) presented results of tests on sealed and unsealed concrete cylinders at temperatures up to 140°C for loading periods up to 80 days. Results of tests on sealed cylinders for the 80-day loading period showed that the creep at 80°C and 140°C was about 3.5 and 4.2 times, respectively, as great as that at 20°C . Variability in the results of the tests on the unsealed specimens make interpretation difficult; however, it appears that England and Ross's data indicate a creep maximum at approximately 100°C for both sealed and unsealed specimens (see figures 2.2, 2.3 and 2.4).

A novel, and rather startling, observation was first reported by Nasser and Neville (26) in 1965. Studying the creep behaviour of mass and water-cured 6000-psi concrete that was cured at test temperature 24 hr after casting within a temperature range from 21.1 to 96.1°C at stress/strength ratio from

0.35 to 0.70, they found a pronounced maximum for the creep rate at a temperature of about 71°C (figure 2.4). This creep rate was based on creep measurements made during a period from 21 to 91 days after loading, but it appeared to remain essentially constant up to 15-month loading period for which measurements were reported.

However, the creep rate of several specimens for up to 21 days of loading was substantially different from the creep rate after 21 days of loading; hence, the creep values after 90 days of loading did not consistently show a maximum in the vicinity of 71°C (figure 2.2). In fact, one test series indicated a steady increase of 90-day creep with increasing temperature, in spite of a maximum creep rate at about 71°C for the 21 to 91-day loading period (Figures 2.2 and 2.3)

Nasser and Neville found the other aspects of the patterns of creep behaviour, specifically the creep-time curve and creep recovery, to be the same at elevated temperature as at normal temperatures. They also found creep to be proportional to the stress/strength ratio at any temperature within the range tested. A later extension of the program to old concretes (1 to 50 years old), reported by the same investigators in 1967, (27) gave essentially identical results.

The occurrence of the "creep maximum" in the range of 50 to 70°C has attracted much attention, and some additional evidence pointing to the occurrence of such a maximum has been reported.

Peschel (28) has reported experiments in which the viscosity of thin water films (about 80 Å) between quartz glass plates was measured as a function of temperature. A number of pronounced maxima were recorded among them, one at 62.5°C with a corresponding minimum around 50°C . It can be speculated that there is a relation between these extremes and the phenomenon of a creep maximum observed between 50 and 70°C . An explanation based on the influence of the temperature upon the properties of load-bearing water, which is similar to the hypothesis outlined by Nasser and Neville (26) and Marechal (29, 30), has been suggested by Powers. Marechal has also observed a pronounced creep rate maximum (and a corresponding maximum for total creep) at a temperature substantially below 80°C . Performing creep tests at temperatures up to 400°C on unsealed specimens of siliceous and siliceous-calcareous aggregate concretes loaded after 1 year of moist-curing at 20°C and 15 days of air-curing at a respective test temperature, he found, at the end of a 100-day loading period, a definite maximum for both creep rate (1 to 100 days) and creep magnitude at 50°C (Figures 2.3 and 2.4). Predrying the specimen for 30 days at 105°C before application of the load reduced creep at temperatures below 105°C drastically and eliminated the creep maximum (Figures 2.3 and 2.4), a result consistent with Theuer's earlier findings (21).

Several other investigators who have conducted creep tests at elevated temperatures in recent years have not observed a maximum for total creep, at least not at comparably low temperatures. However, most results indicate a maximum for the rate of creep in the temperature range of 50 to 100°C (i.e., if the

creep rate is computed for some period between 1 and 100 days under load).

Hannant (1), for instance, conducted creep tests on sealed 4-1/8 by 12 in. cylinders of an approximately 9000 psi, limestone aggregate concrete after curing them 5 months in water and an additional month in a sealed, saturated condition. The results showed a nearly linear increase of specific creep with temperature within a range of 27 to 77°C for all loading periods up to 2 years, the creep at 77°C being approximately 4 to 4.8 times that at 27°C. A still higher rate of increase was observed in the 77 to 93°C temperature range (Figure 2.2). Up to stresses of 2000 psi, creep remained proportional to stress. The values of creep strains for concrete that had been dried before loading were again only small fractions of the values for "wet" concrete. In triaxial tests, stresses at right angles to the principal stress were found to reduce creep in the direction of the principal stress; however, considerable creep remained even under a hydro-static state of stress. Poisson's ratios for creep determined on sealed specimens were similar in magnitude to the elastic value. Temperatures also had a considerable influence on the modulus of elasticity, as reported by other investigators, (24, 31, 33) with Young's modulus decreasing almost linearly with increasing temperature for short heating periods; however, if the specimens were heated for an extended period (such as 2 years), the modulus recovered to values approaching those at normal temperatures.

Hickey (33), performed tests on unsealed 6 by 16 in.

cylinders of a 7000-psi, amphibole-schist aggregate concrete loaded to 800-psi uniaxial compression after 1 month of moist-curing and an additional month of curing at 50 percent relative humidity (RH). Subsequent to the application of loads, the specimens were heated to 54.4, 82.2, 100, and 143.3°C. After 6 months under load, creep appeared to be a fairly linear function of temperature up to 82.2°C, but increased nonlinearly with temperature between 82.2 and 143.3°C, indicating a maximum around 143°C. At 143°C, creep was five times as great as the creep measured on companion specimens kept at room temperature. Six-month creep values at 54.4, 82.2, and 100°C were, respectively, 2-2/3, 4, and 4-2/3 times the creep values at 22.8°C. Initial creep rates (up to 1 or 2 days) of the unsealed specimen increased dramatically with temperature; however, after the first few days, the creep rates at the two highest temperatures (100 and 143.3°C) decreased substantially. At these temperatures, about 90 percent of the 6-month creep occurred^r in the first month, apparently due to the accelerated loss of water. When creep rates were computed for a period between 1 and 10⁷ (or 1 and 180) days after loading, they showed a distinct maximum at approximately 90°C. Figure (2.3).

An almost linear increase of creep with temperature in the 20 to 75°C range was also observed by Arthanari and Yu (34) on sealed and unsealed 12-by 12-by 4-in. slabs of 6000-psi, Thames River gravel aggregate concrete loaded to 1000-psi, both uniaxially and biaxially, at an age of 10 to 100 days. At the end of a 40-day loading period, for which results were

given, creep at 80°C was approximately twice the creep at 20°C in sealed specimens and three times the creep at 20°C in unsealed specimens. The influence of age at loading upon the rate and magnitude of creep appeared to be less at higher temperatures than at room temperature, and an incremental increase of temperature to a maximum tended to result in higher creep values than a continuous exposure to the same maximum temperature. As in Nasser and Neville's earlier investigations (26,27) creep recovery and Poisson's ratio in creep were found to be essentially the same at elevated temperatures as at room temperature.

Da Silveira and Florentino (35) have challenged the hypothesis that creep recovery is independent of temperature. Reporting on an extension of earlier work (28) on creep of mass concrete between 20 and 45°C , they concluded that creep recovery as well as creep itself was higher at 45°C than at 20°C , and they also confirmed that the Poisson's ratio in creep (in sealed or water-cured specimens) was approximately equal to the elastic value. McHenry's (36) expression, rather than a logarithmic representation, appeared most suitable to fit Da Silveira and Florentino's experimental data.

Browne and Blundell (37) have described results on limestone and dolerite aggregate concrete loaded to 2110 psi at temperatures between 20 and 95°C and ages between 7 and 400 days. Although initial data indicated that creep could be regarded as linear with respect to logarithmic time from loading, a definite upward deviation from the straight line was consistently observed at longer periods under load. Plotting the results on

a log/log basis showed that for the sealed specimen, the experimental creep curves remained linear up to 6 years. Browne and Blundell demonstrated the improved linearity of creep curves in a $\log \epsilon$ versus $\log t$ or $\log \epsilon$ versus $\log (t + 1)$ presentation on the basis of the work of England and Ross (25), Hannant (1), Nasser and Neville (26), and Marechal (29). The creep curves were finally expressed by $\epsilon = a (t)^n$ or $\log \epsilon = \log a + n \log (t)$ where

ϵ = specific creep strain,

a = a factor decreasing with age at loading, k , and increasing with absolute temperature,

t = time under load in days,

n = a factor decreasing with age at loading, k , and varying with absolute temperature. θ

Concerning the slope of the straight lines in a log/log presentation, Browne and Blundell suggested hypothetically that n can be expressed in terms of a modified Arrhenius activation energy equation, i.e.

$$n = C \cdot \sigma \cdot e^{\frac{-E}{R\theta}}$$

Where

C = a constant

σ = stress

E = activation energy

R = Boltzman constant

A similar approach has been described by Marechal (29), and Sullivan (38). Although many of the data obtained or reviewed by Browne and Blundell (37) indicated a decrease of n with a decrease in $\frac{1}{\theta}$, or an increase of the creep rate with increasing temperature, the inconsistency of the results did not allow a conclusive answer as to how temperature in general affects the creep rate. Browne and Blundell's own results, however, did consistently show an increase of total creep with temperature up to 95°C .

Gross (39) has conducted creep tests on unsealed 60 mm. by 180 mm. cylinders of an approximately 42 N/mm^2 , Thames River gravel aggregate concrete loaded uniaxially to several stress/cold strength ratios 0.2, 0.4 and 0.66 at ages between six and twelve months. All specimens were preheated to the desired temperature; then loaded for seven days. He concluded that for conditions milder than stress/cold strength ratio equal to 0.66 and temperature 300°C , the total creep strains are linear in the stress variable, within these limits, also he reduced the non-linear thermal creep phenomenon into a linear thermovisco-elasticity problem. See figure 2.5.

Shneider and Kordina (40) recently carried out creep tests on unsealed 80 mm. by 300 mm. cylinders of an approximately 47.5 N/mm^2 , Quartz aggregate concrete loaded uniaxially to stress/strength ratios 0.15 and 0.30. The temperatures were 135°C , 300°C and 450°C . The concrete specimens subjected to uniaxial pressure at steady state and transient temperatures

which are intended to provide information about the additional creep deformation to be expected in the transient case. At the beginning of the tests the specimens were subjected to load and were then heated up to the selected temperature at 2°C per minute. The temperature was kept constant for about 120 hours, which was followed by a cooling down period of 48 hours. Total deformation of concrete under these test conditions may be envisaged as consisting of the following:

$$\epsilon_{\text{ges}, u} = \epsilon_{e, 20^{\circ}\text{C}} + \epsilon_{\text{tr}, u} + \epsilon_{\text{st}, u}$$

Where

- $\epsilon_{e, 20^{\circ}\text{C}}$ is the elastic deformation at 20°C ,
- $\epsilon_{\text{tr}, u}$ is the transient during the heating up phase,
- $\epsilon_{\text{st}, u}$ is the thermal creep at steady state.

Their results at lower temperature were higher than those reported by Gross (39) as shown in figures (2.6, 2.7). They also concluded that the thermal creep values at temperatures 135°C and 300°C are almost the same, they attributed that to subsequent diffusion processes at 135°C .

Wang (41) has performed tests on unsealed 3 by 6 in. cylinders of a 5500 psi and 6700 psi compressive strength, crushed quartz aggregate concrete. The specimens were first loaded to the desired testing values for a period of one day, and then heated to the expected temperature at an average rate of 1.7°C per hour. Temperature range was 200 F to 800 F. Strain measurement being made after the measuring apparatus as well as the whole system reached steady state. On the other hand, no recording of the results was made during the transient period. It is interesting

to note that compressive strength of concrete at elevated temperature is much higher than that predicted by others (34, 90, 91, 92), these values are listed in table 2.1. He also concluded that creep rate is higher when concrete is subjected to a high temperature and high stress strength ratio, and the latter affects creep more than the former. See figures (2.8, 2.9).

2.2.4 Other Factors Affecting Creep

Creep is also affected by the following additional factors (44),

- a) The water-cement ratio (45),
- b) The conditions of curing,
- c) The ambient relative humidity,
- d) The maturity of the concrete at the time of loading,
- e) The surface-volume ratio of the concrete member (46, 47),
- f) The type and fineness of the cement (47, 48),
- g) The nature of the aggregate,
- h) The aggregate grading,
- i) The aggregate-cement ratio.

(a), (b), (c) The Effect of Water

Since both the seepage and viscous flow theories of creep consider the effect of water to be paramount, it is reasonable to expect that the water-cement ratio (45), the conditions of curing, and the ambient humidity will have an influence on the rate and degree of creep. See figure 2.1b Creep varies as the W/C ratio.

Colloidal seepage is governed by water content, in addition, the degree of hydration, influenced by the amount of water present or available, directly affects the strength of the concrete and its creep. The greater the degree of hydration and hardening, the less creep will be permitted. Furthermore, free moisture adjacent to or within the concrete will result in swollen concrete more susceptible to creep, for example, than dry-cured concrete.

d) Concrete Maturity

The strength of concrete is a function of the degree of hydration and so is proportional to age. Creep can be said to vary roughly inversely as the strength of concrete and the ultimate total creep decreases with increase in hydration.

Thus the total creep and the rate of creep reduce with an increase of the age at first loading (49, 10).

Creep effects can be reduced in concrete framework structures by a) delaying the time of striking of the supporting formwork and b) by delaying, as long as practicable, the application of live loading.

At great ages, the creep rate is small and is practically independent of age of loading.

e) Surface-Volume Ratio and the Effect of Size

Ross found that creep is a function of the surface-volume ratio (50). It is certainly true that as the length of capillary

channel increases, the frictional resistance to seepage increases.

Tests by Hansen and Mattock (51) confirm that creep decreases with increase in specimen dimensions but expulsion of water from the outer gels is unaffected. For very large members the creep approaches the values obtained from similar sealed specimens.

f) The Effect of the Cement

David has shown that creep is proportional both to the cement content and to the W/C ratio (47).

Creep and shrinkage are, in fact, properties of the cement paste rather than of the aggregate. Since finer cements hydrate more quickly than other types, high strength are obtained relatively quickly although ultimate strength may not be greatly affected. It cannot be concluded, however, that the finer the cement the lower the creep rate exhibited. The findings of Bennett and Loat are briefly reported in a summary of factors affecting creep at the end of this section.

Neville (52) has shown that mixes having the same paste content develop the same proportion of their long-term creep after any given time.

Creep increases with cements in the order aluminous, rapid-hardening Portland, ordinary Portland, blast furnace Portland Cement.

(g), (h), (i) The Effect of Aggregates

Aggregates also have an effect on the degree of concrete creep. Efficient grading of aggregates increases the compressive strength of a concrete and thus reduces creep. Crystalline creep in the mineral aggregates varies with the different types, and porosity of the stone has a strong influence on the seepage of water and thus on the creep of a concrete.

Sandstone aggregates tend to exhibit greater mineralogical creep when used in concrete than do limestone aggregates.

The amount of creep decreases with increase in maximum size of aggregate. If the concrete is well-compacted and the stone well-graded, a strong interlocking framework which tends to resist creep deformations well is formed.

2.2.5 Creep Recovery

On unloading a concrete specimen there occurs an immediate elastic unloading of strain which is slowly followed by a recovery of the creep strain (53) extensive at first but reducing after a short period of time. A residual, irrecoverable creep strain will remain.

2.2.6 Poisson's Ratio

The multiaxial behaviour of concrete can be defined in terms of the conventional Poisson's ratio. This may vary considerably under thermal cycling carried over a sustained period. Crispino (54), shows a gradual increase to be greater than 0.5 under cyclic conditions with maximum temperatures of 350°C and 500°C.

In contrast, at constant temperature, Marechal (55) shows a decrease from 0.28 to 0.10 with increasing test temperature from 20°C up to 400°C in pre-dried samples of quartzite concrete. Recently Linse et al (3) have concluded, from multiaxial tests at elevated temperatures 50°C and 80°C, that the poisson's ratio of creep at elevated temperature is larger than at normal temperature at stress strength ratio of 35% and 50%, also, they show increase up to 45. This property appears to depend on moisture conditions and state of principal stresses.

2.3 Moisture, Shrinkage and Thermal Expansion

Like other capillary porous materials, the physical and mechanical properties of concrete are liable to be influenced by water depending on the state in which it occurs. The states in which water can occur in concrete are classified on the basis of its degree of attraction to the solid phase. Powers and Bownyard (56) used this principle to classify the water present in concrete as evaporable and nonevaporable water depending on whether it could be removed under specific conditions of pressure and temperature. Evaporable water is usually present in the capillary and gel pores of the concrete mass and the nonevaporable water occurs as the water of hydration of the cement hydrates.

2.3.1 Classification of Water Present in Concrete

Luikov (57) suggested that the degree of attraction or the form of bond between the moisture and a moist capillary porous material such as concrete be classified on the basis of the bond energy expressed in terms of the heat of evaporation.

He classified the evaporable water described by Powers and Brownyard (56) into two parts according to this principle as:

A) Adsorptionally bound water which is represented by a monomolecular layer on the internal and external surface of a capillary porous material,

B) Capillary bound water contained in the capillaries, except for the very thin layer of water which is adsorptionally bound to the walls of the capillaries.

Nielsen (58) classified Luikov's (57) capillary bound water into two categories:

i) Water which is capillary fixed to the cement gel,

ii) Water which is contained in comparatively large voids in the concrete without being chemically or physically fixed to the solid constituents.

He referred to the last category of water as free water.

Ishai (59) sub-divided the evaporable water into five separate states:

a) Capillary water.

b) Gel pore water,

c) Water adsorbed on crystal surfaces,

d) Water adsorbed and confined between adjacent crystal surfaces,

e) Zeolitic intra-crystalline water.

From the above subdivisions the water present in hardened concrete can be conveniently divided into four major states:

- 1) Chemically bound water or water of hydration,
- 2) Adsorptionally bound water mainly present in the gel pores of the hardened cement paste,
- 3) Capillary fixed water present in comparatively small capillaries,
- 4) Free water present in the larger capillaries.

At any given time, the water present in concrete tends towards a condition of equilibrium between the above four states. This condition of equilibrium depends, among other factors, on the specific properties of the cement, the mix proportions on the environmental conditions. So long as the hydration of the cement paste continues, the condition of equilibrium changes as the water passes gradually from states (3) and (4) into the states (1) and (2).

The formation of the water of hydration in the cement paste is not reversible under normal thermal and hygrometrical conditions i.e at 23°C and under water vapour pressures higher than 6×10^{-4} mm. of mercury, according to Powers and Brownnyard's (56) definition of nonevaporable water. However, the water in the states (2), (3) and (4) generally tend to be in equilibrium with the water in the surroundings of the concrete. The free water contained in the larger capillaries evaporates if the

water vapour pressure in the surroundings drops below the saturation pressure, on the other hand if the vapour pressure of the surrounding is above the saturation pressure of the air within the concrete, it will absorb moisture from the surroundings by an amount depending on the volume of the larger pores. The amount of adsorptively and capillarily fixed water also varies with the water vapour pressure in the surroundings of the concrete as shown in figure 2.1D which shows the absorption isotherms for neat cement paste as determined by Powers and Brownyard (56). The intercept of each curve on the vertical axis represents the amount of chemically combined water.

Generally speaking, the order in which the evaporable water is lost bears some relationship to the pore size in which it occurs (60), with the water from the largest pores being lost first, followed by water from the next sized pores and so on. However, this process is not a discrete one as may appear here, in fact it is a continuous process in the sense that when water is being lost from the largest pores, moisture movement in the next sized pores may already be in progress. This is particularly so under higher ambient temperature and lower vapour pressures. A reversed order is followed when concrete reabsorbs moisture from the surroundings i.e the smaller sized pores become saturated first and the largest sized pores are saturated last of all.

2.3.2 Effect of Moisture on Concrete Properties at Room Temperature

Strength, modulus of elasticity, shrinkage and creep of concrete are directly dependent on the amount of chemically

bound water present in the concrete which assists in the hydration of cement paste and thus leads to the strength development. Adsorptionally and capillarily bound water generally speaking, result in a reversible reduction of the concrete strength. For example, Gilkey (61) showed long ago that concrete specimens gently air-dried before testing can exhibit ultimate compressive strengths 20 to 30 per cent higher than those obtained from saturated specimens. Lea (62) also pointed out that if matured and dried concrete is subsequently wetted in water, there is a drop in compressive strength of 20 to 40 per cent. This phenomenon has been ascribed by various authors to:

i) A general weakening of the cement gel due to the adsorbed water,

ii) A reduction of the surface forces (van der Waal) between gel particles in the presence of adsorbed moisture.

or in other words, removal of adsorbed water causes an increase in the van der Waal forces thereby producing a stronger, more rigid structure.

Mills (63) suggested that the strength of cement gel is due to a combination of inter-locking gel particles plus van der Waal forces between gel particles. Dilation of the gel by adsorbed water attenuates forces and renders them less effective. Free water in the gel exerts a wedge action between the solid part weakening the material.

The shrinkage and the creep of concrete are also adversely affected by the adsorptively and capillarily bound water (64). The thermal properties, density, permeability and the ability to absorb electro-magnetic radiation are dependent only on the total amount of water present rather than on their individual states (58). Fire resistance of concrete primarily depends on the amount of water which can evaporate at the temperature in question. Table 2.2 presents a rough survey of the concrete properties which are most influenced by the various states of water in concrete. It should be noted that a property such as the frost resistance of concrete, though directly dependent on the percentage of freezable water (which roughly corresponds to the free water content), may also depend indirectly on other types of water present. For example, the ultimate strength and rheological properties of concrete which are dependent on the gel pore water, would dictate the stresses and strains of concrete under frost action, but this is not shown in table 2.2..

Various research workers have carried out tests to study the properties of concrete as influenced by moisture content at elevated temperatures. These will be briefly discussed in the next section.

2.3.3 Effect of Moisture on Concrete Properties at Elevated Temperatures

Sealed and unsealed specimens have been tested to investigate the effect of moisture on compressive strength, flexural strength and modulus of elasticity of concrete at elevated Temperatures. In one such series of tests, Hannant (65) observed

that sealed concrete specimens which were heated to 150°C and then cooled to room temperature, retained only 65 percent of the reference strength as opposed to 85 percent for the unsealed specimens tested under similar conditions. His specimens consisted of 150 mm. (6 inches) diameter by 300 mm. (12 inches) high cylinders, some with a 50 mm. (2 inches) diameter, concentric hole. These were made from ordinary portland cement and limestone aggregates, and were cured for two months under water at 20°C and for a further period of two months either under water or in the laboratory air. Some of the specimens were covered with copper foil wrappings to prevent any change in the moisture content of the specimens during testing. The specimens which were heated up to 150°C at a rate of 15°C per hour to avoid thermal shock, were loaded in tri-axial compression to failure, after they had been cooled to room temperature. He observed that the strength of all cylinders tested at 100°C could be related to the moisture loss, the drier specimens showing greater strength. He also established that the residual strength was dependent on the moisture content of the specimens at the time of test as well as on the test temperature.

Lankard, et al (66) also tested sealed and unsealed cylinders (100 mm. diameter by 200 mm. high) as well as beam specimens (150 mm. by 62.5 mm. by 255 mm.) made from ordinary portland cement and gravel or limestone aggregates. The specimens, after curing in a fog room for 28 to 200 days, were heated up to 500°F (260°C), some of which were tested hot and others were tested after being cooled to room temperature.

They observed that with the unsealed specimens:

i) The compressive strength either increased or only slightly decreased,

ii) The flexural strength increased slightly at 175°F (80°C), but showed moderate to slight decreases at 250°F (120°C) and 500°F (260°C),

iii) The modulus of elasticity showed slight decrease at all temperatures.

They also observed that the evaporable water was nearly completely lost at 175°F (80°C) with all evaporable water lost at 250°F (120°C). Further, an increase in the chemically combined water occurred at 175°F (80°C), and a loss of 10, 20 and 40 percent of this water took place at 250°F (120°C), 375°F (190°C) and 500°F (260°C) respectively.

They attributed the apparent increase in the compressive strength for unsealed specimens slowly heated up to 500°F (260°C) to the removal of the free moisture, since this leads to drier and hence stronger concrete. The drop in the flexural strength after 175°F (80°C) was attributed to the loss of the chemically combined water, which offset the beneficial effect of the removal of the evaporable water. The reduction in the modulus of elasticity under all heating conditions was due to the loss of an incompressible phase i.e the evaporable water in the concrete system.

In general, their test results indicated a further deterio-

ration when the specimens were tested after being cooled to room temperature. Though they did not give any explanation for this, it is probable that this was due to reabsorption of moisture by the specimens during cooling.

They suggested that test conditions capable of producing a large thermal gradient or of influencing the amount of moisture content in a given concrete or both at the time of testing can override the effects produced by the removal of evaporable water under unsealed condition. They also proposed that parameters such as aggregate type, water-cement ratio, age etc. which do not affect the thermal gradient or the moisture content condition will not have a significant influence on the structural properties of concrete. Though this observation itself may be correct, the parameters mentioned by them such as aggregate type can have a considerable effect on the properties of concrete at high temperatures.

In case of the sealed concrete specimens, they observed that:

i) The compressive strength decreased continuously with increased temperature and retained only about 50 percent of that at room temperature,

ii) The modulus of elasticity was severely reduced at all temperatures, particularly at 500°F (260°C) where it was only about 30 percent of that at room temperature.

They also noticed that for the sealed concrete specimens, a change in the hydrated cement phase took place after heating to 375°F (190°C) and 500°F (260°C). An X-ray analysis showed the presence of new and more highly crystalline phases which had poorer cementing qualities than the original highly cementitious calcium silicate hydrates. Further, the steam pressure developed in the pore structure of the cement paste and concrete acted as a potential crack driving force. However, they claimed that the vapour pressure of the adsorbed water at the same temperature, consequently changes in the surrounding water vapour pressure would be expected to modify the behaviour of the adsorbed water with unknown consequences. Local variations in temperature in the specimen also could result in steam pressures inside existing cracks thereby producing stress intensity sufficient to propagate the same.

They suggested that since exposure time at different temperatures determine the amount of evaporable water content, it must be taken into consideration while using results from tests on unsealed specimens. In fact, this parameter can assume great significance under transient heating conditions. They pointed out that with sealed concrete specimens, the possibilities of hydro-thermal reactions should be considered as well.

Hannant (1) indicated that the values of creep strains for concrete that had been dried before loading were only 0.1% of the values for "wet" concrete, which are surprisingly low, in contrast Kennedy et al (2) observed that the values of compressive creep and tensile creep strains were generally larger

for an air-dried specimens than an as-cast specimens (except in the case of low tensile creep, where the reverse was true), whatever the outcome of these investigations, they show the significance of moisture influence on creep.

2.3.4 Shrinkage

Shrinkage can be considered to be a time-dependent deformation in the concrete which occurs due to the movement of gel water from within the gel pores. The evaporative process of the drying out of the concrete is sufficient to cause these shrinkage strains. See figure 2.1 .

In other words, stress is not involved in this process although shrinkage due to drying out and creep due to stress occur in the same concrete member. Since moisture seepage takes place in the cases of both shrinkage and creep. It is not possible to observe the two phenomena separately in the same specimen.

The type of cement has an influence on the shrinkage of the resulting concrete, Carlson (67) claims that shrinkage increases in the sequence ordinary Portland cement. Low heat cement, Portland Pozzolona cement.

The fineness of cements has been commented on the Davis, Carlson, Troxell and Kelley (68) in connection with the construction of the Hoover Dam. Finer cements showed a slight increase in shrinkage.

As would be expected, experiments linking shrinkage and water-cement ratio indicate a direct proportional relationship (67). Both the type of the aggregate (69) and its maximum size of aggregate increases (70).

Shrinkage rate decreases with time and shrinkage strain-time curves take a very similar form to creep-time curves. Wet curing of concrete specimens can be expected to delay the process of drying shrinkage of the onset of shrinkage can be held back, the concrete will have an opportunity to increase its tensile strength with time, thus, as would be expected, a high relative humidity considerably inhibits drying shrinkage.

2.3.5 Theories of Shrinkage

Shrinkage, however, is now generally assumed to be made up of two parts:

- a) The drying shrinkage due to the evaporative loss of the free water,
- b) The chemical shrinkage resulting from the cement hydration.

The cement and water at mixing occupy a larger physical volume than do the products of chemical reaction after hydration. Although drying shrinkage is the predominant component of the total, both components influence construction techniques to a considerable extent and not only in the curing procedures. (42)

The chemical or plastic shrinkage naturally increases with cement content and can result in a shrinkage of the order of 0.6% of the original volume during the first few hours after mixing (71). Concrete forms are usually filled 3 or 4 mm. proud to offset this form of shrinkage which manifests itself as a settlement of the top surface of the concrete as it sets.

2.3.6 Moisture Movement and Shrinkage at Elevated Temperature in PCPV

Concrete used in prestressed concrete pressure vessels for reactors is subjected to conditions unlike those in conventional civil engineering structures. It is used to form massive sections, often with little bonded reinforcement, it is subjected to sustained biaxial and triaxial states of stress and is called upon to operate under elevated and non-uniform temperatures, at the same time it represents an economic radiation shielding material, because it contains light as well as heavy elements, the light elements, predominantly hydrogen, serve to slow down fast neutrons. From the view-point of radiation protection therefore the highest possible moisture content spread evenly over the cross section of the vessel and not or just slightly changing with time would be desirable. This however, for different reasons cannot be achieved (72).

In the wall of a reactor vessel there are driving forces created by the vapour pressure resulting from non-uniform temperature and these cause water to migrate within and from the vessel wall. Shrinkage and swelling can result from these water movements.

Although it may be possible to estimate strains close to outer face of a vessel by employing diffusion analyses, this approach cannot be applied to the region of the vessel close to the liner where moisture migration is likely to be taking place at a higher rate than anywhere else in the vessel (73). A better understanding of the manner in which moisture migrates from regions of elevated temperature is therefore of vital importance if the effects on other concrete properties are to be understood fully. Ross, Illston and England (74), found that on a simulated thick-wall section, sealed at the hot face and subjected to a thermal gradient of $12^{\circ}\text{C}/\text{ft.}$, drying had only taken place in the foot of concrete adjacent to each face after one year of heating. Sharp (75) investigated this in more detail with sealed cylindrical specimens in lengths ranging from 2 ft. to 10 ft. Open at one end to atmosphere, and heated for 2 years at their bases to temperatures of 80°C , 105°C and 135°C . He observed that penetration of the drying front at the hot end was temperature related. Depending on the length of the moisture path, water content values in excess of the mixing water values were observed in the intermediate regions of the specimen. Evaporation drying took place at the ends open to atmosphere. A typical phase diagram for water in such specimens was produced by England and Ross (73), and can be seen in figure (2.11).

Similar migration studies to those of Sharp have been reported by Hornby (78) and McDonald (79), neither of whom observed these areas of increased water content. However, Hundt and Schimmelwitz (80), in a sealed moisture migration specimen with

a hot face temperature of 80°C , observed an area in the intermediate region of the specimen with a small increase in moisture over the moisture content before heating.

Parkinson (76) investigated water movement and any accompanying shrinkage in sealed concrete to a thermal gradient. Measurements of moisture content showed an accumulation of water in the cool regions and this was accompanied by "swelling" of the concrete. This "swelling" phenomena was also reported by England and Ross (73) in a series of laminated specimens subjected to a thermal gradient and unidirectional moisture flow. England and Sharp (77) reported measured pore pressures at various points along the axes of sealed cylindrical specimens under the action of sustained thermal gradients. The pore pressures were observed to dissipate slowly in the hot regions as water migrated towards the cooler parts of the specimen, and were indicators of the changing water content in the specimens. They concluded that at temperatures in excess of 100°C , the pore pressures became significant and major factors affecting the movement of moisture.

Recently, Chapman and England (81), have investigated the uniaxial migration in long, upright, limestone concrete cylinder of two lengths, 5 ft. and 10 ft. and of 6 in. diameter, sealed at the base and sides, and open at the top. The design represents a section through a concrete pressure vessel wall.

The cylinders are subjected to a sustained temperature difference between their ends, with maximum temperatures between

105°C and 200°C. Readings of pore pressure, water content and temperature are taken at various positions along the axis of the cylinders. In one cylinder, transverse and longitudinal shrinkage readings are also recorded. See figure (2.12).

The results for the cylinders show that moisture migration is away from the hot face of the specimens, causing reduction in both pore pressure and water content values in this region. The moisture migration creates a drying front which moves slowly up the specimens. The rate at which this drying front moves is influenced by the base temperature, the magnitude of temperature and pressure gradients and the coefficient of permeability of the concrete. Samples taken from the hot side of the drying front show a considerable increase in the coefficient of permeability, and Scanning Electron Microscope photographs of the microstructure show both a break-up and reduction in size of the hydration products.

Proceeding the hot drying front are waves of moisture resulting in "wet" regions in the centre of the specimens, where the total water content becomes greater than at the time of mixing. Scanning Electron Microscope photographs of samples from these "wet" regions indicate that the elevated temperatures and excess moisture enhance the concretes' hydration. Values of the coefficient of permeability are in the order of 100 times less than those for the hot side of the drying front moves into the concrete and considerable weight loss is recorded as moisture escapes to the atmosphere. The rate of movement of the

drying front is slower than that of the hot front and is proportional to the temperature differenced between the top of the specimen and the surrounding atmosphere.

In the shrinkage specimen, values of transverse and longitudinal shrinkage reflect the water content results. The specimen indicates that shrinkage occurs in a concrete pressure vessel, in the regions where moisture is lost. The restraint of the mass of concrete surrounding these regions sets up a three dimensional state of internal tensile stress. The areas into which the moisture migrates tend to swell, creating an internal stress situation, which is this time compressive. These results indicate that if shrinkage strain values for concrete pressure vessels are calculated by subtracting the initial thermal movement from the total predicted movement, a higher value is generally obtained than occurs in practice.

The experiments reported indicate that when the hot inner face temperature of a concrete pressure vessel is increased above 100°C , the drying rate inside the wall increases considerably. However, it is unlikely pressure vessels of the size currently in use will ever completely dry out.

2.3.7 Thermal Expansion of Concrete

The thermal coefficient of expansion of concrete varies with the moisture content, decreases with age, and increases with the richness of the mix and with the temperature. For an oven-dry gravel concrete, Hatt (86), observed a coefficient of $4.0 \times 10^{-6}/\text{F}$ at 60 F ($7.2 \times 10^{-6}/\text{C}$ at 15.6C) and $6.5 \times 10^{-6}/\text{F}$

at 150 F ($11.7 \times 10^{-6}/^{\circ}\text{C}$ at 66 C). The same trend was noted for saturated and partially dry samples. Davis (82), reported that alternations of high and low temperatures within the atmospheric temperature range reduce the thermal coefficient. In series of tests of 100 alternations between 15 and 130 F (- 9 to 54 C) the thermal coefficient was found to decrease by 14.1 percent. Richness of mix increases the coefficient of thermal expansion of concrete, because the hydrated cement constituent has a higher thermal coefficient than that of the aggregate constituents (85). Data reported by Mitchell (83) and Walker et al (87), indicate that air-entraining admixtures have a minor effect upon the magnitude of the thermal coefficient of expansion. Bonnel and Harper (85), reported that age had a minor effect upon the thermal coefficient for concretes tested for 1 year; however, Berwanger (88), found that on the basis of concretes tested within a temperature range of - 40 to 27 C (- 40 to 80 F) the coefficient for a given water-cement ratio will reduce at an estimated rate of $0.01 \times 10^{-6}/^{\circ}\text{F}/\text{day}$ ($0.018 \times 10^{-6}/^{\circ}\text{C}/\text{day}$) for specimens 29 to 200 days old and will level off at some later age.

It has been reported that the moisture content of concrete, which is related to the relative ambient humidity, strongly affects the thermal coefficient of expansion but that the water-cement ratio has little effect. The variation between saturated and partially dry concrete specimens is less than that for hardened cement paste. From data reported by Meyers (84), the thermal coefficient for traprock concrete at the maximizing moisture content is 17 percent higher than that at the saturated

condition, whereas the maximum coefficient of limestone concrete for partially dry specimens is 56 percent higher than for the saturated condition. Data from other investigators (83, 85, 87) for concrete of various aggregates indicate that a maximum thermal coefficient for the partially dry state is approximately 15 percent higher than that for the saturated condition.

Because moisture has little effect upon the coefficient of thermal expansion of an aggregate, it is readily apparent that the proportion of aggregate is the primary factor contributing to the reduction of the effect of moisture content on the variation of the thermal coefficient for concrete as compared to cement paste. In other words, an increase in the amount of aggregate will decrease the moisture effect.

Walker et al (87), reported that the thermal coefficient of concrete is dependent upon the volumetric weighted average of its ingredients.

Recently Emanuel and Hulsey (89), have suggested empirical equation, which takes into account age, moisture content, and mix proportion as the principal factors.

Let

α_c = thermal coefficient of linear expansion for concrete

α_{CA} = thermal coefficient of linear expansion of coarse aggregate

See table (2.3).

α_{FA} = thermal coefficient of linear expansion of fine aggregate

See table (2.3).

f_T = correction factor for temperature alternations (1.0 for controlled environment; 0.86 for outside exposure).

f_M = correction factor for moisture (see figure 2.13)

f_A = correction factor for age (see figure 2.13)

B_P = proportion by volume of paste

B_{FA} = proportion by volume of fine aggregate

B_{CA} = proportion by volume of coarse aggregate

then

$$B_P + B_{FA} + B_{CA} = 1.0$$

The aggregate occupy approximately 70 to 80 percent of the volume of the concrete. If the proportions are unknown, it will be assumed that the aggregates occupy 75 percent of the volume, with 30 percent of the aggregate considered to fine aggregate.

By combining the above, the thermal coefficient of expansion becomes

$$\alpha_C = f_T (f_M f_A B_P \alpha_S + B_{FA} \alpha_{FA} + B_{CA} \alpha_{CA})$$

Where

α_S is the thermal coefficient of expansion for saturated cement paste ($10.8 \times 10^{-6}/C$)

2.4 The Separation of the Strain Components

The total strains occurring in the concrete of pressure vessels consist of the elastic, the creep, the shrinkage and the thermal strains, and the value of the strains measured experimentally is the algebraic sum of these components (42).

The elastic strain can be observed directly after loading the specimens as can the thermal strains immediately after heat application. The thermal strains can, in any case, be arrived at by calculation knowing the coefficient of linear thermal expansion of the concrete. The shrinkage strains can be plotted against time, as a result of continuous observations on unloaded specimen which are allowed only to shrink. Thus each of the components of strain can be separately evaluated for use in the subsequent calculations as required.

There is an interrelationship between creep and shrinkage since they are both greatly influenced by the ambient relative humidity, there is also some evidence to indicate that shrinkage strains in causing stresses, may increase creep strain, and consequently of strain, including shrinkage, from the total, may lead to small errors.

However most analytical work on this problem in the past has assumed that creep is independent of shrinkage, that loading neither increases nor decreases shrinkage and that shrinkage either uniform or non-uniform, does not increase the stress.

2.5 Concluding Remarks

The preceeding review of the effect of temperature on creep, moisture and shrinkage indicate that:

1) Creep at elevated temperatures follows the same general pattern as creep at room temperature, i.e., it is approximately an exponential function of the time under load and a fairly linear function of the stress applied at least up to a stress/strength ratio of about 0.5 and temperature below 300°C . Creep decreases with increasing maturity and increases with increasing moisture content of the specimen at loading. Poisson's ratio in creep appears larger at elevated temperature than that at room temperature.

2) The effect of elevated temperatures (at least up to 50°C) is to increase creep, creep at 50°C being approximately two to three time as great as creep at room temperature.

3) For temperatures of 50 to about 100°C , some controversy exists about whether or not there is a further increase of total creep with increasing temperature. Some investigators have found a definite maximum of total creep in the range of 50 to 80°C , but most have not and have concluded that creep of concrete increases with temperature up to around 100°C . The creep at 100°C being on the order of four to six times as great as the creep at room temperature (at the end of 60 to 100 day loading period).

4) For temperatures of 100 to about 150°C , some

controversy also exists about whether, or not there is a further increase of total creep with increasing temperature. Some investigators (39, 40) have found a definite maximum of total creep in the range of 120 to 140°C, but most (30, 41, 29, 30) have not. Beyond 140°C, it is generally agreed that creep rate and creep magnitude increase with temperature (unsealed specimen).

5) Only few data are available concerning the influence of triaxial tests, stresses at right angles to the principal stress were found to reduce creep in the direction of the principal stress within the range of 20 to 100°C; however considerable creep remains even under a hydrostatic state of stress. Beyond 100°C, to the Author's knowledge no data are available so far.

6) Some controversy apparently exists concerning creep recovery. Although Theuer (21) and Nasser and Neville (26, 27) found creep recovery to be essentially independent of temperature and stress, Da Silveira and Florentino (35) reported significantly a higher creep recovery at 45°C than at room temperature.

7) The properties of concrete both at room temperature and at elevated temperature, are influenced by the moisture content. However, the extent and manner in which the various properties are affected by moisture content, depend on the state in which it occurs. These states can be classified in terms of the bonding energy existing between moisture and the solid phase.

8) If the hot inner face temperature of a concrete pressure vessel is increased above 100°C , the drying rate inside the wall would increase considerably causing reduction in both pore pressure and water content values close to the liner, where shrinkage occurs. The cooler areas into which moisture migrates tend to swell, creating an internal stress situation. This indicates quite clearly that thermal triaxial creep tests on fully saturated specimens are unnecessary when temperature exceed 100°C , contrary to common opinion.

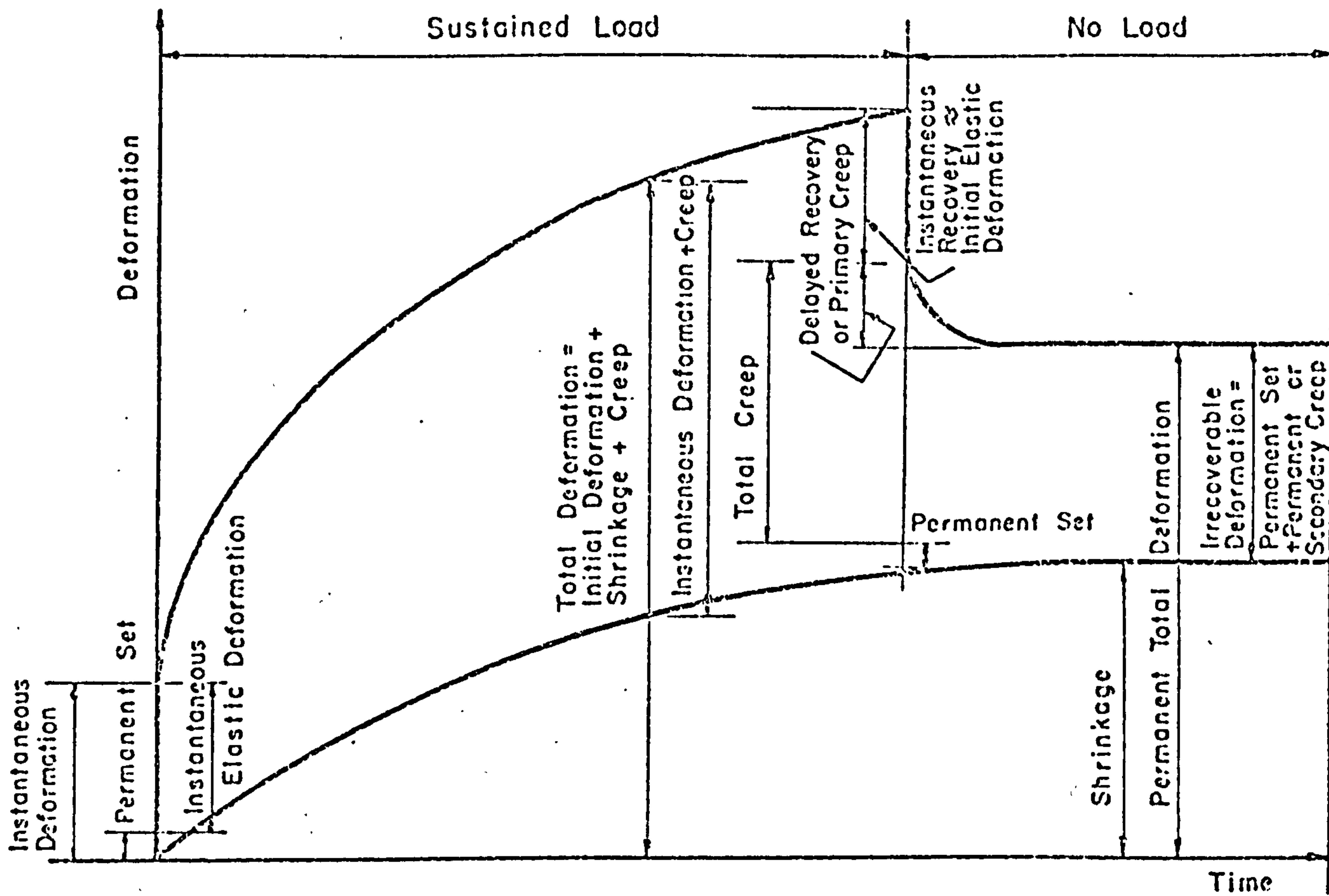


FIGURE 2.1a TYPICAL DEFORMATION V.S. TIME CURVE FOR CONCRETE UNDER CONSTANT LOAD

Ref. 93

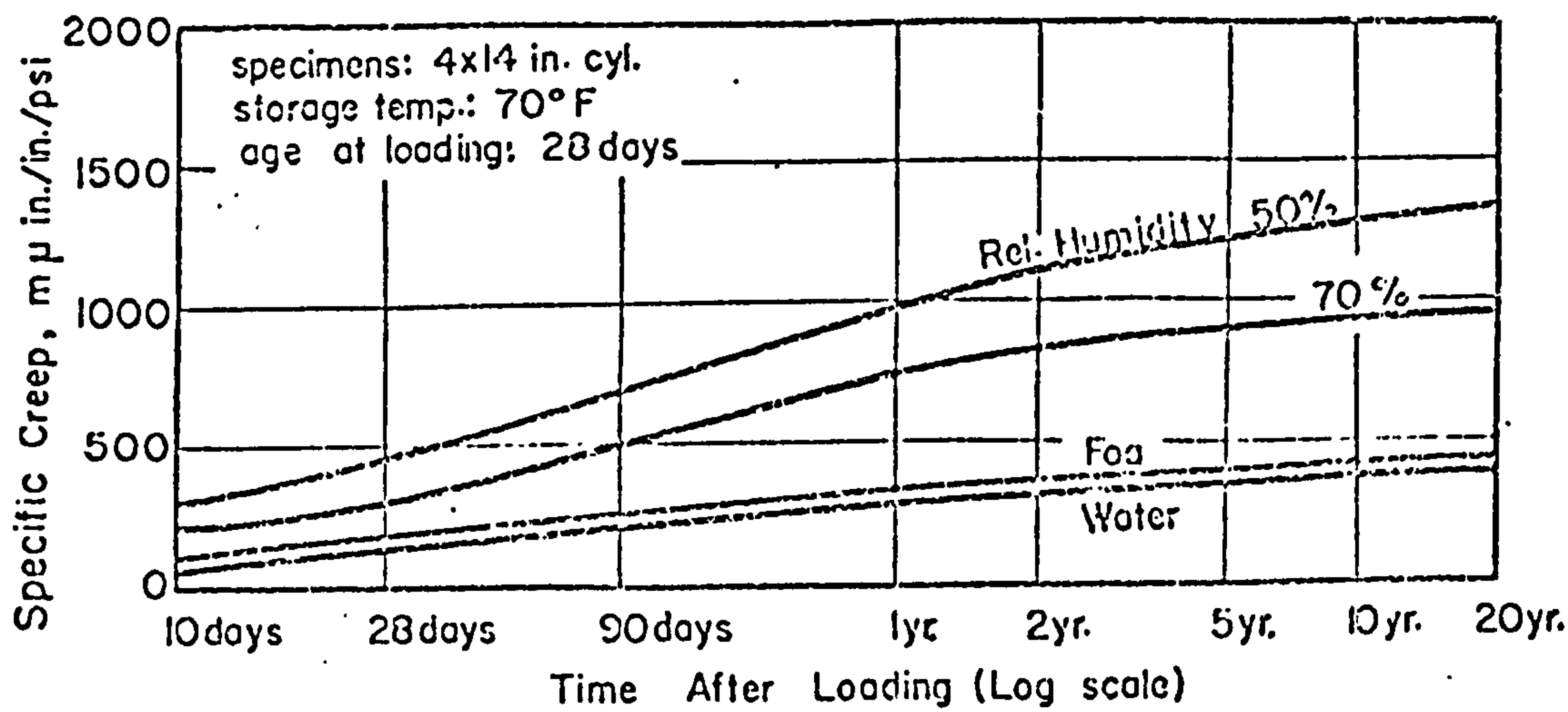
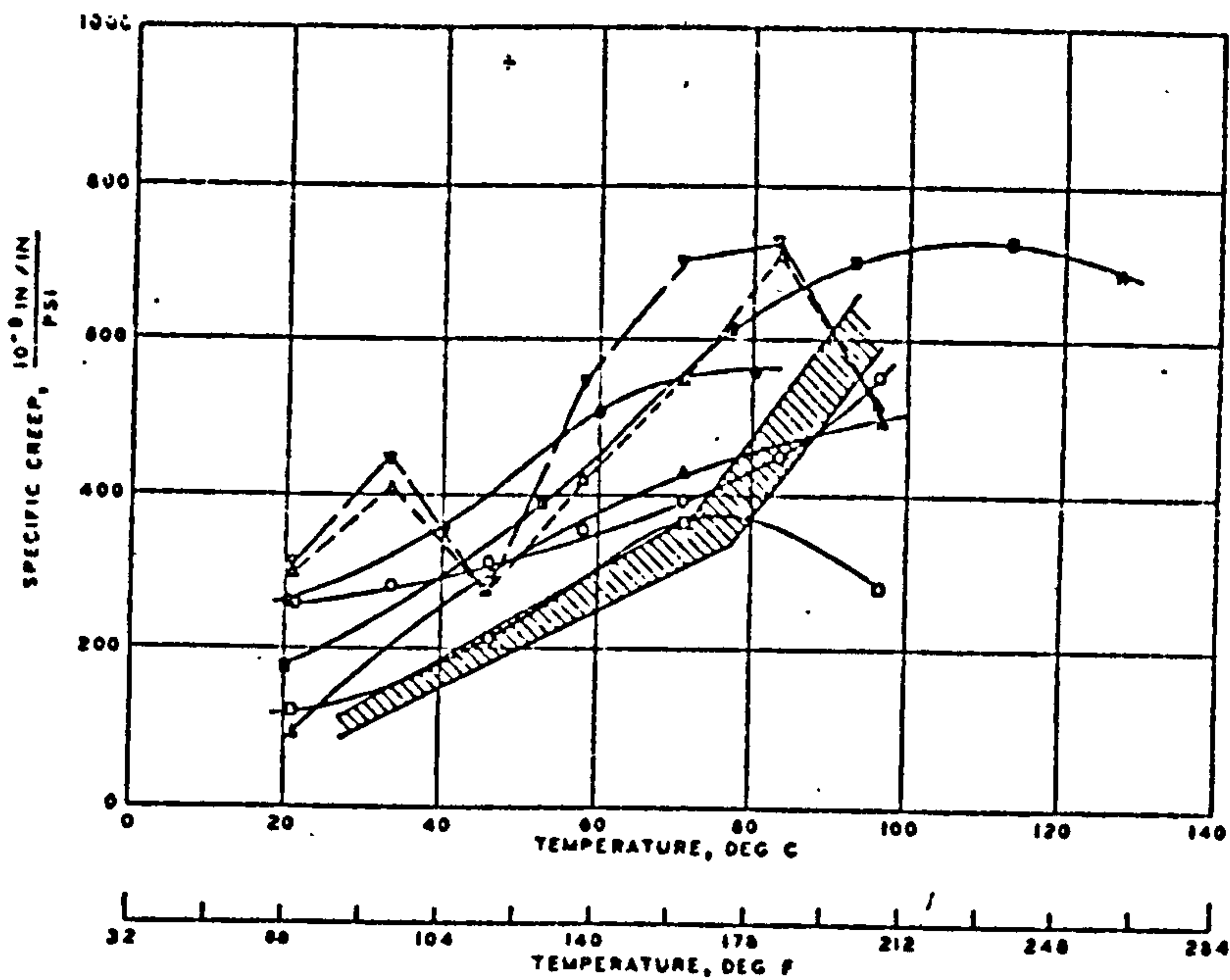


Figure 2.1b EFFECT OF MOISTURE CONDITION OF STORAGE ON CREEP

Ref. 93

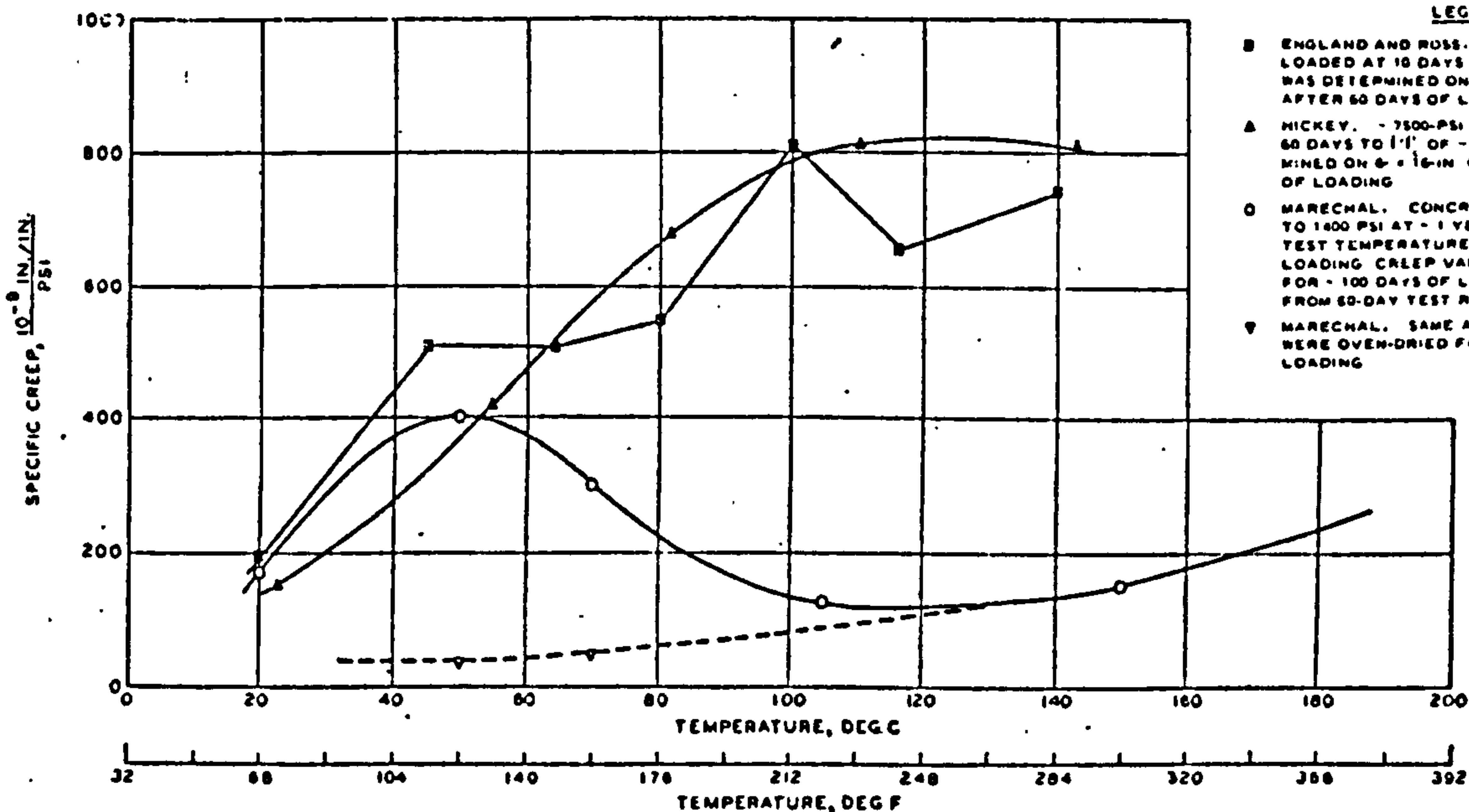


- LEGEND**
- MASSER AND NEVILLE. 5370-PSI, 10-YEAR-OLD CONCRETE LOADED TO $1/2$ OF f'_c OF 8.45, CREEP MEASURED ON 3- $\times$ -9-1/4-IN. CYLINDERS AFTER 90 DAYS OF LOADING STORED IN WATER
 - MASSER AND NEVILLE. 5000-PSI DOLOMITE HORNBLENDE CONCRETE LOADED AT 14 DAYS TO $1/2$ OF f'_c OF 8.35 CREEP DETERMINED ON 3- $\times$ -9-1/4-IN. SEALED CYLINDERS AFTER 90 DAYS OF LOADING
 - △ SAME AS ABOVE, BUT f'_c = 8.63
 - ▽ SAME AS ABOVE, BUT f'_c = 8.78
 - ENGLAND AND ROSS. 5000-PSI CONCRETE LOADED AT 10 DAYS TO $1/2$ OF f'_c OF 8.20; CREEP DETERMINED ON 4-1/2- $\times$ -12-IN. SEALED CYLINDERS AFTER 90 DAYS OF LOADING
 - ▨ HANNAH. 8000-PSI LIMESTONE CONCRETE LOADED AT 6 MONTHS TO $1/2$ OF f'_c OF 8.09 TO 8.25, CREEP DETERMINED ON 4-1/2- $\times$ -12-IN. SEALED CYLINDERS AFTER 100 DAYS OF LOADING
 - ARTHANARI AND YU. 5000-PSI GRAVEL CONCRETE LOADED AT 13 DAYS TO $1/2$ OF f'_c OF 8.20, CREEP DETERMINED ON 12- $\times$ -12- $\times$ -4-IN. SLABS AFTER 90 DAYS OF LOADING
 - ▲ MASSER AND NEVILLE. 7250-PSI DOLOMITE HORNBLENDE CONCRETE LOADED AFTER 1 YEAR WATER STORAGE TO $1/2$ OF f'_c OF 8.45, CREEP DETERMINED ON 3- $\times$ -9-1/4-IN. CYLINDERS AFTER 90 DAYS OF LOADING UNDERWATER

Figure 2.2 from ref. 43

INFLUENCE OF
TEMPERATURE ON CREEP
SEALED OR WATER-STORED
SPECIMENS AFTER 60, 80, 90, OR
100 DAYS OF LOADING

(Uniaxial Results)



- LEGEND**
- ENGLAND AND ROSS. 5000-PSI CONCRETE LOADED AT 10 DAYS TO $1/2$ OF f'_c OF 8.20; CREEP WAS DETERMINED ON 4-1/2- $\times$ -12-IN. CYLINDERS AFTER 90 DAYS OF LOADING
 - △ HICKEY. 7500-PSI CONCRETE LOADED AT 60 DAYS TO $1/2$ OF f'_c OF 8.10, CREEP WAS DETERMINED ON 4- $\times$ -16-IN. CYLINDERS AFTER 107 DAYS OF LOADING
 - MARECHAL. CONCRETE IF UNKNOWN LOADED TO 1800 PSI AT 1 YEAR AGE, SUBJECT TO TEST TEMPERATURE FOR 14 DAYS PRIOR TO LOADING CREEP VALUES PLOTTED ARE THOSE FOR 100 DAYS OF LOADING (EXTRAPOLATED FROM 60-DAY TEST RESULTS)
 - ▽ MARECHAL. SAME AS ABOVE, BUT SPECIMENS WERE OVEN-DRIED FOR 1 MONTH BEFORE LOADING

Figure 2.3 from ref. 43

INFLUENCE OF
TEMPERATURE ON CREEP
UNSEALED SPECIMENS AFTER
60, 100, OR 107 DAYS OF LOADING

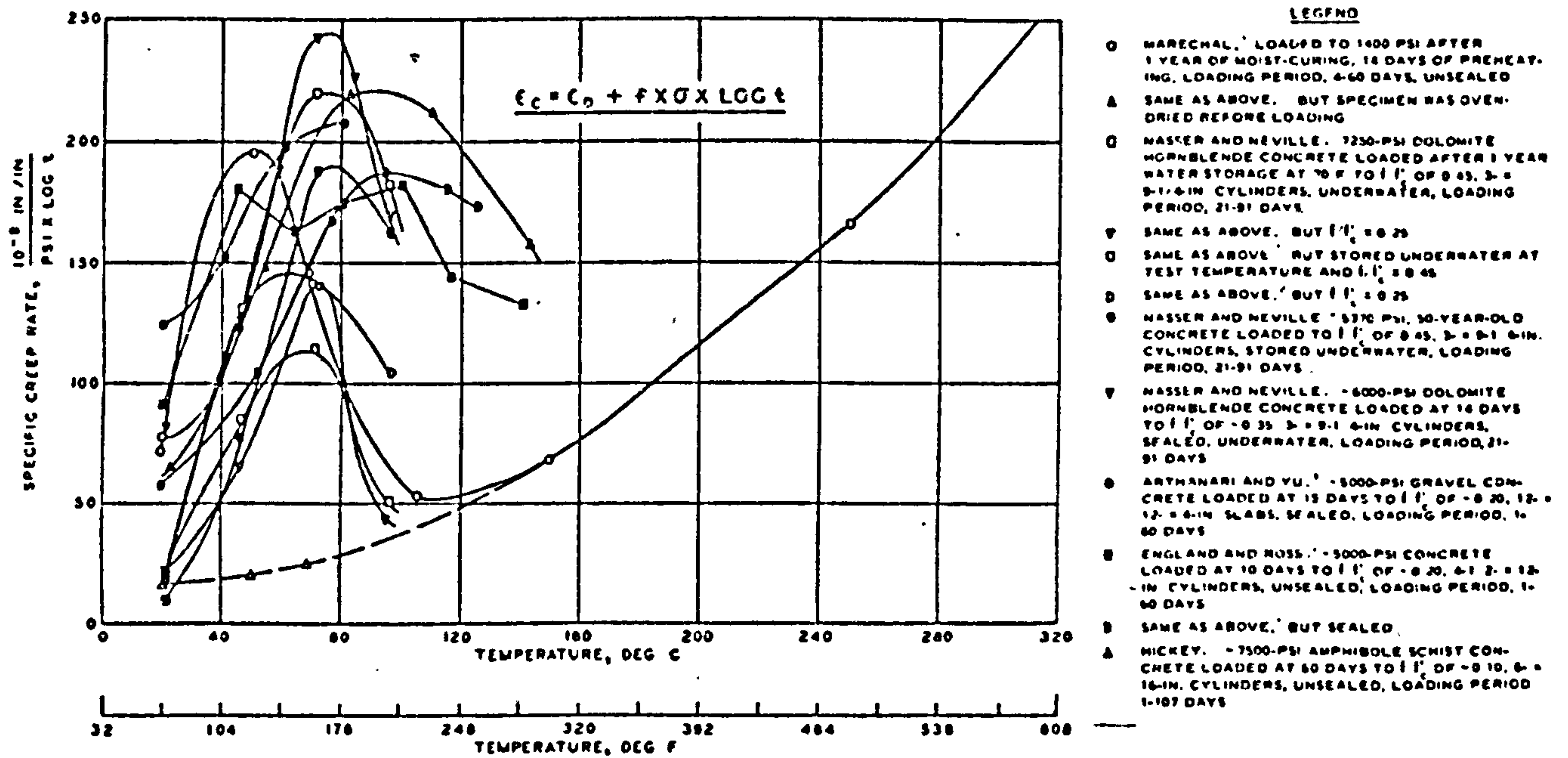


Figure 2.4 from ref. 43

INFLUENCE OF TEMPERATURE
ON CREEP RATE

$$f/f'_c \leq 0.50$$

(Uniaxial Results)

TABLE 2.1-SPECIMEN FAILURE

Failure order	f'_c (psi)	f_c/f'_c	Temp. °F	Failure time: after loaded
1	6700	0.6	800	21 days
2	5500	0.6	800	256 days
3	6700	0.6	600	326 days, about 2 minutes heat was cut off
4	6700	0.4	800	326 days, about 40 minute heat was cut off
5	5500	0.6	600	326 days, and temperature back to ambient
6	5500	0.4	800	326 days, and temperature back to ambient

From ref. 41

 f_c/f'_c is stress-strength ratio

(Uniaxial Results)

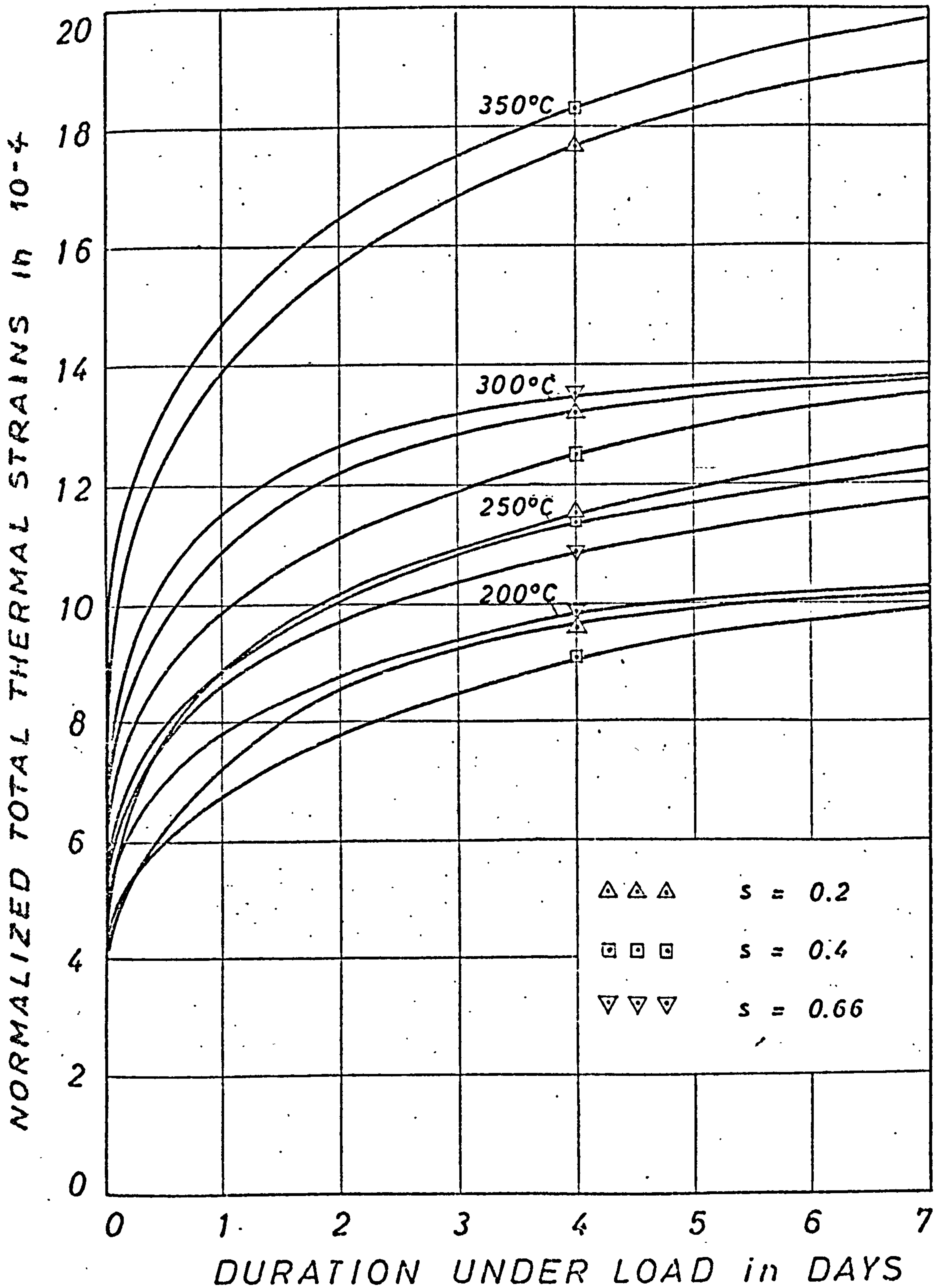


Figure 2.5 High Temperature Creep Normalized to a Stress Level of 0.2 from ref. 39

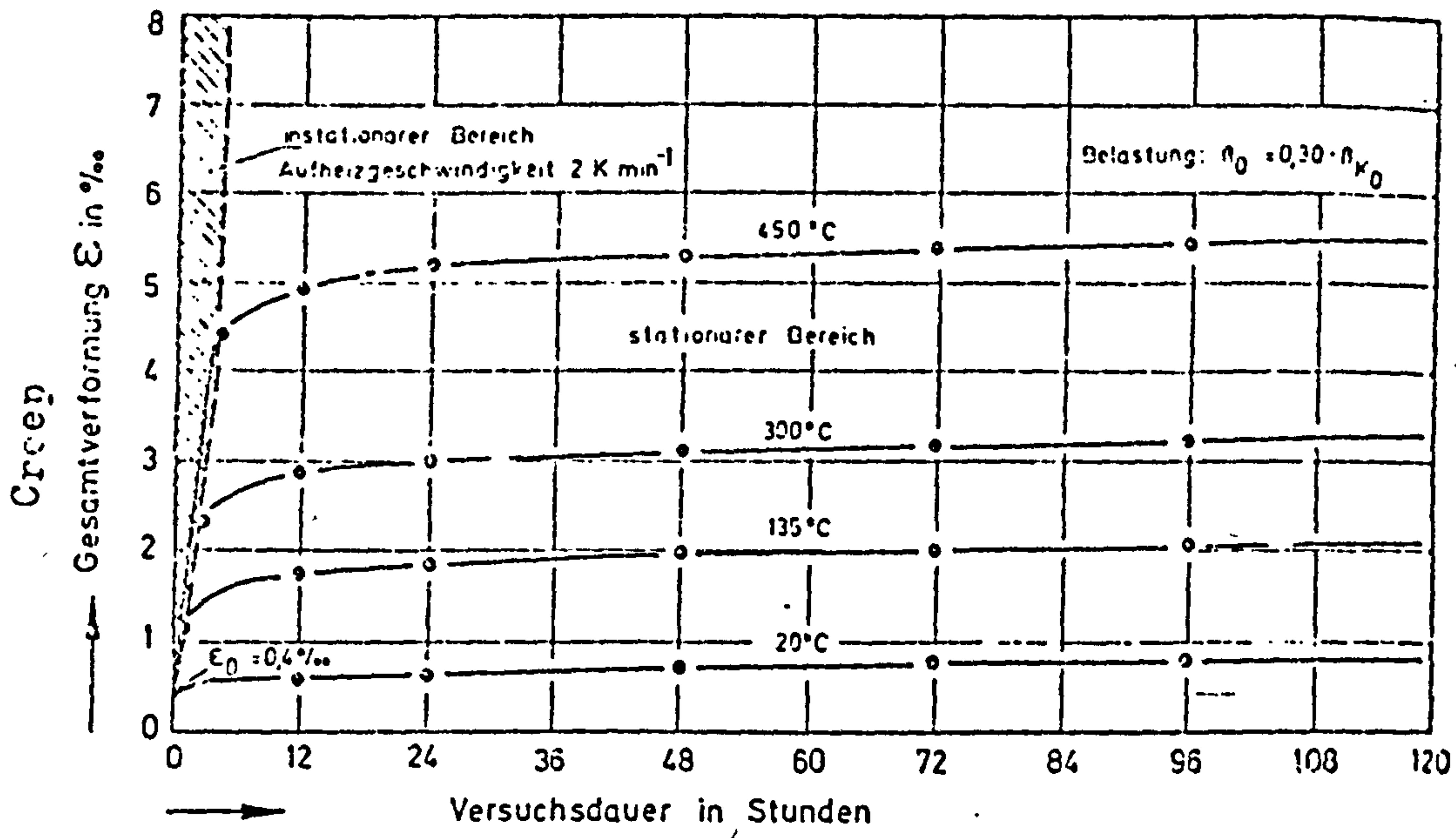


Figure 2.6 Ref. 40 Time in hours.

(Uniaxial Results)

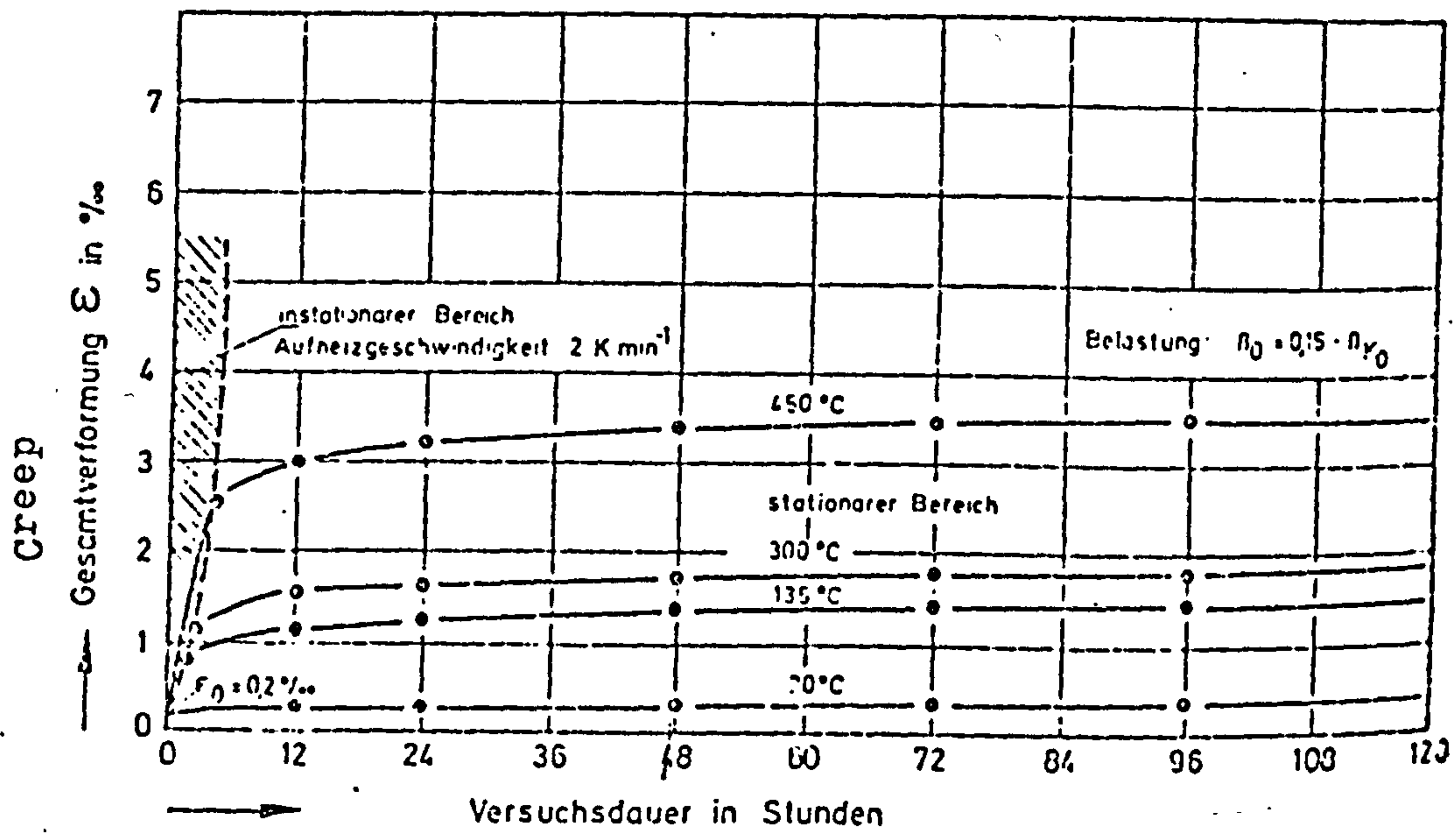


Figure 2.7 Ref. 40 Time in hours.

(Uniaxial Results)

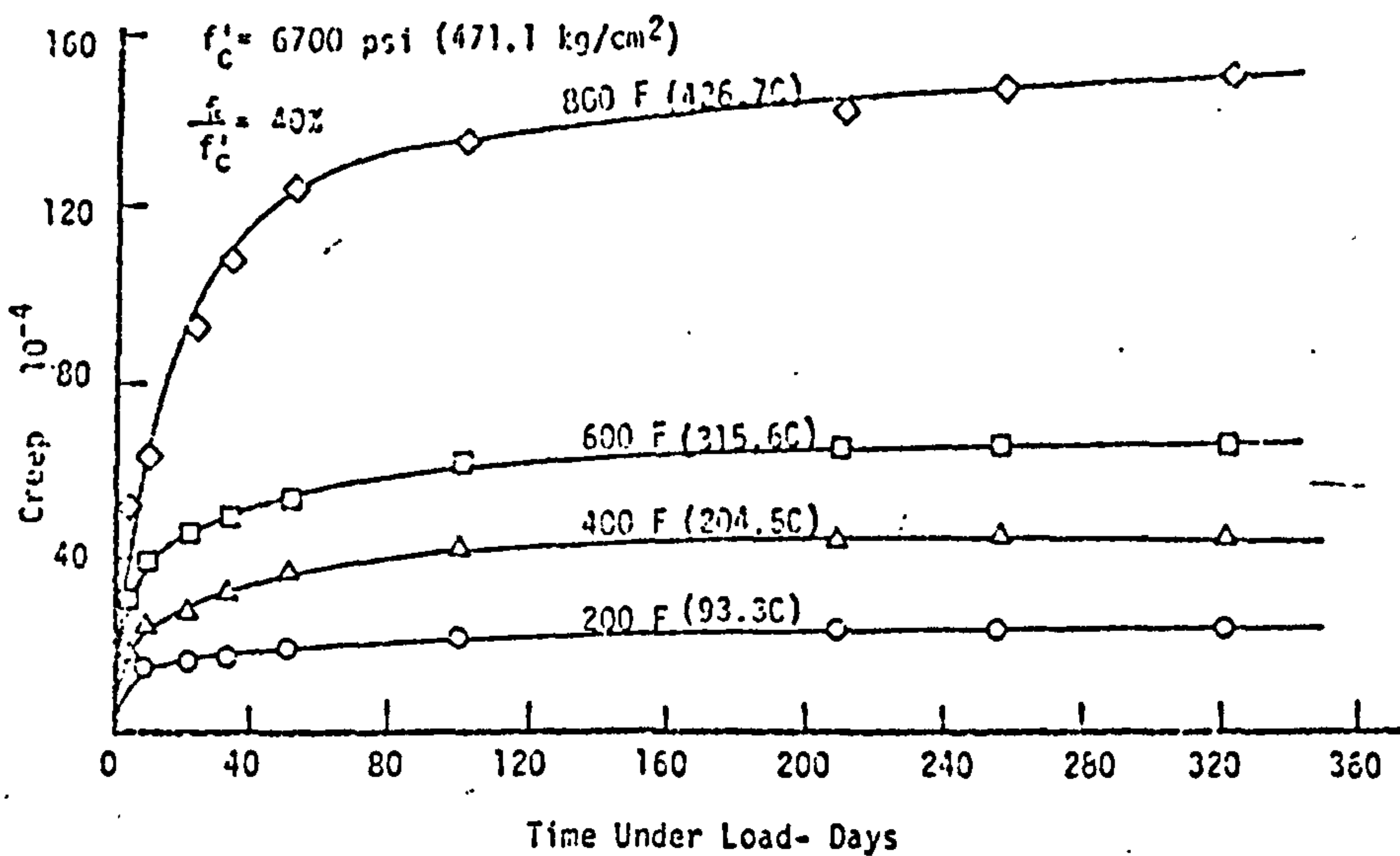


Fig. 2.8 -Relation between creep and time under load
From ref. 41

(Uniaxial Results)

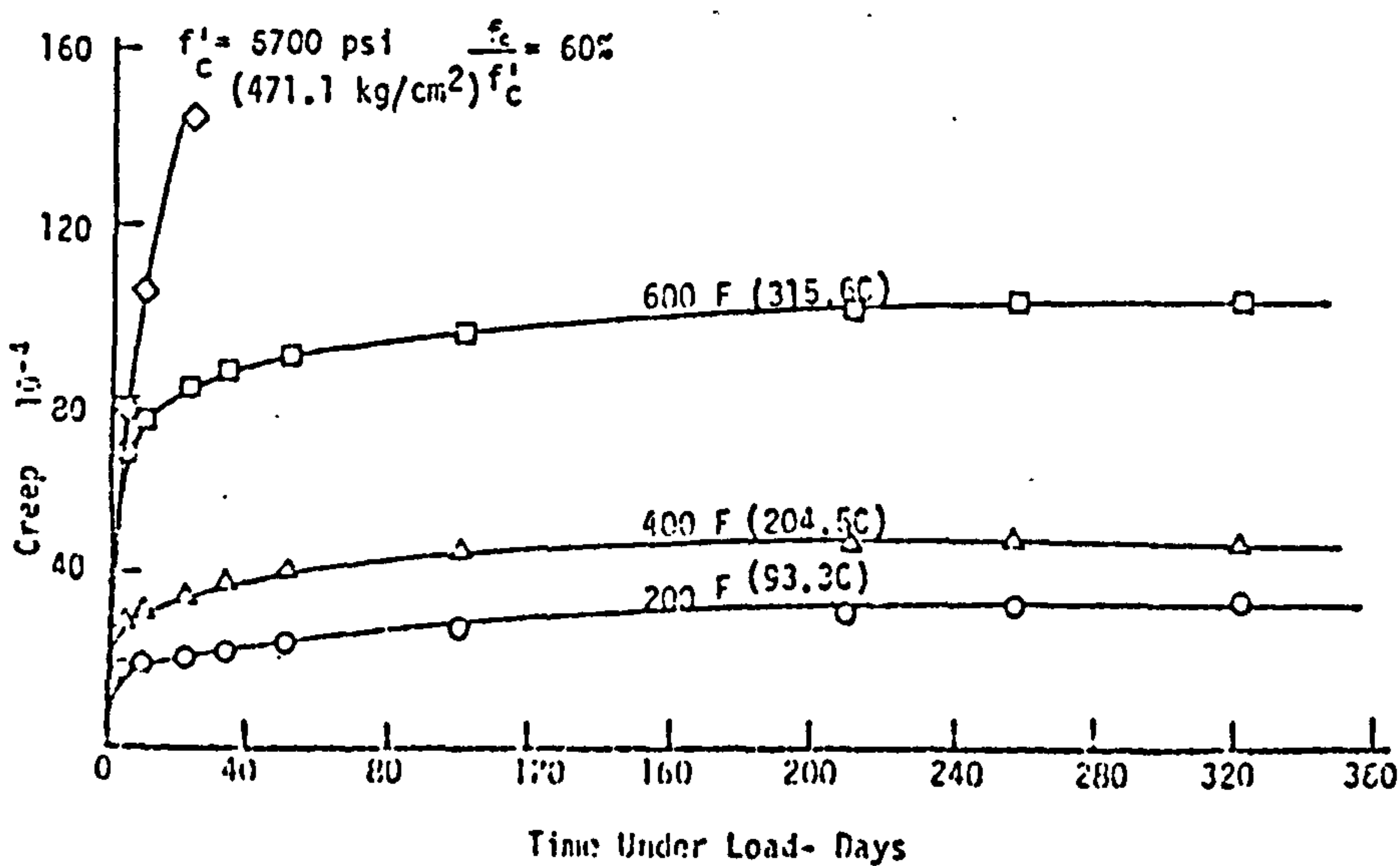


Fig. 2.9 -Relation between creep and time under load

(Uniaxial Results)

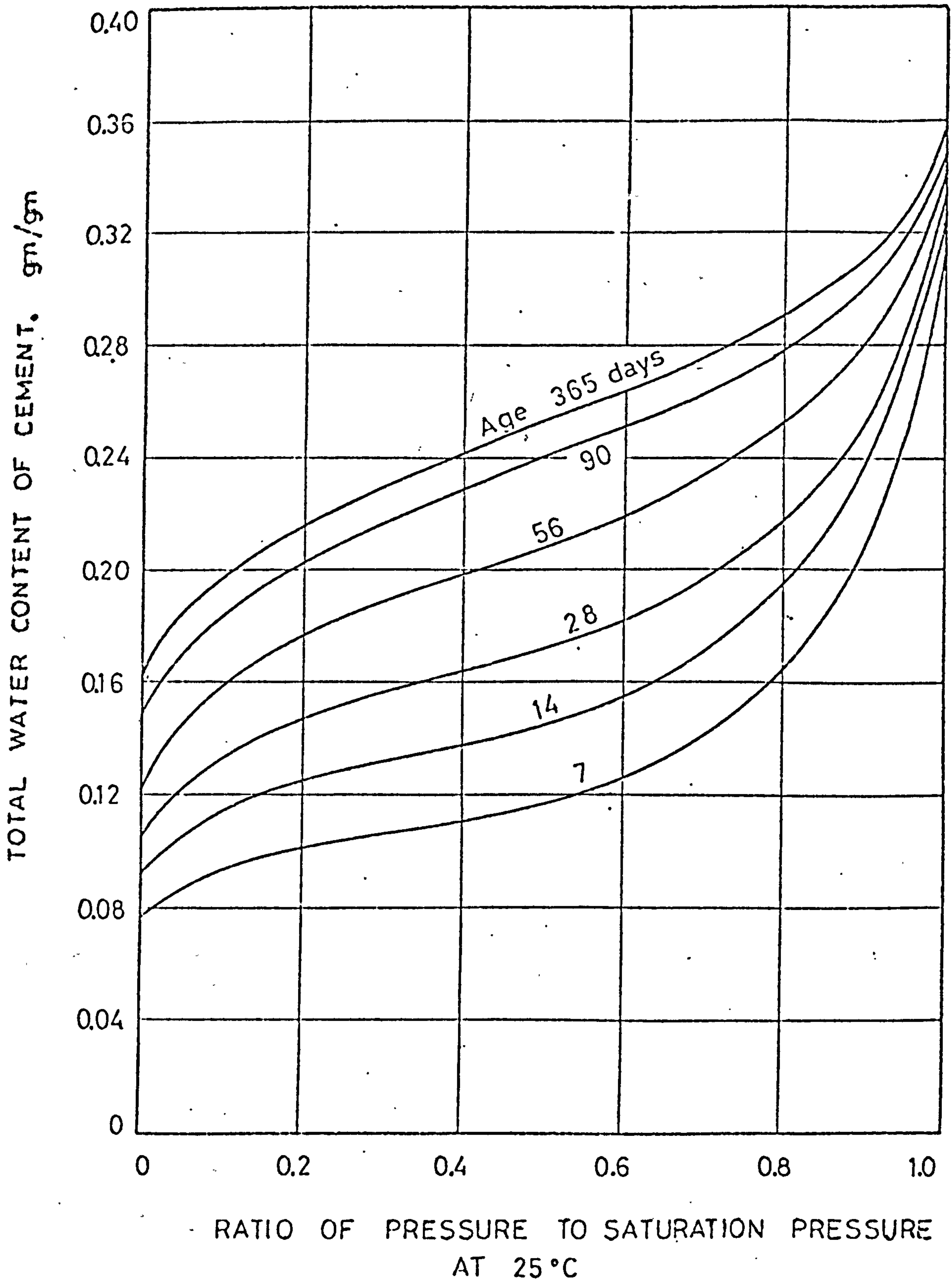
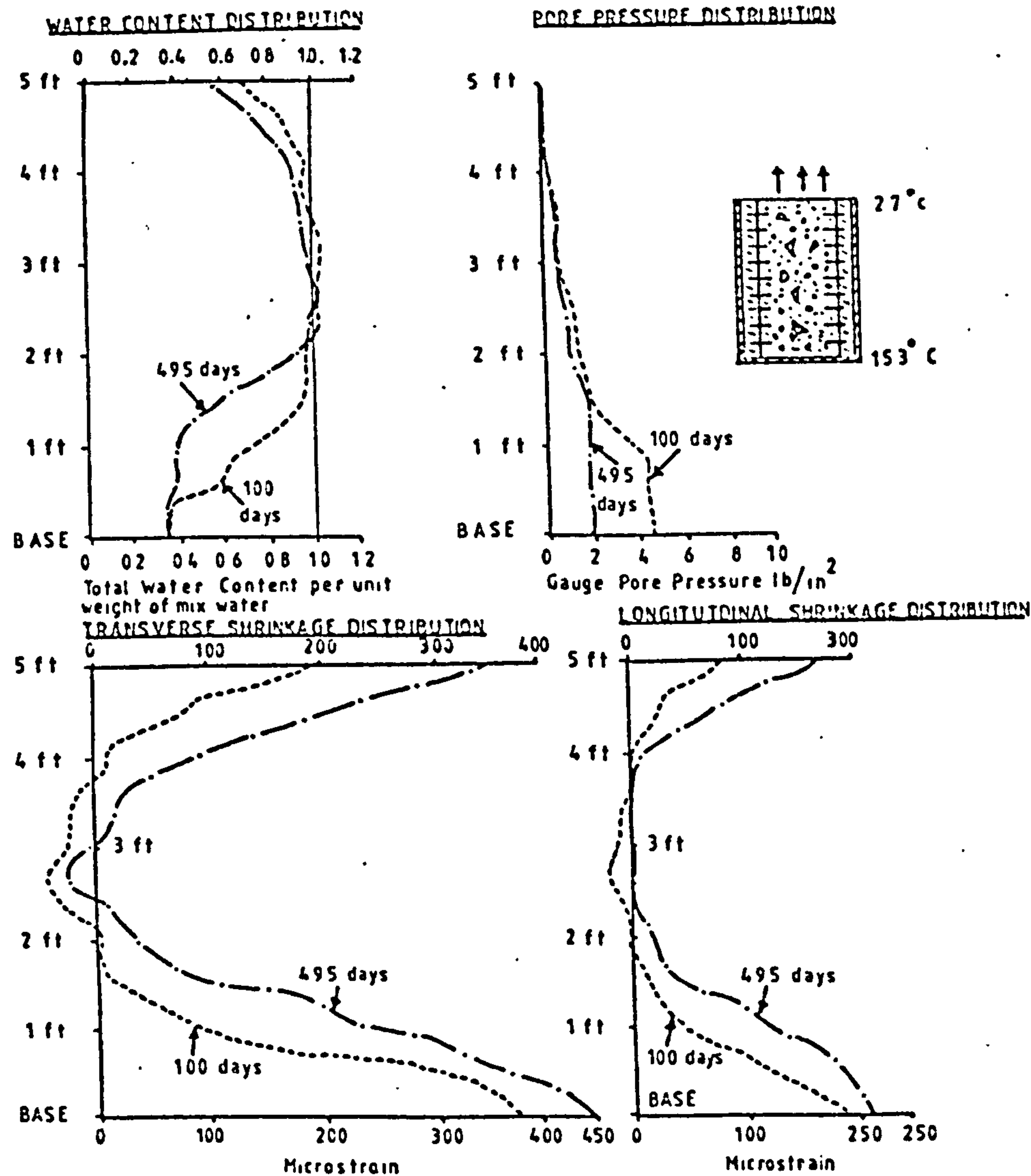


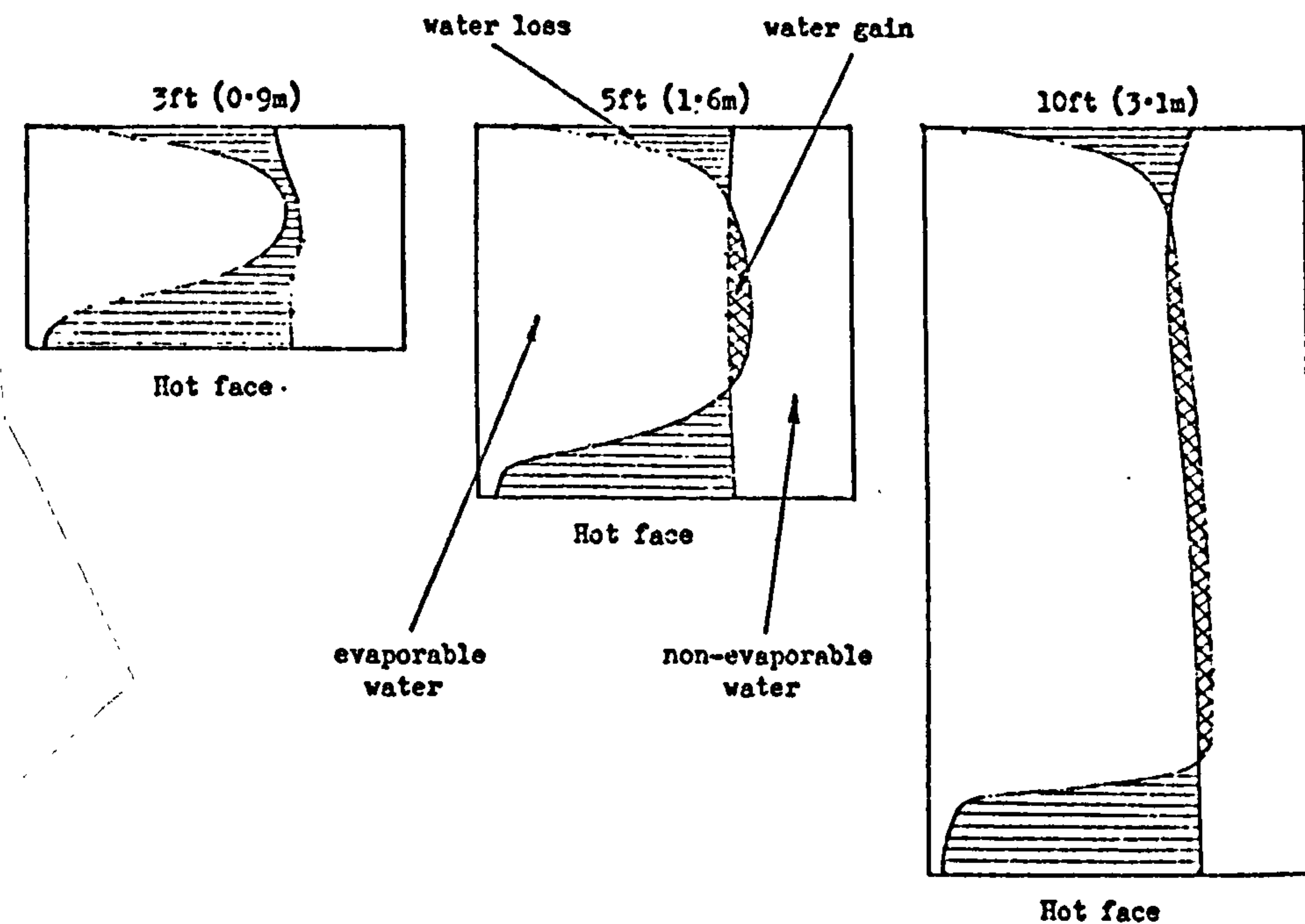
Figure 2.10 Effect of Curing on Adsorption Isotherms of Neat Cement Paste (from ref. 56)



WATER CONTENT, PORE PRESSURE, TRANSVERSE AND LONGITUDINAL SHRINKAGE DISTRIBUTIONS FOR THE SHRINKAGE SPECIMEN AFTER 100 AND 495 DAYS OF HEATING

Figure 2.12

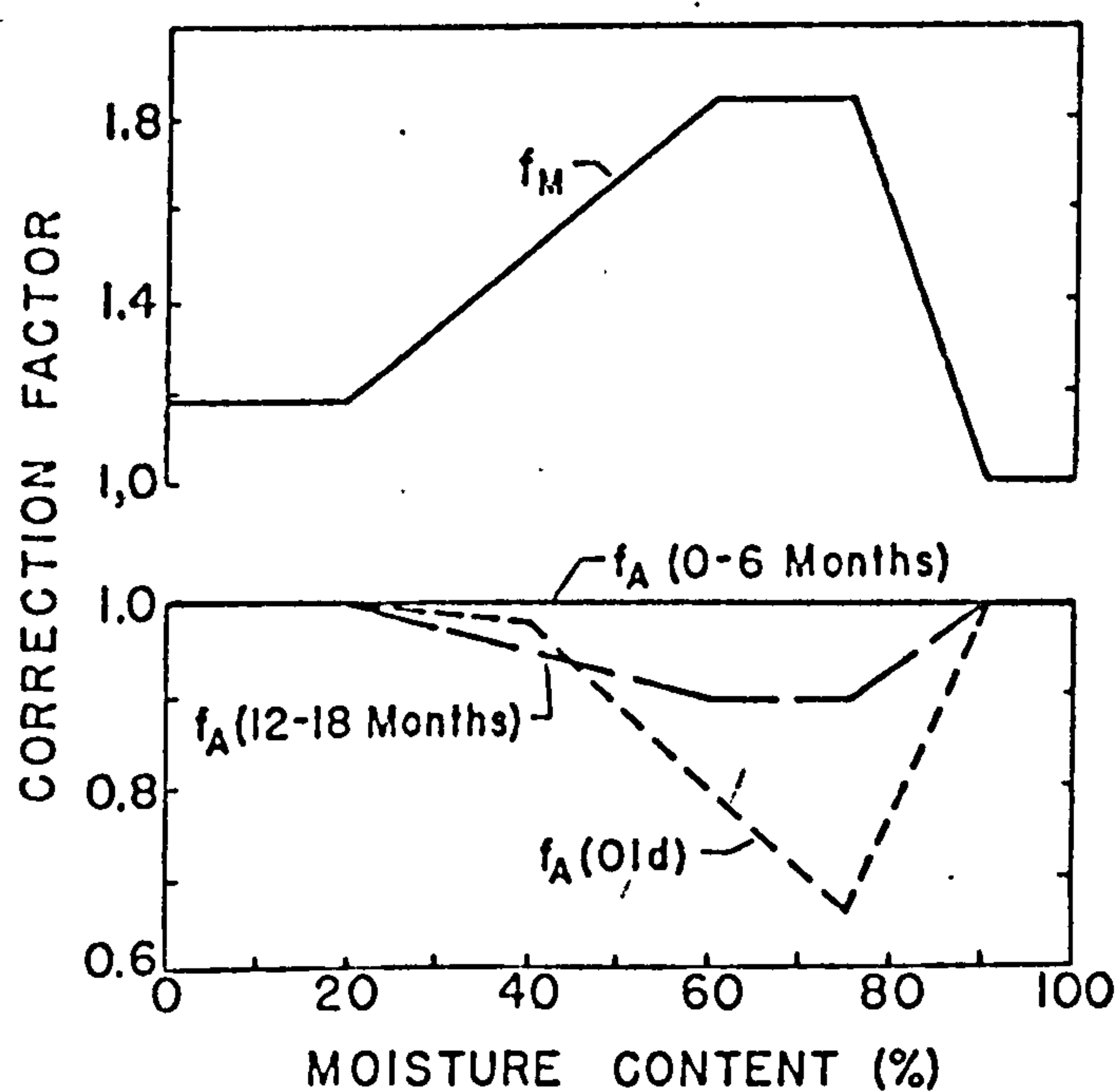
From ref. 81



Phase diagrams for water in concrete at 887 days. Specimens of various lengths and hot face temperatures of 125°C.

Fig. 2.11

From ref. 73



—Correction factor for moisture and for age

Figure 2.13.

From ref. 89

Table 2.2

ROUGH SURVEY OF THE PROPERTIES OF CONCRETE WHICH ARE INFLUENCED BY THE VARIOUS STATES OF WATER PRESENT IN THE CONCRETE (from ref. 58)

State of the Water in the Concrete	Mechanical Strength	Modulus of Elasticity	Shrinkage	Creep	Thermal Properties	Electrical Properties	Adsorption of Radiation	Resistance to Frost Action	Resistance to high Temperature
Water of Hydration. (non-evaporable water)	✓	✓							
Adsorbed and Capillary fixed Water			✓	✓					
Free Water					✓	✓	✓	✓	✓
Total Water					✓	✓	✓		

TABLE 2.3 AVERAGE COEFFICIENTS OF LINEAR THERMAL EXPANSION
OF ROCKS FOR NORMAL TEMPERATURE RANGES

Type of Rock of mineral	Average Coefficient $\times 10^{-6}$	
	Per F	Per C
Quartzite, silica shale, cherts	6.1-6.9	11.0-12.5
Sandstones	5.8-6.7	10.5-12.0
Quartz, sands, pebbles	5.6-6.9	10.0-12.5
Clay, mica shales	5.3-6.1	9.5-11.0
Granites, gneisses	3.6-4.7	6.5-8.5
Syenites, feldspathetic porphyry diorites, andesite, phonolite Gabbros, diabase, basalt	3.1-4.4	5.5-8.0
Dense, crystalline, porous limestones	1.9-3.3	3.5-6.0
Pure calcite	2.2-3.6	4.0-6.5
Marbles	2.2-3.9	4.0-7.0
Dolomites, magnesites	3.9-5.6	7.0-10.0

From Emanuel and Hulsey reference 89

Chapter 3

APPARATUS FOR LONG TERM MULTIAXIAL TESTING OF CONCRETE AT ELEVATED TEMPERATURES

3.1 General Remarks

In this chapter the principal techniques which form the basis of the actual design, are discussed. These deal mainly with various factors influencing testing technique particularly when stresses are increased up to failure at a temperature about 200°C , in the presence of radial pressure at the end of a creep test. To obtain sufficient and reliable data, to predict the influence of temperature on creep, strength and stiffness of concrete, all strain components in the test specimen should be measured. In order to obtain precise strain measurements, it is necessary to develop a reliable strain measuring system which can withstand prolonged exposure to 200°C .

To avoid toxicity and flammability hazards, it is necessary to develop a thermal system which satisfies safety measures and keeps the temperature controlled accurately throughout the period of testing.

The apparatus design is intended to be economic, simple and can be constructed from available commercial components with the exception of a few parts.

Various loading techniques used by previous workers have

been reviewed by Neville (19) or their details published elsewhere (95). Some of the techniques, while suitable in the research laboratory, may not be so in the conditions of a site or even in a commercial testing laboratory. The design constraints imposed by raising the applied load to ultimate at the end of each test to obtain the residual strength, virtually eliminates apparatus of the "nut-cracker" type on the grounds of the bulk and cost of the cantilever members. Similarly dead-loading is impractical. On the other hand the requirements of 200°C would also eliminate the use of conventional soil mechanic pressure cell on the ground of the deterioration of a rubber oil sealing jacket under a few days pressure and exposure to temperature about 200°C . These types of pressure cell have been used successfully in multiaxial testing at ambient temperature as well as at elevated temperatures below 100°C (1,2). See figures (3.1, 3.2). A new fundamental change has to be introduced to overcome the difficulties in carrying out long term multiaxial test at higher temperatures.

3.2 Load Maintenance Technique

Descriptions of long term testing apparatus are frequently concerned with how the load is applied and how it is maintained, both aspects often being intimately related. Creep of the specimen has relaxed the force being applied to it by the loading frame. The methods of overcoming this problem are discussed below and arranged approximately in order of increasing effectiveness. With some exceptions, the order is also that of increasing cost and convenience, though not necessarily of increasing cost per loading rig.

3.2.1 Low-Stiffness Loading Frame

The idea of using highly strained members in order to store energy is incorporated in several designs (10, 97-99). A thin-walled tubular dynamometer (10) has been used in compression in series with the specimen (or specimens), while small-diameter high tensile wires have been used as tension members as in post-tensioned concrete (97). This has the advantage that the load in the concrete can be monitored via the highly stressed members, but suffers the disadvantages of needing frequent adjustment and being awkward to adjust accurately. See Figure 3.4.

3.2.2 Hydraulic Loading

An elementary hydraulic system appears to offer numerous advantages and has been used in a straight forward way by several workers (97, 98, 100). Substantial stresses can be applied quickly and uniformly to the specimen using a hand pump and a pressure gauge of moderate accuracy. The problem of a non-leaking, low-friction jack can be overcome by means of the flexible diaphragm arrangement (97, 101, 1, 103), discussed below, or by using a commercially available flat-jack. The latter method is an economical arrangement and may have a better performance particularly at elevated temperature.

The use of a flexible-diaphragm jack or flat-jack is a most important development in concrete creep rig designs. Its advantages should be considered in designing new apparatus, especially when substantial forces have to be applied quickly.

A typical form of the flexible-diaphragm jack is shown in figure (3.3). The diaphragm is clamped to the retaining ring by a number of high-tensile bolts. The ring also serves to limit travel of the piston and diaphragm. Since the required travel of the jack is generally less than a few millimeters, the diaphragm easily follows the piston.

It should be noted that without an auxilliary accumulator or other stability arrangement hydraulic systems are sensitive to creep of the specimen and oil leakages. Methods of hydraulic stabilization are considered further below.

3.2.3 Spring Loading

Apparatus in which one or more coil springs are placed in series with the specimens have been in use for many years (105-108, 114). The arrangement has the advantage of long-term reliability and of requiring relatively infrequent load adjustment. In several respects, however, the method suffers by comparison with hydraulic systems. Greater attention must be given to ensure uniformity of stress, special jacking arrangements are needed to apply the stress in a reasonable time, load measurement and adjustment is less simple and accurate, and the apparatus becomes quite bulky when high loads are to be applied.

3.2.4 Gas-Operated Loading

In conjunction with flexible-diaphragm jacks, some workers (101, 103, 104) have used bottled nitrogen gas, instead of oil, as a readily available source of high pressure. Two

methods of operation have been reported. In the first (10), a reducing valve maintains constant operation pressure to all rigs in use at a given time. In the second (104), the gas lines to each test rig are pressurised to the desired level and then isolated from the pressure source. The gas in the lines tends to stabilize the effects of creep of the specimen, but must be monitored and adjusted periodically. Gas-operated systems have most of the advantages of oil-operated, and are even more compact and clean. However, leaks in valves and pipe work tend to be prevalent and harder to detect. The most unfortunate and serious drawback is associated with the safety measures have to be observed with high-pressure gas; for this reason gas-operated systems can not be completely recommended for general use.

3.2.5 Regulated Hydraulic Loading

Many of the advantages of both gas and hydraulic operation can be combined and some disadvantages eliminated by use of a regulated hydraulic supply. Several systems have been constructed in conjunction with flexible-diaphragm jacks and simple reaction frames (109-113, 103).

One scheme is shown in figure (3.5). Either may be constructed from commercially-available components. This scheme uses an electrically driven diesel fuel injection pump. The pump can supply oil at a low rate but a high pressure (27.6 MN/m^2) and is controlled by a precision pressure switch to operate within close limits. A gas charged bladder accumulator is placed in the line so that pressure fluctuations are mini-

mised and pump maintenance can be carried out. The scheme described by ref. (1) uses an air-driven hydraulic "air hydro" pump. This type of pump maintains a precise pressure ratio between the low-pressure driving air and the high pressure oil supply. In this way, a precision regulator in the air line permits the oil pressure to be controlled within $\pm 0.5\%$ over long periods of time.

Other methods of stabilization have been devised (93,95) which rely on auxilliary spring or dead-loaded pistons to control the pressure. Movement of the piston is used to control an oil pump.

The construction of a regulated pressure supply is probably not justifiable for a small number of simultaneous tests but becomes rapidly more attractive in terms of cost, time and space as the number of tests increases.

3.3 Factors Affecting Materials Tests

The main factors which influence the stress distribution in materials specimens and the consequent mode of deformation can be classified as (102);

A) End Conditions of Loading

1. Both ends effectively pinned.
2. Both ends effectively fixed.
3. One end effectively pinned, one end effectively fixed.

B) Testing Machine Characteristics

1. Operation of spherical seating.
2. Longitudinal stiffness.
3. Stability.
4. Ram effect.
5. Testing procedure.
6. Type and size of platens.
7. Lateral stiffness.
8. Alignment of Components.

C) Specimen Characteristics

1. Size and shape of specimen.
2. Non-homogeneity of material.
3. Accuracy of alignment.
4. Lateral stiffness.
5. Intrinsic weakness (mode of failure).

The manner in which the loads may be applied through the specimen ends and how this can be achieved in practice are first discussed. The various testing apparatus characteristics are then examined, followed by a discussion of the effect of the specimen shape and loading platens on the desired state of stress.

3.3.1 Pinned and Fixed Loading Conditions

The testing of a material specimen in uniaxial compression or tension involves loading it at its ends. Basically there are three methods by which the loads can be applied (115);

- (i) both ends of the specimen are effectively pinned,
- (ii) both ends of the specimen are effectively fixed,

(iii) one end of the specimen is effectively pinned, whilst the other end is effectively fixed.

In the first case, the loading is applied in a straight line between the pins, irrespective of the location of the specimen in the testing apparatus. Consequently, in non-homogeneous and/or misaligned specimens, elements within the specimen strain at different rates, thereby maintaining the centroid of resistance co-linear with the line of the action of the load.

In the second method, no tilting of the specimen ends during loading is possible. Thus, the specimen is deformed uniformly throughout, with the centroid of action of the machine being co-linear with the centroid of resistance of the uniformly deformed specimen.

In the third method of loading, any non-uniformity or misalignment will cause different straining of opposite sides of the specimen, which must result in bending of the specimen. This will produce a lateral movement of the pinned end relative to the fixed end, so inducing a lateral reaction from the apparatus.

3.3.2 Operation of Spherical Seating

Most conventional materials testing apparatus have a spherical seating or similar device located at least at one end of the specimen. In apparatus for testing concrete cube or cylinder compression specimens, the uppermost platen must be fitted with a spherical seating, the centre of which coin-

cides with the central point of the face of the platen (116, 117). The first function of such a seating is to allow the apparatus platens to bear evenly on the specimen even when opposite ends of the specimen are not geometrically parallel. During the loading of the specimen, the seating performs a second function when it can tilt freely, thereby being effectively pinned, or locked thus becoming fixed, or attaining a condition between these extremes.

That an effectively pinned or fixed condition can be achieved in practice has been shown by Sigvaldason (118) from coefficient of friction tests on different spherical seatings. With a very high degree of machine finish at the seating interface (4 to 8 microinch), a large contact area ensuring low average pressures, a minimum of boundaries or edges and a soft grease or low viscosity oil, the coefficient of friction of a seating can be reduced to virtually zero.

3.3.3 Longitudinal Stiffness of Testing Apparatus

The loading of a specimen in a testing apparatus results in deformation of both the specimen and the apparatus components. The actual deformation of the specimen is less, therefore, than the total applied deformation of the frame.

A testing apparatus is defined as soft when the total elastic deformation of the apparatus is larger than that of the specimen, or as hard when the elastic deformation is of the same order of magnitude or smaller than that of the specimen (125).

Tests by Wright (119) and Sigvaldson (120) indicate that the ultimate strengths of concrete specimens are independent of the longitudinal stiffness of the testing apparatus. However, for control testing purposes, the longitudinal stiffness should be large enough to prevent explosive failures occurring with the consequent deterioration of components and loss of accuracy from repeated impact loadings. A minimum longitudinal stiffness of 10×10^6 lb./in. has been recommended for the control testing of 6 in. concrete cubes.

3.3.4 Stability of Testing Apparatus

Chilver (121) has shown that buckling of both tension and compression apparatus can occur when specimens are loaded with both ends effectively pinned. In a compression test, a small lateral movement δ of the top cross-head, encastre with the apparatus columns, will result in small changes in length of both the specimen l_s , and the apparatus columns l_m . When $l_s < l_m$, the change in length of l_s will be greater than that of l_m resulting in a reduction of the deformation and load on the specimen. As the system will assume a condition of least energy, the cross-head will move laterally by an amount which increases with increasing load.

Equilibrium of the composite system is ensured if the lateral resistance of the apparatus balances the lateral component of the force P in the specimen. If it is assumed that the lateral component of P is resisted by n columns of the apparatus of uniform flexural stiffness EI , the testing apparatus may buckle about the specimen at a load stage given by the equation

$$\frac{l_s}{l_m} = 1 - \frac{\tanh \phi}{\phi} \quad (3.1)$$

Where

$$\phi^2 = \frac{Pl_m^2}{4nEI} \quad (3.2)$$

The right hand side of Equation 1 is always less than unity, and consequently instability is only possible when $l_s < l_m$. As l_s approaches zero, instability will occur at a lower load, and this has been confirmed by the difficulties encountered in the testing of short struts (121).

3.3.5 Rate of Loading

The deformational behaviour of almost all materials is very dependent on the rate at which the load is applied. In modern testing apparatus, pacing devices are provided which help the operator to control the loading rate. However, operation of the automatic axial load servo-control system is extremely difficult to initiate, and involves a lengthy and intricate process of pressure balancing. The process takes a number of minutes to complete and results, in some cases, in relatively large increases of specimen strain under constant hydrostatic stress. These are particularly evident at high pressures for concrete exhibiting high creep rate (94).

In the design of a simple regulated hydraulic system, it may be improved if testing apparatus are provided with,

- 1) non-return valve after the pump,
- 2) stop valves close to the loading systems to

- ensure that a predetermined pressures are reached within the pressure circuit prior loading,
- 3) a very fine control valves which allow the fluid flow to be more easily regulated (102),
 - 4) a large ram area which ensures low working pressures in the fluid and a large flow through the control valve (102),
 - 5) accurate load-measuring system.

The way in which the load is applied and measured should be carried out precisely and not be subject to random variations. The accuracy of testing apparatus and the precision with which a load may be read is described in B.S. (124).

3.3.6 Effect of Apparatus Platens, Packing Materials and Specimen Shape on the State of Stress in Specimens

The loading of any material specimen in uniaxial compression through large apparatus platens produces a complex and indefinable state of stress in the ends of the specimen (123). The actual state of stress causing failure will depend on the type and size of apparatus platens and packing materials and the shape of the specimen. In general, two distinct loading effects in the ends of uniaxial compression specimens must be considered; (i) lateral stresses which are induced in the direction normal to the applied load due to differences in the lateral deformations of the material specimen and the apparatus platens; (ii) the loading distribution in the axial direction which is seldom uniform.

**PAGE
NUMBERING
AS ORIGINAL**

- (iii) locally within the surface of the specimen, due to imperfections in the specimen or apparatus platen surfaces, or to the heterogeneous nature of the material itself.

The influence of the complex state of stress at the ends of uniaxial compression specimen on their mode of deformation and ultimate strength is almost entirely dependent on the ratio of the height of the specimen to its diameter or width. Whenever the applied loading system is not uniform, the lateral deformation profiles show a distinct "bulging-out" at the ends. These loading effects are confined mainly to the ends of the specimen and most of the irregularities die away and a uniform strain distribution is obtained at a distance from the ends equal to the width of the specimen. Short specimens, therefore, are subjected to a triaxial state of stress throughout their height, whereas long specimens have a central zone which is subjected to the desired state of uniaxial stress.

In order to produce a more uniform materials test in uniaxial compression the following recommendations are made; (94)

1. The material should be loaded so that no significant restraint stresses are induced at the boundaries. It is particularly important to ensure that tensile stresses are not present since these will tend to initiate failure from the end zones of the specimen, where the stress state is undefined, at a much lower level of applied stress. Thus, materials based on grease layers which may extrude under

load should be avoided, but some form of interfacial layer of a material which primarily 'beds-down' under load will be satisfactory.

2. The specimen should have a height/width ratio at least 2:5, if no attempt is made to reduce compressive restraint stresses at the platen/specimen interface.
3. It should be ensured that the loaded surfaces of the specimen and the surfaces of the loading device are plane and parallel. It is recommended that this be achieved by a process of grinding using an accurately aligned toolroom grinding machine with diamond abrasive wheel and suitable clamping devices.
4. The loading platen should be sufficiently stiff to prevent distortion which may lead to uneven loading. This is of particular importance if a spherical seating device is used to minimise the risk of eccentric loading, since such a device may tend to concentrate the load at the centre of the platen due to the distortion and/or mismatch between the male and female components. It is also important to ensure that the seating unit locks under load so that a state of uniform strain is applied throughout the loading history despite the degree of anisotropy of the material (96).
5. The loading platens should be at least as large as the specimen cross section, and preferably of the same nominal

cross-sectional dimensions as the specimen to be tested, as platens of a smaller than the specimen induce stress disturbances.

6. Any deformation measurements should be made with the central third of the specimen, where the stress state is uniform and relatively definable.

3.4 The Proposed Apparatus for Long Term Multiaxial States of Stress

Numerous studies of behaviour of concrete under multiaxial states of stress have been made in which various types of specimen and special loading devices have been used. Most of these studies can be divided into two categories, depending on the nature of the state of stress generated in the test.

Studies concerned with compressive cylindrical states of stress fall into first category, and are characterized by the following:

- 1- All principal stresses are compressive.
- 2- Two components (lateral) are equal in magnitude.
- 3- Cylindrical specimens are used.
- 4- Pressure cells are utilized for lateral compression.

Problems in controlling leakage with increasing pressure level and controlling pressure with the volume change caused by ingress of the piston into the pressure cell have been encountered, but these problems have been overcome in a variety

of special devices. A more difficult problem may be the penetration of surrounding fluid into the pores of the specimen after puncturing the sealing "jacket".

The second group of studies is concerned with the general multiaxial states of stress, usually utilizing prismatic specimens (cubes, prisms, slabs, or cruciform) and orthogonal loading devices. The advantage of developing three independent components of stress is offset by the non-uniform stress and strain fields caused by the discontinuities at the corners of the specimen and by the difficulty of minimizing platten restraint on the loaded faces of the prism. For tests on sealed specimens there is the further difficulty of adequately sealing a prismatic specimen without additional mechanical restraint. Another sub-group of these are tests utilizing hollow cylinders subjected to lateral pressure combined with axial loading.

Because concrete is used primarily in compression, the majority of studies focus attention on compressive states of stress. Also, because loading, strain measurements and observation of failure are greatly complicated in specimens subjected to elevated temperature, almost all of the studies of behavior under multiaxial states of stress have been carried out at normal temperature.

The general range of stress and temperature which should be examined for concrete in nuclear reactor structures are shown in table 3.1

Table 3.1 General Multiaxial State of Stress Conditions

Stress Ranges			Temperature Range
σ_1	σ_2	σ_3	$^{\circ}\text{C}$
$f_t^1 < \sigma_1 < f_c^1$	0	0	$25 \leq T \leq 175$
$f_t^2 < \sigma_1 < f_c^2$	0	$f_t^2 < \sigma_3 < f_c^2$	$25 \leq T \leq 175$
$f_t^3 < \sigma_1 < f_c^3$	$f_t^3 < \sigma_2 < f_c^3$	$f_t^3 < \sigma_3 < f_c^3$	$25 \leq T \leq 175$

Subscripts c and t denote compression and tension, respectively and superscripts 1, 2, 3, denote uniaxial, biaxial, and triaxial strength, respectively. Compression is considered positive.

If a study were carried out on specimens subjected to a cylindrical state of stress the conditions which could be investigated would be limited to those shown in table 3.2. The principal limitation resulting from the use of cylindrical specimens is the elimination of biaxial and triaxial tension states. It is, however, possible to investigate uniaxial tension combined with lateral compression.

The choice of the type of test, whether for a general or cylindrical state of stress, must be determined by the consideration of the difficulties encountered experimentally in each type of test. In the study considered here it was thought that the advantages of using cylindrical specimens were greater than the limitations imposed by this choice.

Table 3.2 Cylindrical State of Stress Conditions

Stress Ranges		Temperature Range
σ_1	$\sigma_2 = \sigma_3 = \sigma$	$^{\circ}\text{C}$
$f_t^1 < \sigma_1 < f_c^1$	0	$25 \leq T \leq 175$
0	$\sigma_2 < f_c^2$	$25 \leq T \leq 175$
$f_t^3 < \sigma_1 < f_c^3$	$\sigma_2 < f_c^3$	$25 \leq T \leq 175$

The proposed apparatus is limited to the range of stresses and temperatures shown in Table 3.3. But the apparatus could be modified without difficulty to cover tension-compression and wider ranges of temperature and stress.

Table 3.3 Apparatus State of Stress Conditions

Stress Ranges		Temperature Range
σ_1 psi	$\sigma_2 = \sigma_3$	$^{\circ}\text{C}$
$\sigma_1 \leq 12000$	0	$20 \leq T \leq 200$
0	$\sigma_2 \leq 1000$	$20 \leq T \leq 200$
$\sigma_1 \leq 12000$	$\sigma_2 \leq 1000$	$20 \leq T \leq 200$

This system has the following components:

- 1- Longitudinal compressive load system,
- 2- Lateral loading system,
- 3- Strain measurement,
- 4- Pressure circuit,
5. Thermal system.

The design for these systems are discussed below.

3.4.1 Longitudinal Compressive Loading System

The general requirements which have been discussed previously, include a stiff, stable frame and a means of applying the compressive load. The frame used satisfied three main criteria:

1. The frame is capable of carrying the maximum load expected during testing. With triaxial compression stresses in which confining pressure is limited to 1000 psi., the ultimate stress at elevated temperature could reach 10000 psi., and therefore a 6 in. diameter cylindrical specimen the maximum load is estimated at 130 tons.
2. The frame has 25×10^6 lb/in. $(\frac{4ES}{l})$ longitudinal stiffness, this stiffness is sufficient to prevent premature failure associated with "instability" resulting from release of elastic energy stored within the specimen upon reaching incipient failure, or release of elastic energy stored in the machine when its overall stiffness approaches that of the specimen.
3. The frame is stabilised laterally by four 3 in. diameter load-bearing columns.

Details of the frame and the axial loading are shown in figures 3.6-3.9. The frame consists of two plates 22 x 22 in. from mild steel 3.5 in. thick and with their faces machined flat. The plates are held in parallel position by four 3 in. diameter columns.

The axial load is applied to the specimen by a circular "Freyssi" flat jack (type 42) acting on a steel thrust plate or circular ram 14 in. diameter and 2 in. thick. The Freyssi flat jack is a hydraulic capsule jack in the form of a flat double saucer. It is made of special steel welded and heat-treated to produce the required internal pressure strength and flexibility. When inflated hydraulically, the upper and lower plates are forced apart and it assumes a new shape, in doing so, the flat jack can exert an extremely large thrust force depending on the hydraulic pressure which can be raised which in normal service is up to 15N/mm^2 ($2,000\text{ lb/in}^2$). This is equivalent to 160 tons total load for the one being used. The flexibility of the jack plates enable the ram to rotate during testing, so that the axial load can be applied evenly on the specimen.

The ram and the thrust plates were machined and provided with locating pins and recesses to ensure correct axial alignment.

A load cell was allocated between the ram and the bottom platen. It was noted during trial tests that the load cell was affected by heat. Therefore to prevent drifting in the load measurements, a cooling system was designed to fit the load cell head. See figure 3.13.

The compression platens are 6 in. diameter 3.25 in. thick mild steel, machined flat and provided with locating pins. During triaxial testing, these platens function as a piston in the

pressure cell. They are shaped to allow for clearance around the pressure cell ends, so that no friction is expected, provided that the axial load is applied prior to lateral loading.

This system is economic, frictionless and avoids leakage, but it is not suitable for repeated loading.

3.4.2 Radial Loading System

This loading is provided by a pressure cell, Figures (3.10 - 3.12) which is capable of generating pressure up to 1000 psi. The independent pressure system is designed to provide the required pressure at various temperatures and is controlled independently of the axial loading. Four principal problems are encountered in the design of the radial loading system.

The first concerns the design of the seals between a pressure containment tube and the platens which can withstand high pressure at elevated temperatures, without excessive friction and without leakage. If the pressure cell body, top and bottom heads, and the loading platen are made of the same material (having the same coefficient of thermal expansion) no significant change in bore tolerance should accompany change in temperature. Various types of seal were considered as follows:

- a- "O" Viton ring or P.T.F.E. seal,
- b- "O" rubber ring reinforced with spring,
- c- pressure activated metallic "O" ring,
- d- "O" rubber ring lined either with copper or aluminium.

However, these seals are not leak proof with the temperature around 200°C and the leaked oil may reach the heating tape which is heated to a temperature well above the flash point of the oil being used which is unsafe. A leak proof joint can be provided by a fused and seam weld between a thin stainless steel jacket and end rings, figure 3.14. The latter process requires special equipment and expertise.

The second problem concerns the design of a special jacket which will not puncture at points of local discontinuities and will prevent leakage of fluid into the specimen. This jacket has to satisfy the following requirements:

1. It should be resistant to corrosion when subjected to steam pressure.
2. It should have low longitudinal stiffness in order to reduce stresses on the joint, the latter could be reduced significantly by forming a circular convolution near the joint.
3. It should be soft and plastic in nature, and bed down under lateral pressure into small local discontinuities and by bridging over small gaps prevent stress concentrations and so prevent puncturing.
4. It should provide minimal radial restraint of the specimen, this could be accomplished by forming vertical convolution.

The appropriate choice of material and thickness is critical and can be determined approximately by analysis (see

Appendix A1). The specimen may contain approximately 7% by volume of evaporable water which upon heating above 100°C will be transformed into steam and will fill available pore space. If the specimen is to be sealed effectively, the pressure developed in the pore system due to saturated steam at 200°C is about 65 psi above atmospheric pressure. The jacket must resist this pressure and prevent expulsion of moisture; for various reasons discussed above a stainless steel jacket 0.01 in. thick has been selected. This jacket can withstand 1000 psi lateral pressure, and normally operates within the plastic range, either in compression or tension, however, the operating limits are substantially below the ultimate strain which is 400,000 micro-strain. This type (B.S. 1449, 320-S16) of stainless steel enjoys unique ability to deform with high resistance to fracture. The surface of the concrete specimens has to be ground smooth before inserting the specimen into the pressure cell to eliminate surface cavities which may cause puncturing of the jacket under pressure or leave permanent deformation of the surface of the jacket. The jacket is provided with 0.01 in. protective stainless steel in the vicinity of leak proof joint to limit deformation in that area.

The third problem concerns the control of radial pressure, due to the changing volume of fluid in the cell as the loading piston moves into the cell and as the specimen undergoes radial deformation. Since most fluids have a high bulk modulus (almost incompressible) even small changes in volume can cause large change in pressure. The solution is to control the pressure.

By means of a pressure accumulator designed particularly for this purpose and also to accomodate slight variation in temperatures which could affect the pressure (see 3.4.4).

The fourth problem concerns the extension of the probe rod out of the pressure cell. This must be done without allowing any leakage of the fluid, therefore a special device was designed for this purpose in which various types of seals were used together with a mechanical lock technique as explained in section (3.4.3.2)

The thin jacket, before pressure is applied, is about 0.02 inch clear from the concrete test cylinder, which is ground to fit when the jacket is pressurised, the inner tube is compressed until it contacts the concrete. To prevent friction against extraction, a high temperature lubricant- such as "ROCOL I,F.L." was used. It was noted that five vertical convolutions were formed and spaced almost regularly, therefore in future they should be induced to ensure that no clearance is left between the diaphragm and the concrete. Thereafter, in a test it acts as a flexible diaphragm, retaining the pressurised fluid and transmitting pressure directly to the concrete. This is the unique feature of the cell and avoids the use of material such as neoprene or of high-temperature seals which may develop leaks before a long term test is completed.

When compressed circumferentially to contact the concrete, the stainless steel tube may be strained slightly beyond the yield point, but at a stress well below ultimate value. The

radial pressure after contact can increase without further inward displacement of the diaphragm. The pressure at the instant of contact must be recorded, and the radial displacement gauges or transducers set at zero, or future readings adjusted for displacements before contact. Thereafter, in a test the radius of the concrete cylinder will increase either by the action of axial load or temperature expansion, or both. Normally, such displacements will be kept within a safe limit of change of circumferential strain in the diaphragm.

3.4.3 Strain Measurement

In selecting the strain instrumentation, it was decided that it is desirable to measure the average strain over a finite region rather than the "local" strain which may vary considerably due to non homogeneity over the region. This average strain over a gauge length several times the maximum aggregate size used in the concrete mix was selected because it is usually taken as a characteristic of the material to be used in structural analysis.

The characteristics of a gauge required to obtain the free thermal strain, elastic and creep strain data are as follows:

1. In order to measure average strain in concrete, the gauge length based on the $3/4$ in. maximum size of aggregate used in the mix should be preferable; 4 to 6 inches.

2. The shape, size and stiffness of these gauges must not materially disturb the strain field in the concrete. To accom-

plish this the lateral dimensions of the gauges must be small, and the effective stiffness similar to that of the concrete.

3. The gauge when embedded in concrete should have long term stability, that is its response to strain should remain constant over long time periods. This means that the materials in the construction of the gauge, such as the sensor, the housing, the leadwires (if any), adhesive, sealants and connections, should not be subject to creep and to corrosion, and should be either insensitive to moisture or sealed to prevent exposure to moisture. This latter requirement becomes particularly critical in the studies of sealed specimens heated above 120°C , as steam pressure may be generated in the concrete pores which further complicates isolation of the gauge sensor from moisture.

4. The gauge configuration should prevent movement between the gauge and the concrete, or at least the gauge ends should be so anchored so that the overall elongation of the gauge and the concrete within the gauge length are equal.

5. The strain range for the gauge should permit accurate determination of extension up to 2000 micro-strain, and of contraction up to 6000 micro-strain. These limits are based on estimated thermal and mechanical deformations.

6. Ideally the sensitivity of the strain gauge should be sufficient to allow readings to the nearest 1 micro-strain. The gauge,

when used with a properly designed circuit and recording equipment, should have an accuracy of ± 5 micro-strain over the desired range, which roughly is the response of concrete to a stress of about 10 psi or a temperature change of 0.5°C . Furthermore, the stability of the gauge and the recording system should be such that with repeated readings the reading should be reproducible within 10 micro-strain.

7. Because it is desired to record strains over a long period of time, it is particularly important to ensure long time stability of the measuring system. A random drift of 20 micro-strain at 100°C and 40 micro-strain at 200°C would introduce possible errors of measurement of 2-4 percent, which is acceptable. A larger drift with time and temperature which is systematic, that is predictable by calibration, may be acceptable, the requirement for data correction would make reduction of data more cumbersome.

8. The gauge should allow continuous recording, so that a continuous plot of stress-strain data may be obtained (by suitable recorders) on specimens subjected to continuous loading at a prescribed rate.

9. A linear response of the gauge is desired, that is the output should be linear within the entire strain range. If the response is non-linear, then its characteristics should be established by calibration.

10. The manufacturing technique for the gauge should be

such that the response characteristics of individual gauges do not vary significantly, in which case calibration of a few gauges would provide an accurate basis for data reduction from a large group of gauges.

11. Sensitivity of the strain gauge factor to variation in temperature within the range of the test program 20 - 200°C should be minimal. Preferably the gauge factor should not be sensitive to temperature and in any case the temperature sensitivity should be uniform for a group of gauges so that suitable corrections can be made on the basis of sample calibrations.

12. Response of the embedded gauges should be predictable from the response of the bare gauge, taking into account the coefficient of thermal expansion of the specimen.

A survey of commercially available strain gauges disclosed two promising types:

1. vibrating wire gauges,
2. specially encapsulated electric wire resistance gauges.

Vibrating wire gauges were eliminated from consideration at an early stage in this investigation, because of the following characteristics:

- a- strain range is substantially less than 6000 micro-strain considered desirable in this study,
- b- sensitivity of their gauge factors to temperature,

c- instability and possible failure when temperature rises above 150°C.

One of the most promising encapsulated gauges perhaps are the microdot embedment gauges 2,4 and 6 in. length. The sensitive element is a single ni-chrome alloy filament and it is housed in 0.04 in. diameter stainless steel tube where it is packed with MgO powder. The gauge can be temperature compensated for use in materials having different coefficients of expansion. Perforated metal discs are attached to the tube at the ends for anchorage in the embedment and the steel tube is embossed with indentations over its entire length to assist the anchor discs in the transfer of strain from the specimen to the sensor element. The nominal value of the gauge factor is given as 1.75 with $\pm 3\%$ tolerance from specified value, the strain range is given as 6000 micro-strain (with deviations from linearity of about 1%), and maximum elongation without failure is 20000 micro-strain.

These gauges are made with three types of integral lead wires. Among these types (CG 122) which has a leadwire consisting of three 28 AWG nickel clad solid copper conductors individually insulated with braided fibre glass and insulated in a stainless steel 0.093 in. diameter tube. This gauge (according to manufacturer's specifications) can operate in an environment of up to 260°C, and pressure of 1000 psi - without malfunction of lead wires.

Bertero et al (126) have carried out a number of tests on these gauges in which twenty-three of them were embedded in con-

crete cylinders and calibrated at different strain levels and temperatures. They concluded that for many individual gauges the discrepancy between bare (not embedded) and (embedded) gauge factors was considerable. For nine gauges the discrepancy exceeded ± 10 percent, and in one case exceeded ± 20 percent. There was no consistent pattern, some embedded gauges having values higher and some lower than bare gauges. In a number of calibrations, they noted that the gauge factor values showed scatter at different strain levels. It appears that the strain transfer from the concrete to the sensing filament exhibits "sticking" and "slipping" in the MgO packing which transmits the strain to the filament through friction.

Recently, Angeli et al (127) have carried out tests on a ring, representing a 1 m. high section of Austrian PCRV-Model, in which two types of strain gauges were installed in the insulating concrete; Resistance strain gauges of the type Microdot CG 122 with built-in thermocouples and fluidic strain gauges by BAM (128). It was noted that the Microdot CG 122 gauges failed at temperatures above 80°C due to penetration of moisture into the gauges and were brought back to use by installation of a vacuum system, despite the latter arrangement, the gauges again broke down at temperatures above 120°C and did not show any values which were within the range of the measuring device, later on resumed work at temperature around 240°C giving values very much higher than the actual deformation of the concrete. They concluded that the measured strain in the tangential direction at 100°C agree well within error limits, while at 250°C the measured strains are substantially higher than the calculated ones.

It seems that these gauges have to be improved greatly to obtain reliable measurements. Therefore, this led the author to design a strain measuring system for the axial as well as the radial deformation as explained below.

3.4.3.1 The Axial Strain Measuring System

The axial deformation is measured by two transducers rigidly attached to two 5/8 in. diameter steel rods. Each rod is fixed to a bottom plate which is welded to the bottom platen, the other end of each rod is passed through hole in the top plate which is welded to the top platen. The spindle of each transducer is positioned against the top plate. In this manner the transducers measure precisely the differential movement between the top and bottom platen (see fig. 3.11).

3.4.3.2 The Radial Measuring System

In the design of the radial measuring system, two schemes were considered. The first was using Sangamo Transducers of the sealed high temperature and displacement type. They are (according to the manufacturer's specifications) pressure sealed to withstand up to 3000 psi, able to measure displacements of up to 0.8 in. and capable of working temperature ranges of -50°C to 600°C . These transducers have to be fitted into the pressure cell cavity in such a way that leakage would be prevented. The latter is critical for various reasons. (see 3.4.2). This scheme is also expensive, therefore it was abandoned.

The second scheme is a device which can be fitted into the

pressure cell cavity and can transmit the radial displacement to an electrical transducer being mounted on a stand, 4 in. away from the cavity. This device has a probe rod in its centre and the radial strain is transmitted through the probe rod. The seals used at first in the device were P.T.F.E. packing which has a low friction drag on the probe rod. Unfortunately this device did not survive the pressure test at ambient temperature and started to leak from the very beginning.

The P.T.F.E. packing was then replaced with P.T.F.E. chevron ring seals designed to be more effective. See figure (3.16). This arrangement has a drawback in that the friction between the probe rod and the seals may influence the movement of the probe rod and reduce its sensitivity. The device was pressurized up to 1000 psi and inserted in an oven, the temperature raised steadily up to 150°C and kept at this temperature for twenty four hours. The oil pressure was monitored outside the oven by a "Budenberg pressure gauge" which has high accuracy, and was kept away from the heat effects. The pressure started to drop after a few hours continuously and the final reduction after 24 hours was 15%. This test clearly shows that the organic seals are unreliable at high temperature, therefore, attention was focused on the use of a metal bellows for a solution.

A metal bellows consists of seamless, thin walled, convoluted tube manufactured to very close limits to give it the properties of a compression spring which can withstand repeated flexing. In addition to axial flexibility, its seamless construction endows it with characteristics of a uni-directional

flexible container which can be hermetically sealed. This makes it a suitable medium for converting a change of pressure into an axial thrust and also allows it to act as a flexible seal for shafts.

Since a bellows has a small effective working area in the same way as a piston it will sense changes in pressure and respond with proportional axial deflection. The resultant movement may be utilised to actuate the transducer through the probe rod.

A coil spring is needed to keep the probe in contact and to supplement the spring rate of the bellows. It can be used not only to limit the deflection for a given pressure change, but also by pre-loading, to restrain movement until a pre-determined pressure has been obtained. This could be calibrated to the desired accuracy.

The design required to satisfy several requirements as follows:

1. The most important is that in no circumstances should a bellows be used in extension; it must always operate from its free length towards its solid length.

2. Bellows units should be so designed that they are readily detachable from the mechanism in which they operate, to facilitate both initial assembly and subsequent servicing. In addition, there should be no possibility of tips or roots of the convolutions rubbing on adjacent parts, as this will result in

early failure of the bellows.

3. Whenever possible it is preferable to use a bellows with a closed end. This reduces the number of soldered joints required, minimising the chance of leakage and saving the cost of a separate end fitting.

Two procedures were considered in the design of the bellows assembly, the first being to attach both ends of the bellows by weld. The weld was not only a more difficult operation to perform without annealing the bellows convolutions adjacent to the joint, but was also difficult to replace. The second was to use a bellows with closed end and to attach the other end fitting by a form of conical seating joint, the latter appears to be preferable, particularly at fairly high temperatures.

Extensive studies were carried out to incorporate the previous ideas, from which several designs have been suggested. Amongst these was the scheme shown in figures 3.19a-3.19d. The radial displacement is transmitted to an invar steel probe rod, pressed by a coil spring within an external mild steel case against the inside of the diaphragm. A flexible bellows with closed end surrounds the probe rod within the pressure cell. The probe rod within the bellows passes through a small hole in the outer tube of the pressure cell. The open end of the bellows is mechanically locked within a conical seating wedged tight, so that there is no leakage of the contained fluid. This scheme appears to satisfy the various requirements mentioned before and can be made with relatively low cost. Two of these

devices were made and built into the pressure cell at mid height.

3.4.4 Pressure Circuit

The load is controlled throughout the experiment by the pressure circuit shown in figures 3.20. It has four main lines, each of which can be controlled independently. The first is the loading line which provides both axial and radial loading system with the required pressure. It uses an air-hydro pump driven by compressed air from the laboratory's air circuit. The oil is supplied by reservoir placed on top of the pressure circuit. The second is for the axial loading system and it uses a Budenberg pressure gauge which has a high degree of accuracy and a 4 litre Fawcett hydro-cushion accumulator to stabilize and minimise pressure fluctuation. The third is for the radial loading system. This line is identical to the second except that the size of the accumulator is 2 litres. The size of the accumulator has to be proportional to the specimen deformation and the number of adjustments required. The accumulators were precharged by nitrogen gas to 90% of minimum working pressure. Under no circumstances should the working pressure exceed five times the nitrogen pressure, also the oil temperature should not exceed 80°C ; otherwise in both cases considerable damage to the accumulator's diaphragm may occur. The fourth is the unloading line which controls the release of the pressure from any line or reduces it to the desired level.

There are several valves inserted in the circuit which control the oil flow either to raise or to relieve excessive pressure, they also facilitate the dismantling of any part of

The circuit for maintenance or replacement when the test is going on.

The pressure circuit can be used for several rigs connected in series at the same time. It proved to be satisfactory not only to stabilize the pressure due to stress deformation but also for slight temperature variations.

3.4.5 Thermal System

It was suggested at the early stage of this research to fit heating tape into the annular space to be filled with silicone fluid. In this case, the most convenient heating unit would be a mineral insulated heating unit with proper seals and connections. However, the heating unit would operate at a very high surface temperature with the loading envisaged and the author estimated that the unit could run around $600-640^{\circ}\text{C}$ if it is allowed to operate uncontrolled at the power densities suggested, though, even if it is controlled, the surface temperature would rise up to $400-450^{\circ}\text{C}$ to supply enough heat, consequently the silicon fluid which would be necessary for such a system would decompose and a considerable risk of damage to the cell was likely to occur. Therefore, it was suggested using the external heating tape, type 7.1m KPG 150/240 V., non-moisture proof high-power with output loading 1000 W., flexible glass fibre insulated heating wrap, similar in appearance to braided rope with Aero Turbine Oil in the jacket for pressurising and heat transfer.

The required winding pattern to give a constant temperature

down the length of the specimen were determined from preliminary tests using "cast in situ" thermocouples at 88 mm. (3.5 in.) spacing small temperature changes measured by the use of the surface fixed thermocouples.

The heating tape was surrounded by a 25 mm. (1 in.) thick insulating jacket made by Thomas French Ltd. Heat losses from the ends of the cylinder were reduced by the insertion of 13mm. ($\frac{1}{2}$ in.) thick compressed asbestos (Sindanyo) discs, between the concrete and the loading platen and sindanyo rings around this platen. After trial the Sindanyo was placed outside the platen as the compression strain was much too high.

Voltage control was effected by variac constant voltage transformers (rotary regvolt) type 71A connected in parallel across the heating wraps from one side, and from the other side, connected to Pye Ether controller temperature type 1990B with thermocouple washer type. A thermocouple washer was fixed on the external side of the cell by a screw. Figure 3.21 shows the details of the heating circuit.

The temperature controller monitored the temperature of the cell and prevented overheating above the top range. The intention was, in fact, to use the temperature controller as an upper limit control to prevent temperature surges in the concrete as well as the oil due to malfunction of the variac.

3.5 Operation of Pressure Cell

In order to separate elastic and creep movements, tempera-

ture and shrinkage displacements are measured on an identical control concrete cylinder heated to the same temperature and with moisture movements similar to those of the principal test cylinder. Measurements are taken at each stage of the creep test. The spacing of the metal gauge points on the cylinders is measured before insertion into the cell.

The principal test cylinder is lubricated and inserted into the cell, so that a ground, smooth surface is opposite each probe point.

Heating, loading and unloading must be done in the following order:

1. Open all valves to avoid excessive pressure during heating.
2. Set the variac to 75 percent of the supply voltage (say 75 percent of 220 to 240 volts), in order to heat the concrete at a rate of about 5°C per hour.
3. Set the temperature regulator 10°C above the operating concrete temperature as a safeguard against over-heating.
4. When the operating temperature is attained, adjust the variac by trial to keep the temperature steady within $\pm 1^{\circ}\text{C}$.
5. To compress the cylindrical diaphragm against the concrete over the whole cylindrical surface, increase σ_2 in stages

to about 300 psi. Ensure that the σ_2/ϵ_2 diagram changes to a gradient approximately equal to E for concrete and that the flow of oil into the annular jacket ceases.

6. To set the recorder at zero, release the oil pressure to zero and compress the probe spring and ensure that radial displacement is zero for an increase of spring compression. The σ_2/ϵ_2 diagram between about 100 and 300 psi. should, when extended to zero, give $\epsilon_2 = 0$ when $\sigma_1 = 0$.

7. To avoid friction between the concrete and diaphragm when σ_1 is applied, apply σ_1 before σ_2 and note the resulting values of ϵ_1 and ϵ_2 . Apply σ_1 in about 2 minutes to reduce the amount of short-term creep influencing the value of E.

8. Apply σ_2 and note ϵ_2 and ϵ_1 . Check that the oil pressure on the closed end of the bellows does not exceed the spring pressure by increasing the spring pressure, if necessary, so that ϵ_2 is zero for an increase of spring pressure.

9. Record ϵ_1 and ϵ_2 daily and ensure that σ_1 and σ_2 and the temperature of the concrete, probes extensometer rods and platens remain constant.

10. On completion of the test, in order to extract the cylinder easily, pressure and temperature are released in the following order:

a- Radial pressure σ_2 ;

b- Axial pressure σ_1 ;

Note the axial and radial strains following release.

- c- Allow the concrete to cool;
- d- Extract the cylinder;
- e- Measure the axial and radial strain recovery from the spacing of the metal gauge points on the cylindrical surface which were measured before insertion into the cell.

11. Test the crushing strength of the cylinder and compare with an identical cylinder not stressed or heated, in order to determine the loss of strength.

12. After extraction, check that the position of any axial convolutions are clear of the probe points, unless these have been pre-formed for this purpose.

3.5.1 Safety

No parts of the apparatus or the non-inflammable oil should be heated above about 4/7 of the guaranteed minimum flash point (centigrade). The stainless steel diaphragm should not be strained in tension beyond 0.001 and special care taken after 25 reversals of stress, or if the axial crushing strength of the concrete is approached.

3.5.2 Significant Correction to be made to Strain Recordings

(1) Reduction of measured ϵ_1 , due to compression strain of steel platens.

3.5.3 Less significant Corrections (normally negligible)

(2) Radial resistance of steel diaphragm due to change of ϵ_2 and circumferential friction. Probably eliminated by deformation of convolutions at very small resistance. The resistance could be measured by applying σ_2 to the diaphragm alone.

(3) Error in zero datum for ϵ_2 and σ_2 due to (2).

(4) Local deviation from steady state temperature of platens, probe rods, and extensometer rods, due to changes of laboratory temperature.

(5) Deviation of measured ϵ_2 value, due to deflection of piston plate displacing transducers or due to displacement of a fixed reference.

(6) Deviation of measured ϵ_2 value, due to friction restraint of platens and distribution of σ_1 and σ_2 over the concrete surface.

(7) In future designs it is proposed to use 3 radial probes at 120° angular spacing in conjunction with reference points on a stiff freely supported ring surrounding the apparatus.

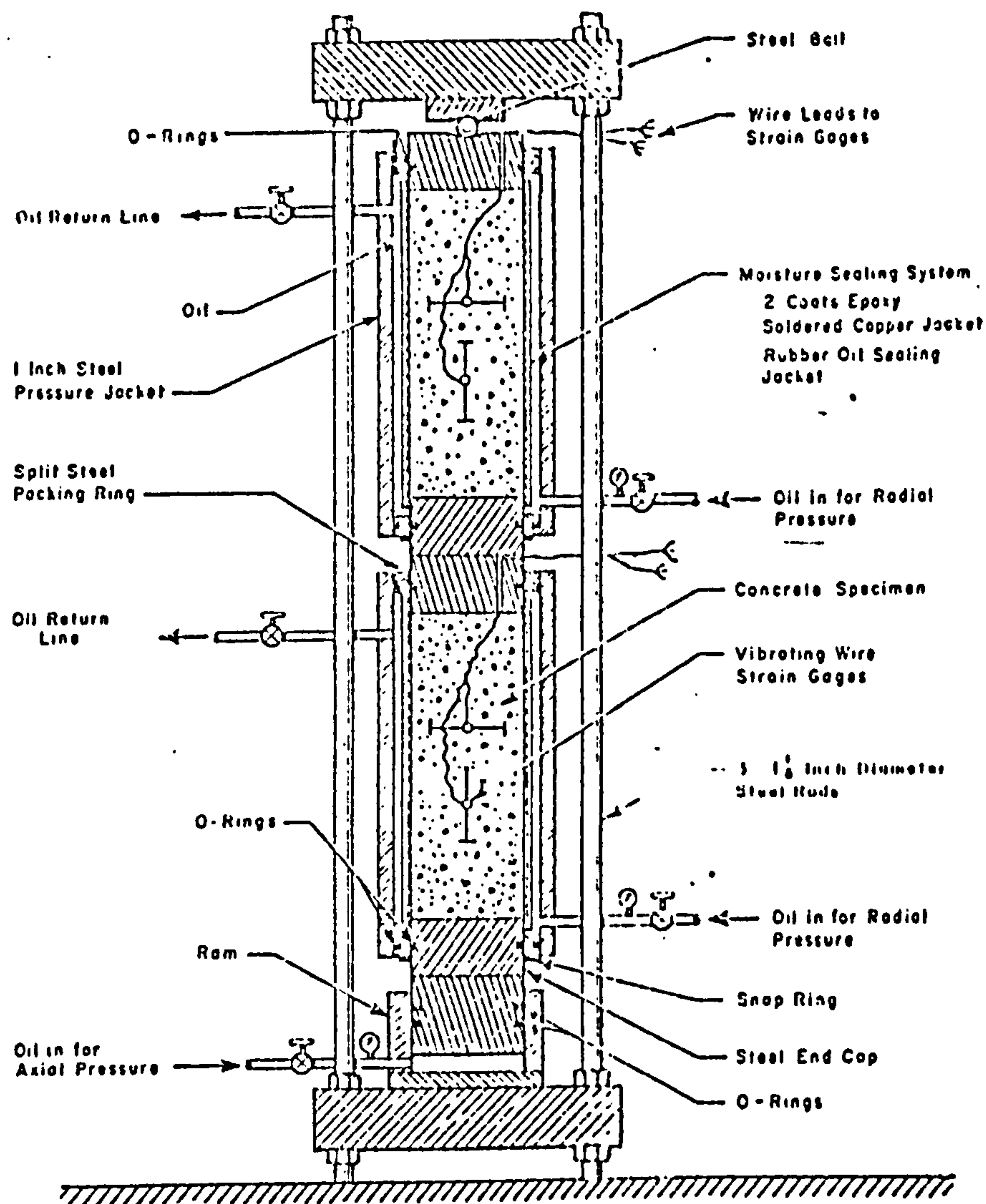


Figure 3.1 Triaxial Creep Rig for Temperature up to 65°C.

Ref. (2)

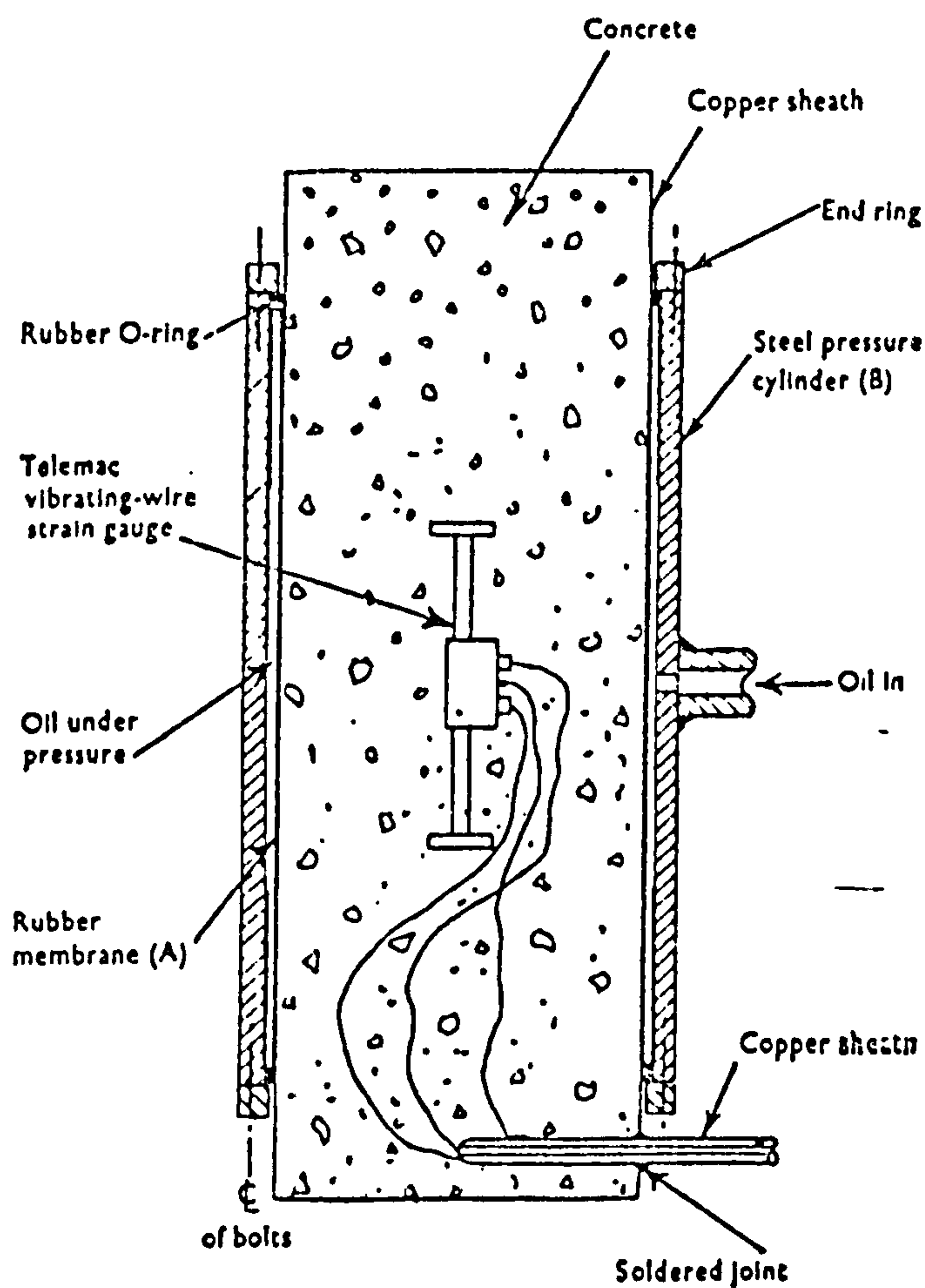


Figure 3.2 Pressure Cell for Temperature up to 95°C. Ref.(1)

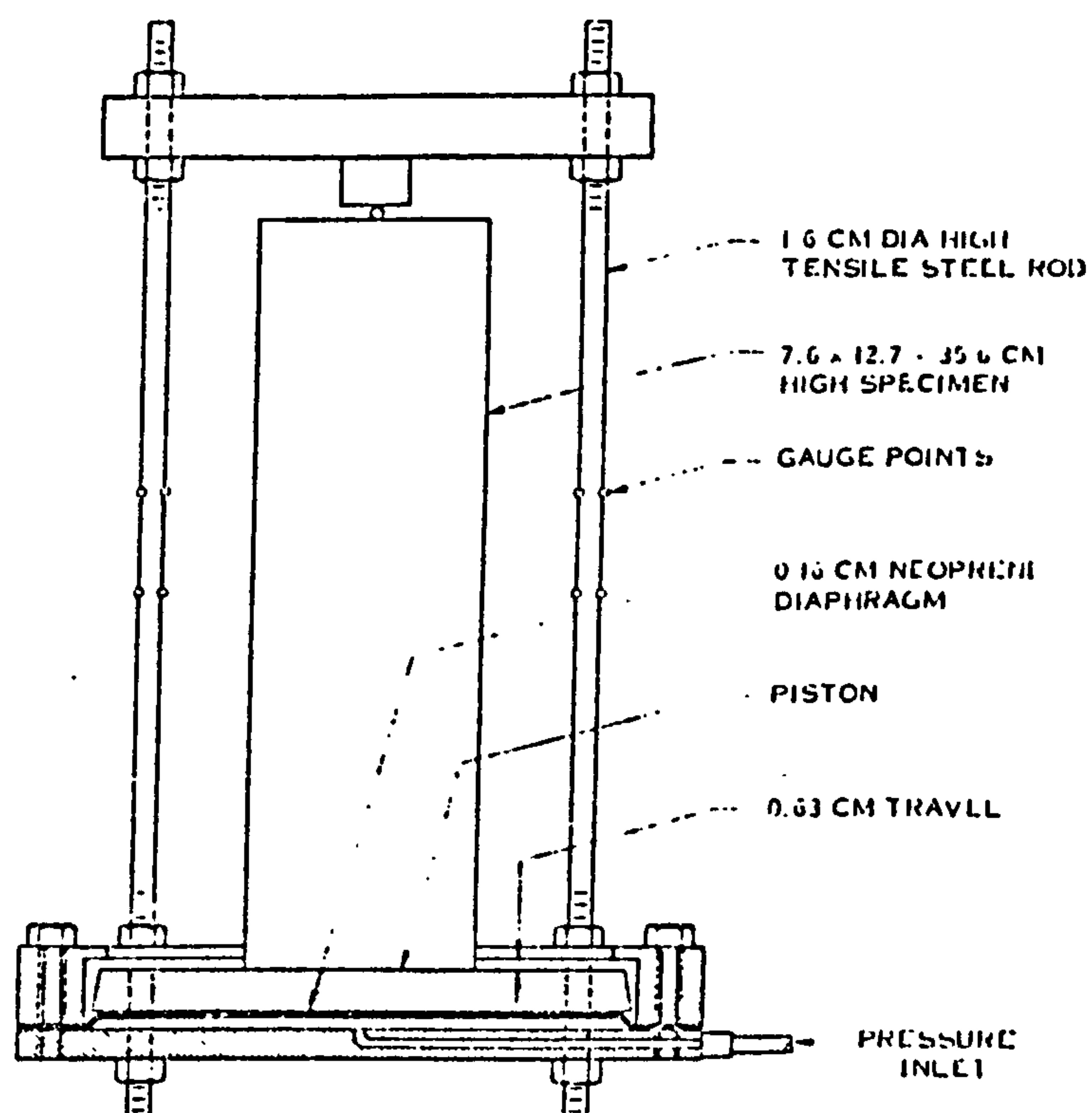


Figure 3.3 Uniaxial Creep Rig Relies on Flexible - Diaphragm jack Ref. (103).

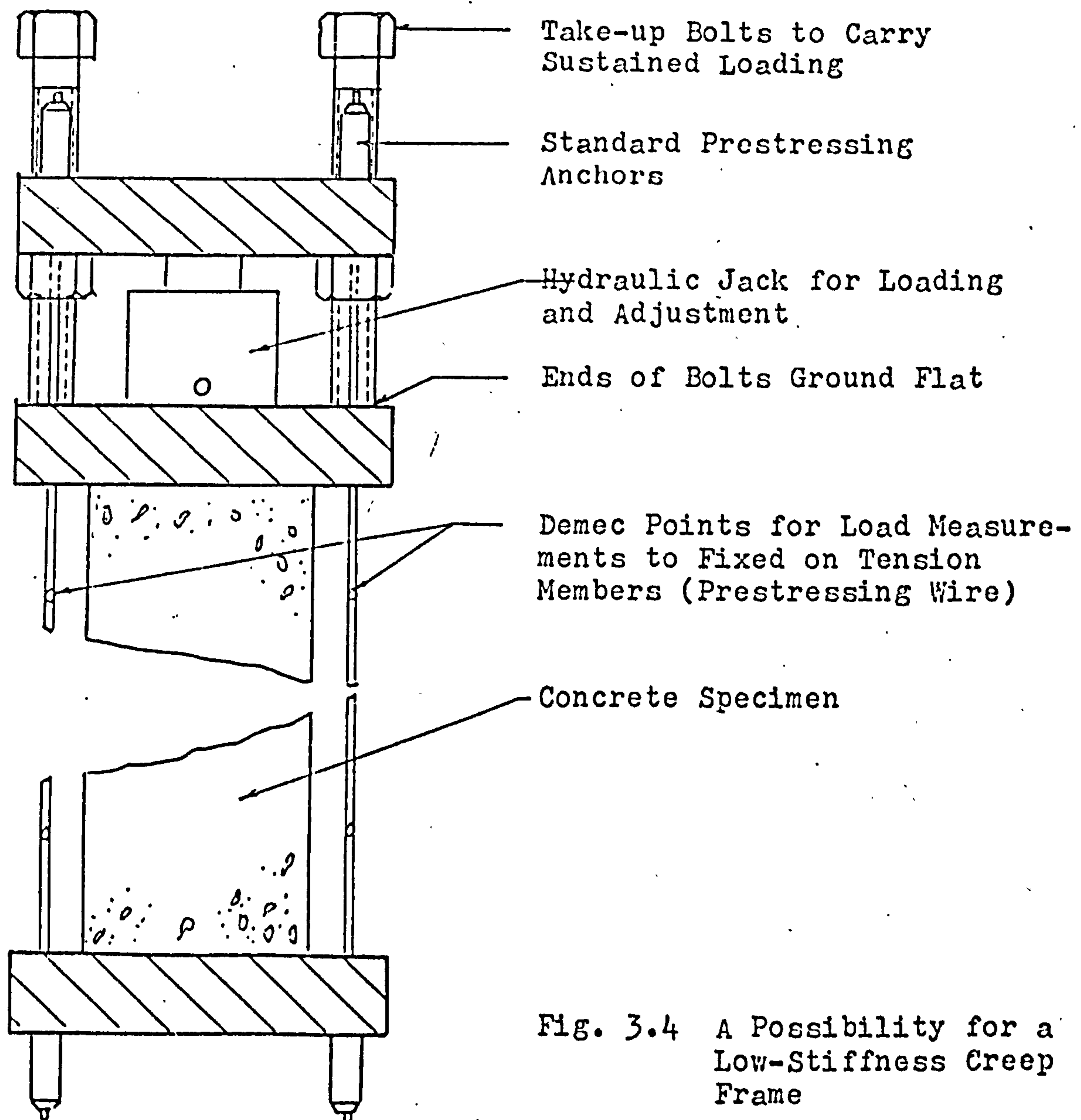


Fig. 3.4 A Possibility for a Low-Stiffness Creep Frame

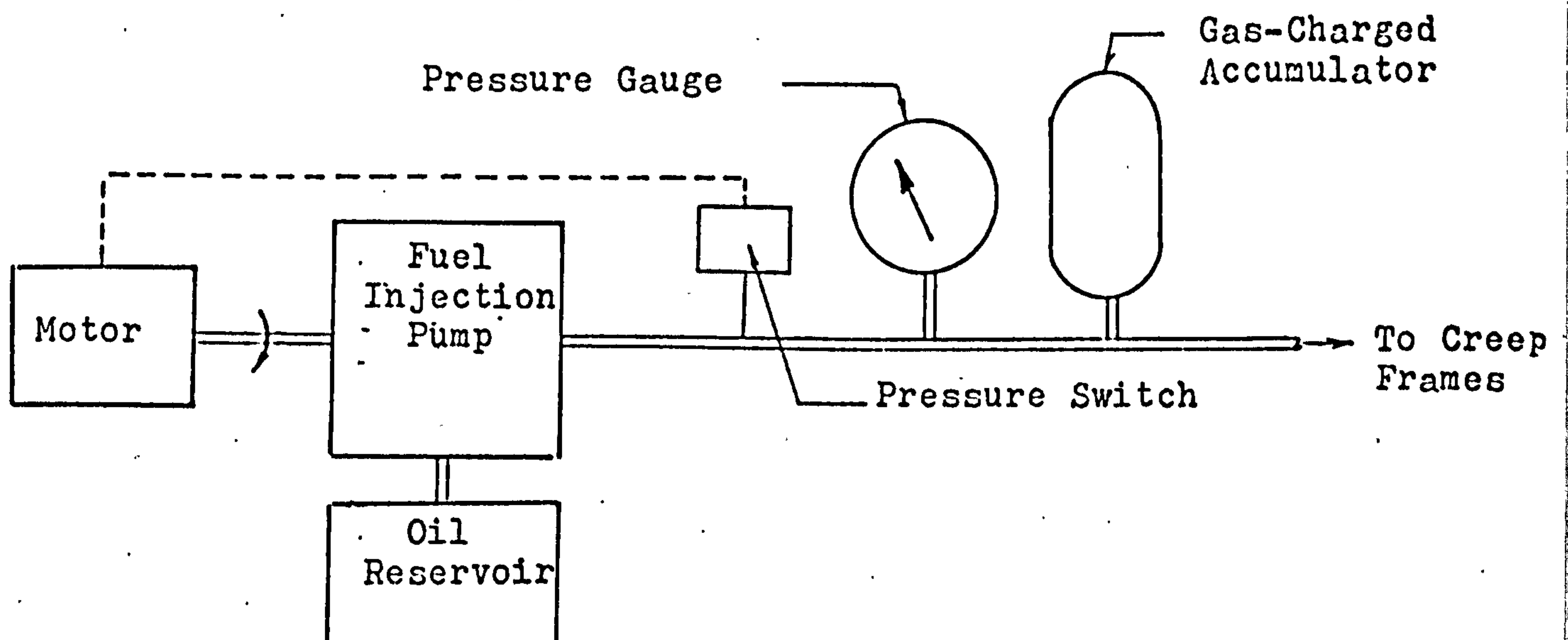


Fig. 3.5 Regulated Pressure Supply

- ① concrete specimen
- ② pressure cell
- ③ insulation
- ④ loading ram (piston)
- ⑤ flat jack
- ⑥ radial probe rod compartment
- ⑦ axial measurement rod

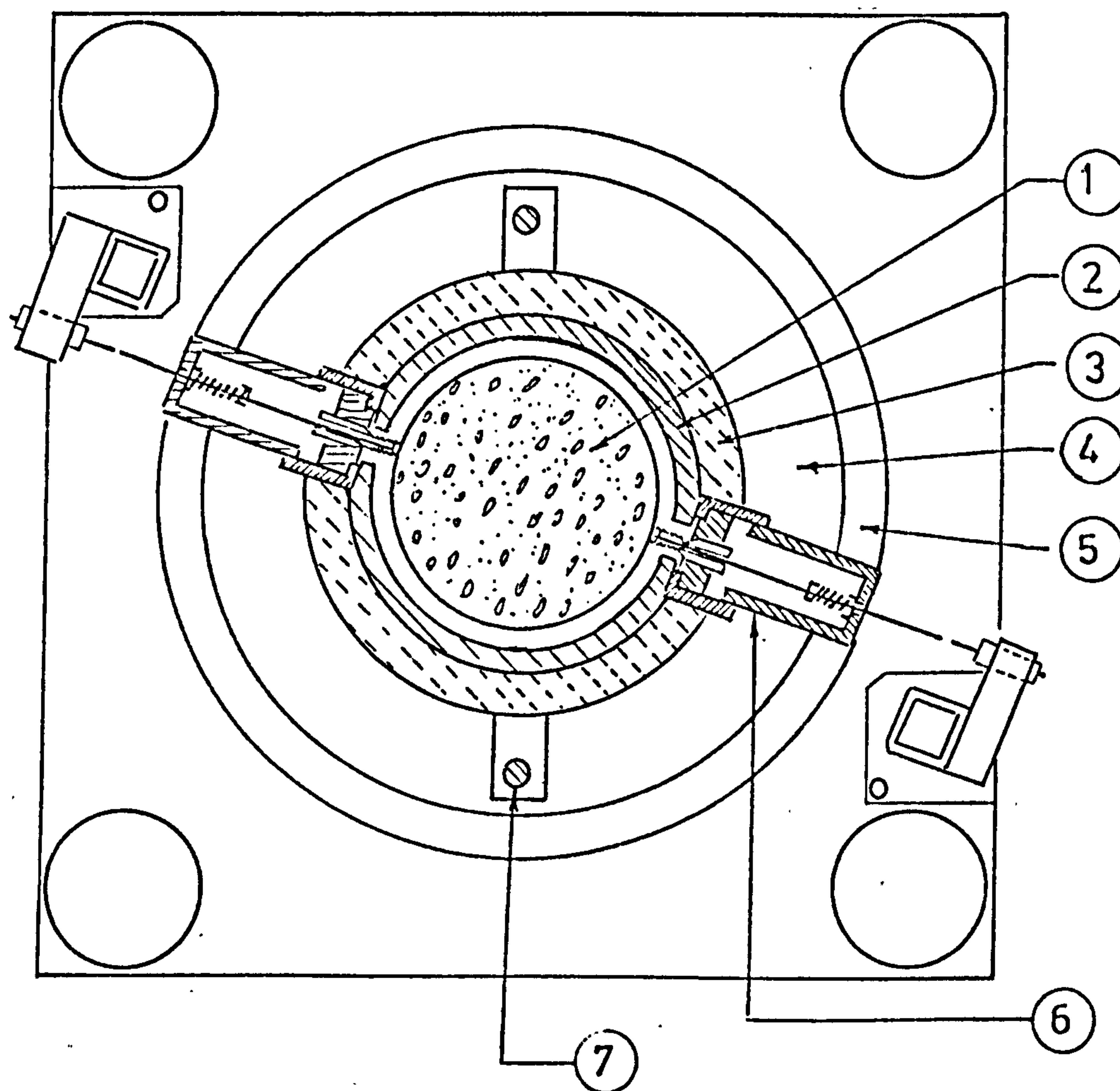


Fig. 3.6 general plan of the loading rig, pressure cell and concrete specimen

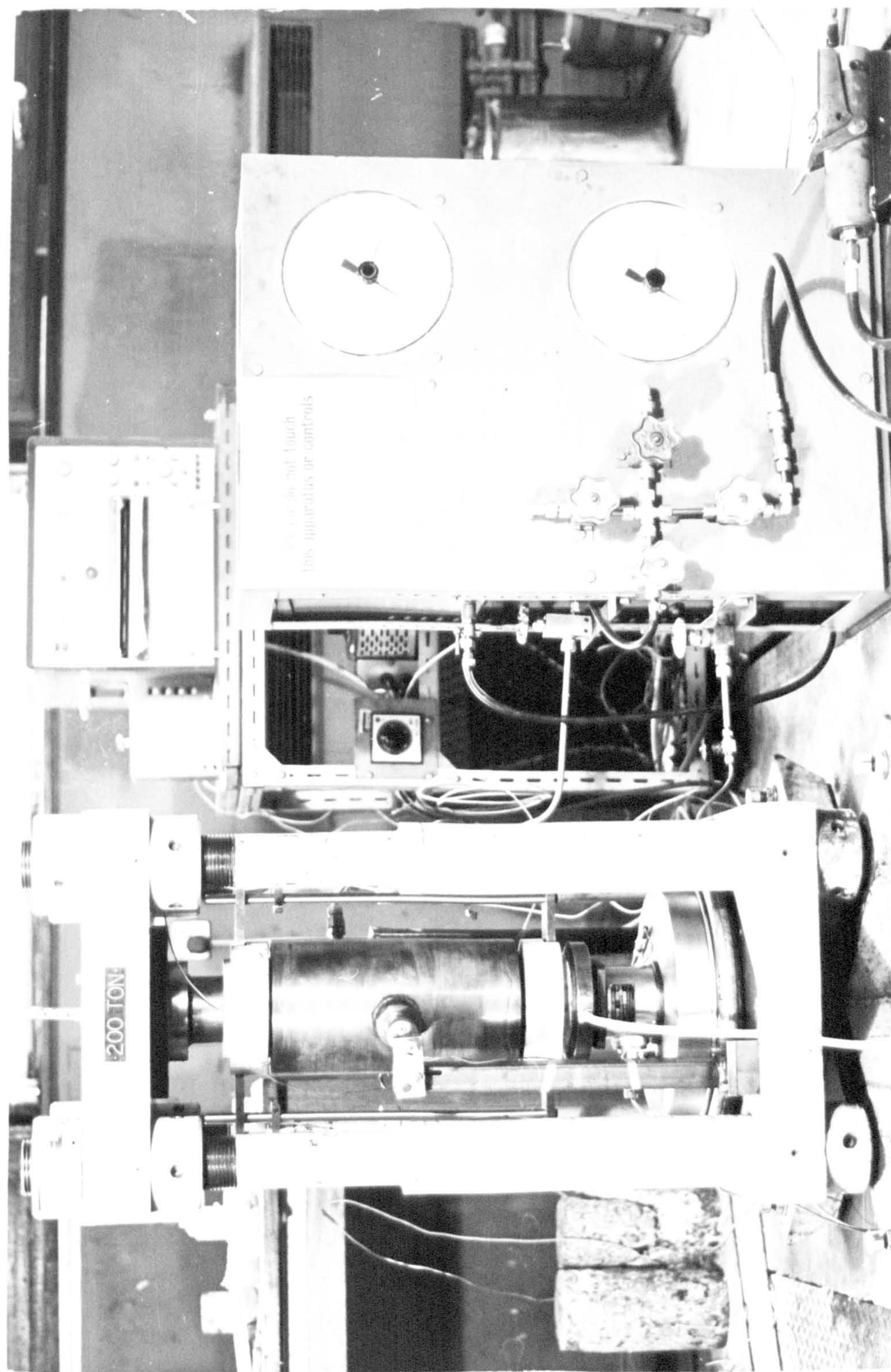


FIG. 3.7 GENERAL VIEW OF CREEP APPARATUS

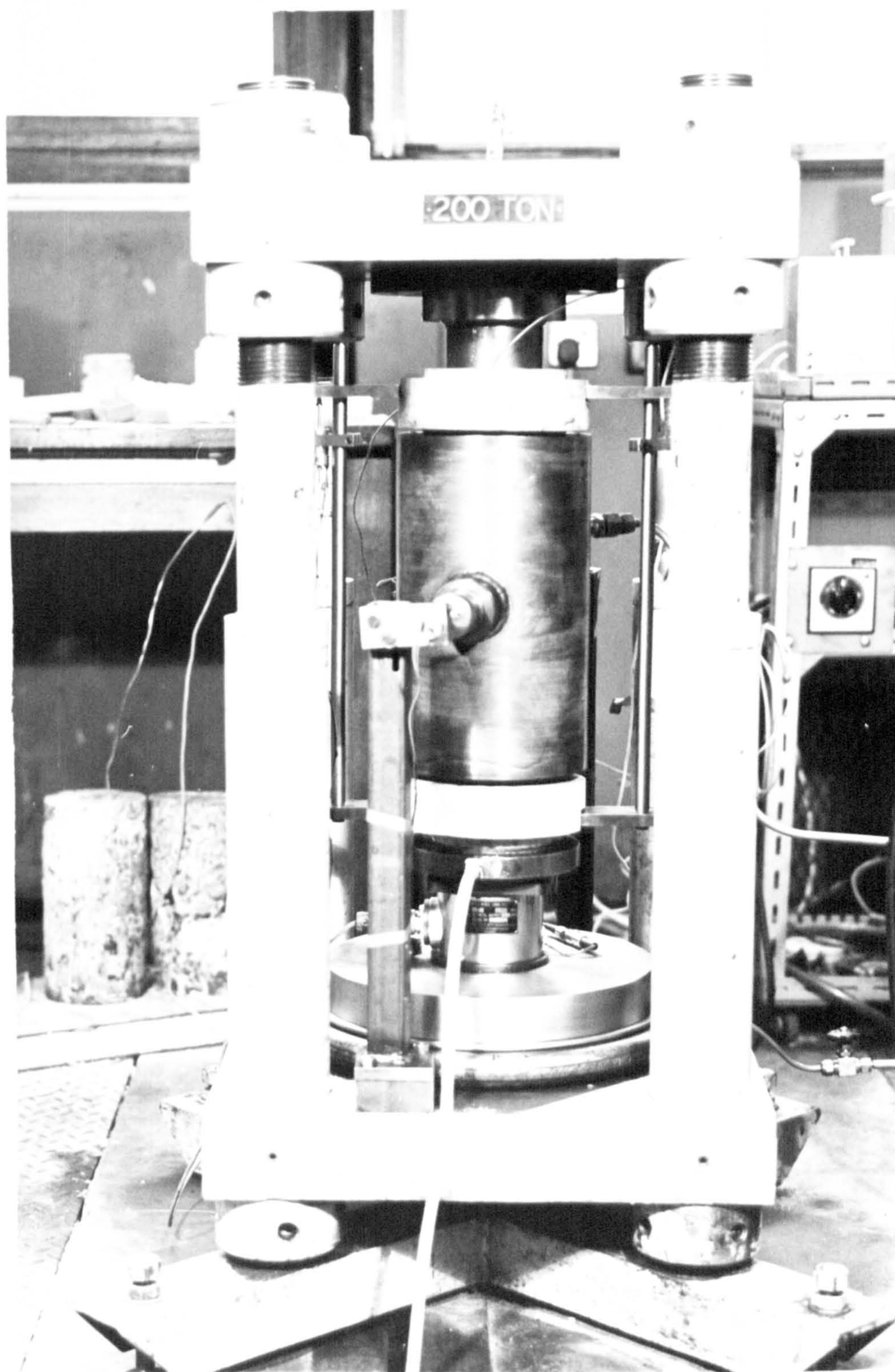


FIG. 3.8 LOADING FRAME & PRESSURE CELL

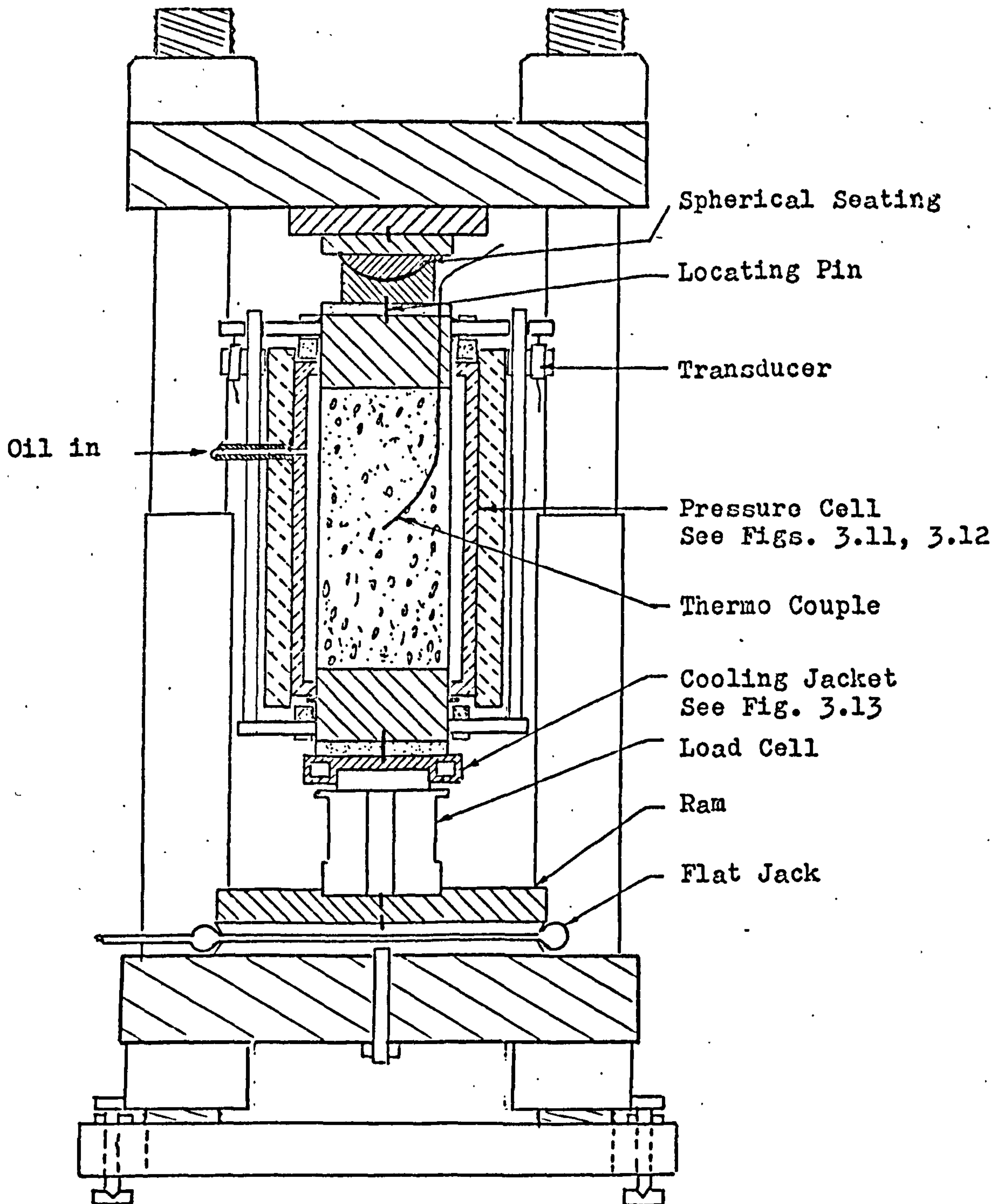
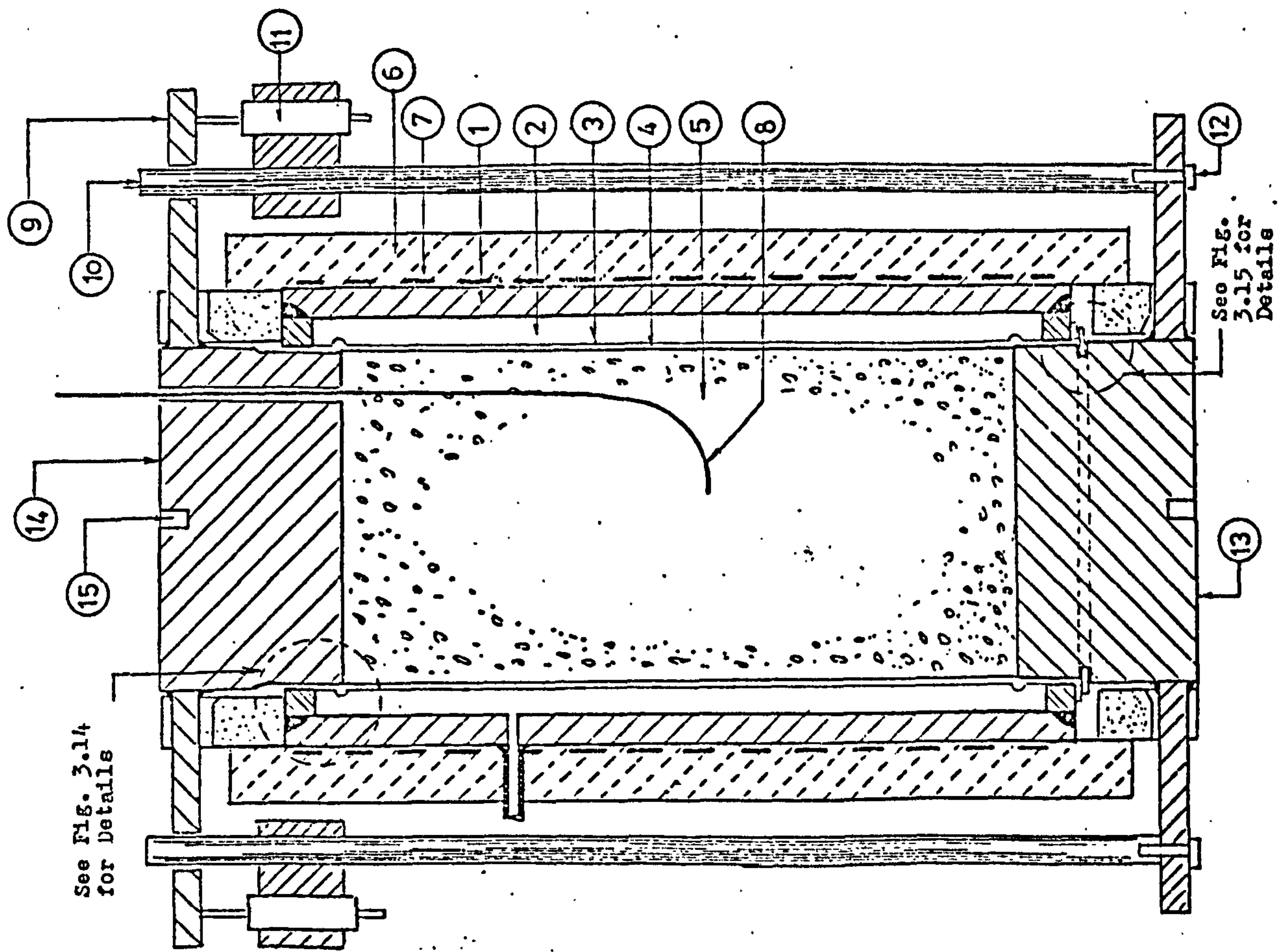


Fig. 3.9 General Elevation of the Loading Rig,
Pressure Cell, Concrete Specimen

FIG. 3.10 PRESSURE CELL



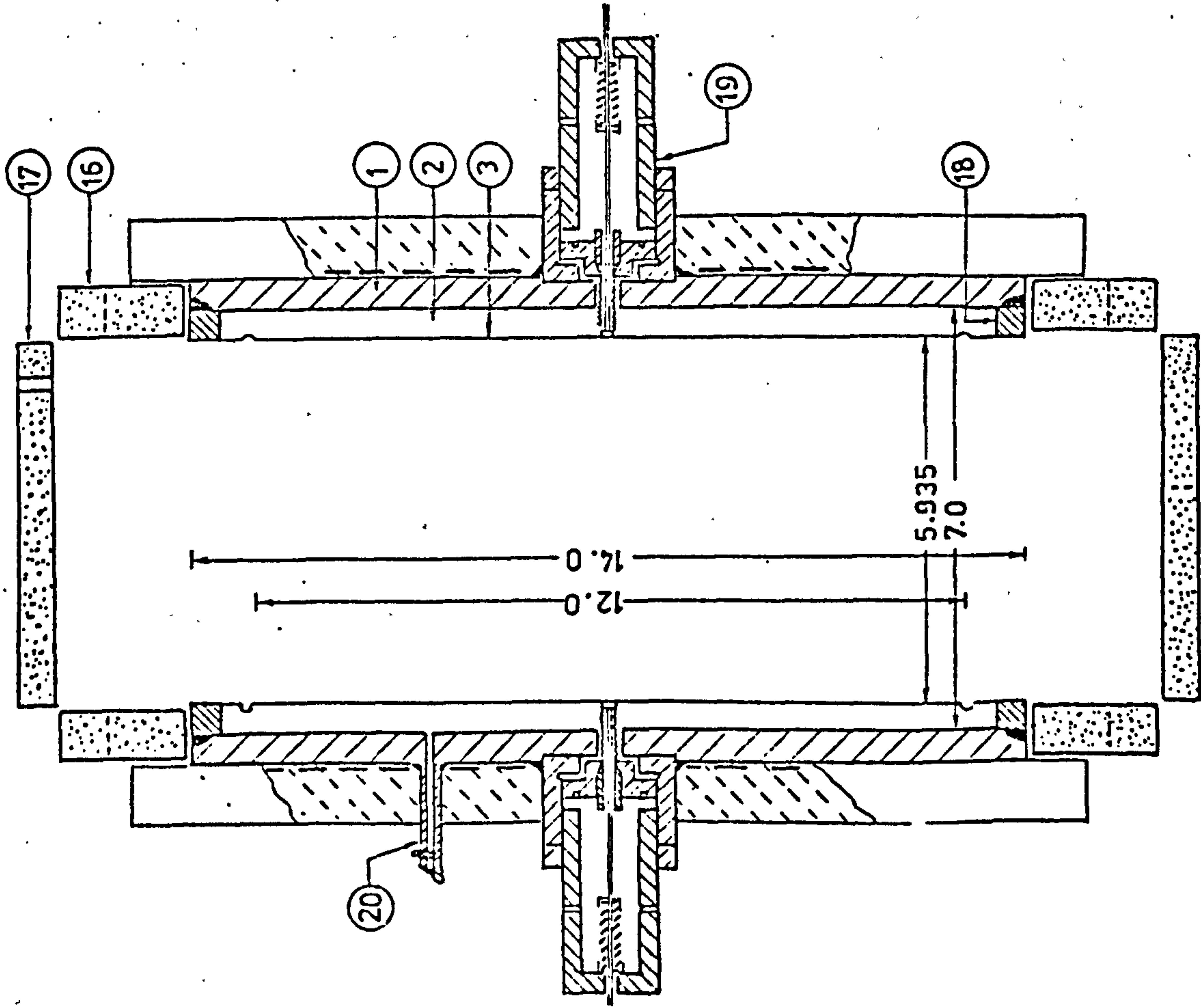
- ① Pressure Cell
- ② Oil under Pressure
- ③ 0.01 in. Stainless Steel Diaphragm
- ④ 0.02 in. Clearance
- ⑤ 6 in. Dia. 12 in. Long Concrete Cylinder
- ⑥ 1 in. Fibre Glass Insulating Jacket
- ⑦ 7.1 in. Long, 1000 W. Output Heating Tape
- ⑧ Mineral Insulated Thermocouple (Ch - Al)
- ⑨ 0.5 X 2 X 5 in. Plate
- ⑩ 5/8 in. Dia. Mild Steel Rod
- ⑪ LVDT Transducer
- ⑫ 1/2 in. Dia. Bolt
- ⑬ 6 in. Dia. 3.25 in. Thick Platen
- ⑭ 6 in. Dia. 3.25 in. Thick Platen
- ⑮ Locating Hole



See Fig. 3.14
for Details

See Fig.
3.15 for
Details

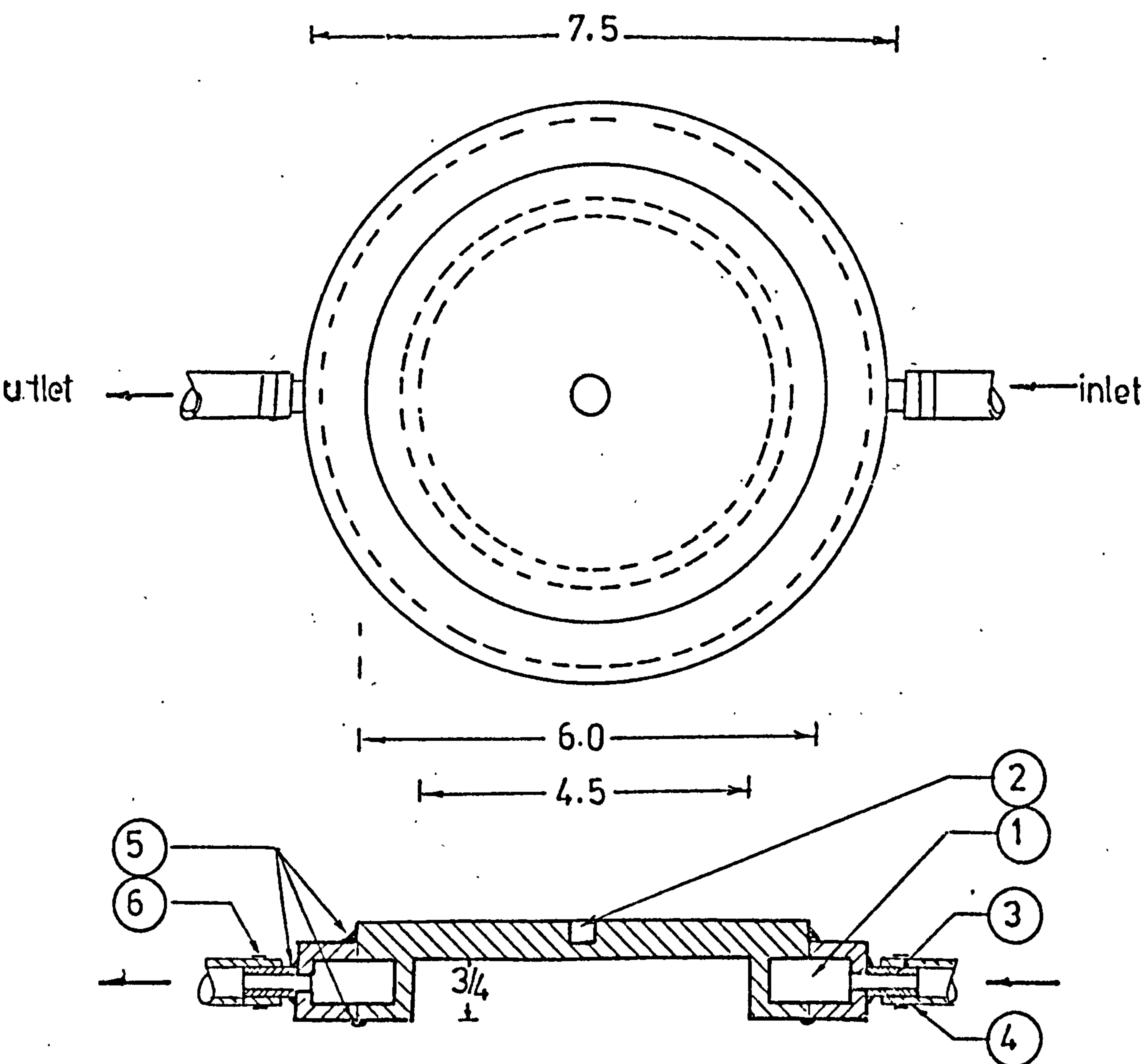
Fig. 3.11 Pressure Cell, Axial Measuring System and Insulation



- Pressure Cell
- Oil under Pressure
- 0.01 in. Stainless Steel Diaphragm
- Syndanyo Ring 6.1 in. Dia., 7.9 in. ext. Dia., 1.4 in. Height
- 5/8 in. Syndanyo Disc
- Radial Loading Device, See Figs. 3.18, 3.19 for Details
- 3/8 in. Dia. Oil Tube

Note: All Dimensions Are in Inches

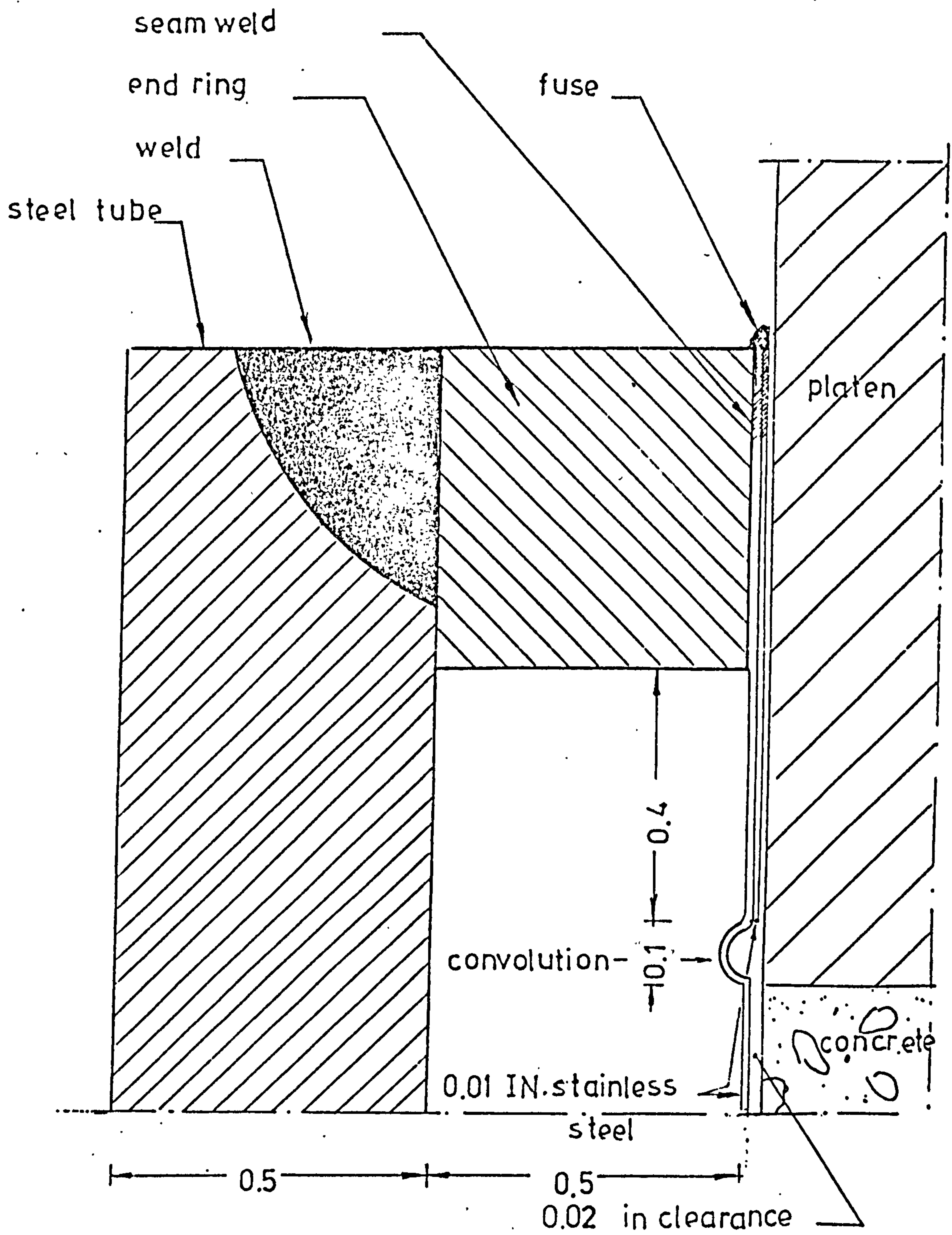
Fig. 3.12 Pressure Cell and Radial Measuring Device



- ① Water under Pressure
- ② Locating Hole
- ③ Brass Adaptor
- ④ PVC Reinforced Hose
- ⑤ Weld
- ⑥ Jubilee Clip

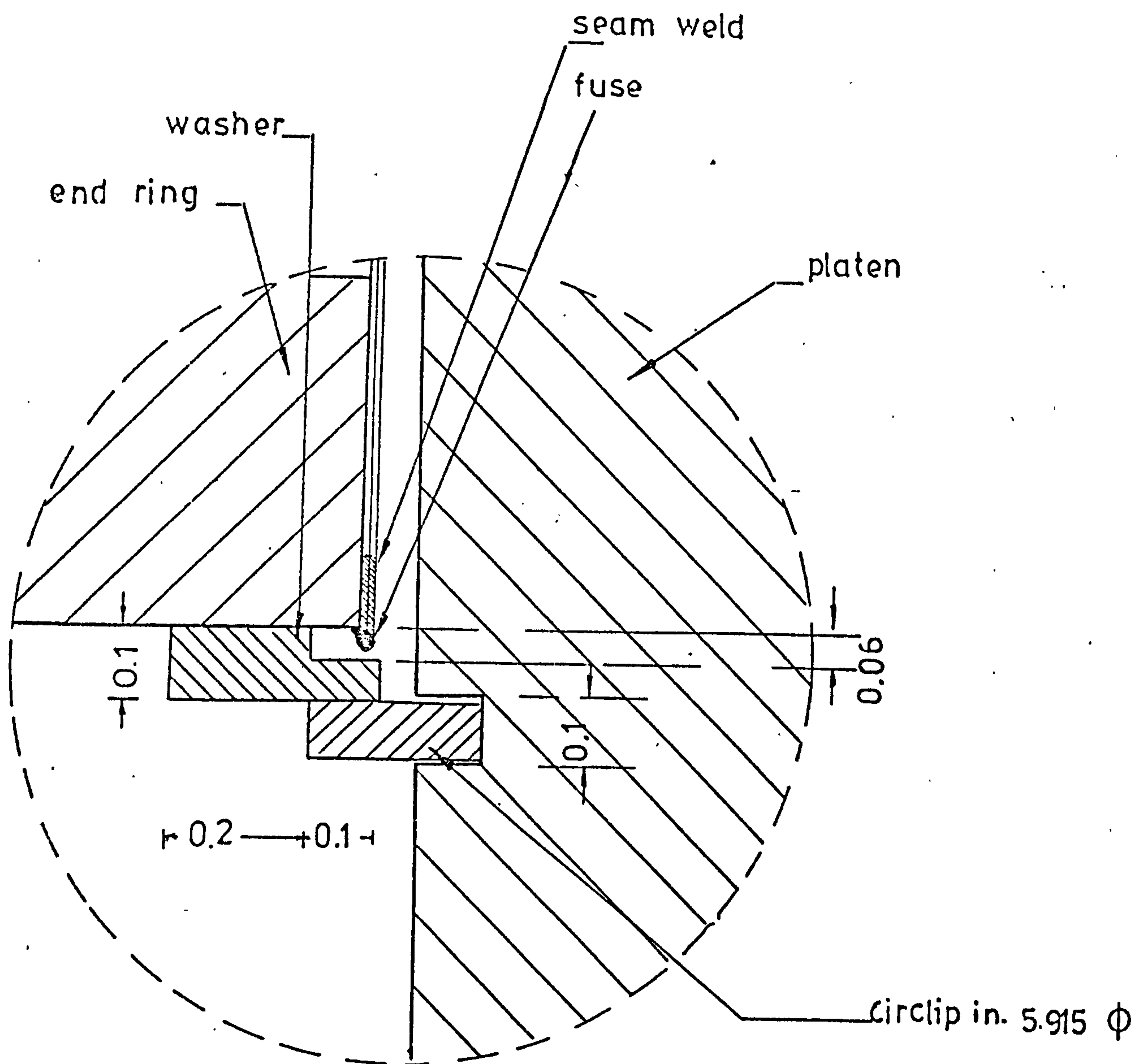
Note: All Dimensions Are in Inches

Figure 3.13 Cooling Jacket



Note: All Dimensions Are in Inches

Figure 3.14 Details of Pressure Cell Ends



Note: All Dimensions Are in Inches

Fig 3.15 Pressure Cell Support at the Bottom End



FIG. 3.16 P. T. F. E. SEALS PROBE ROD

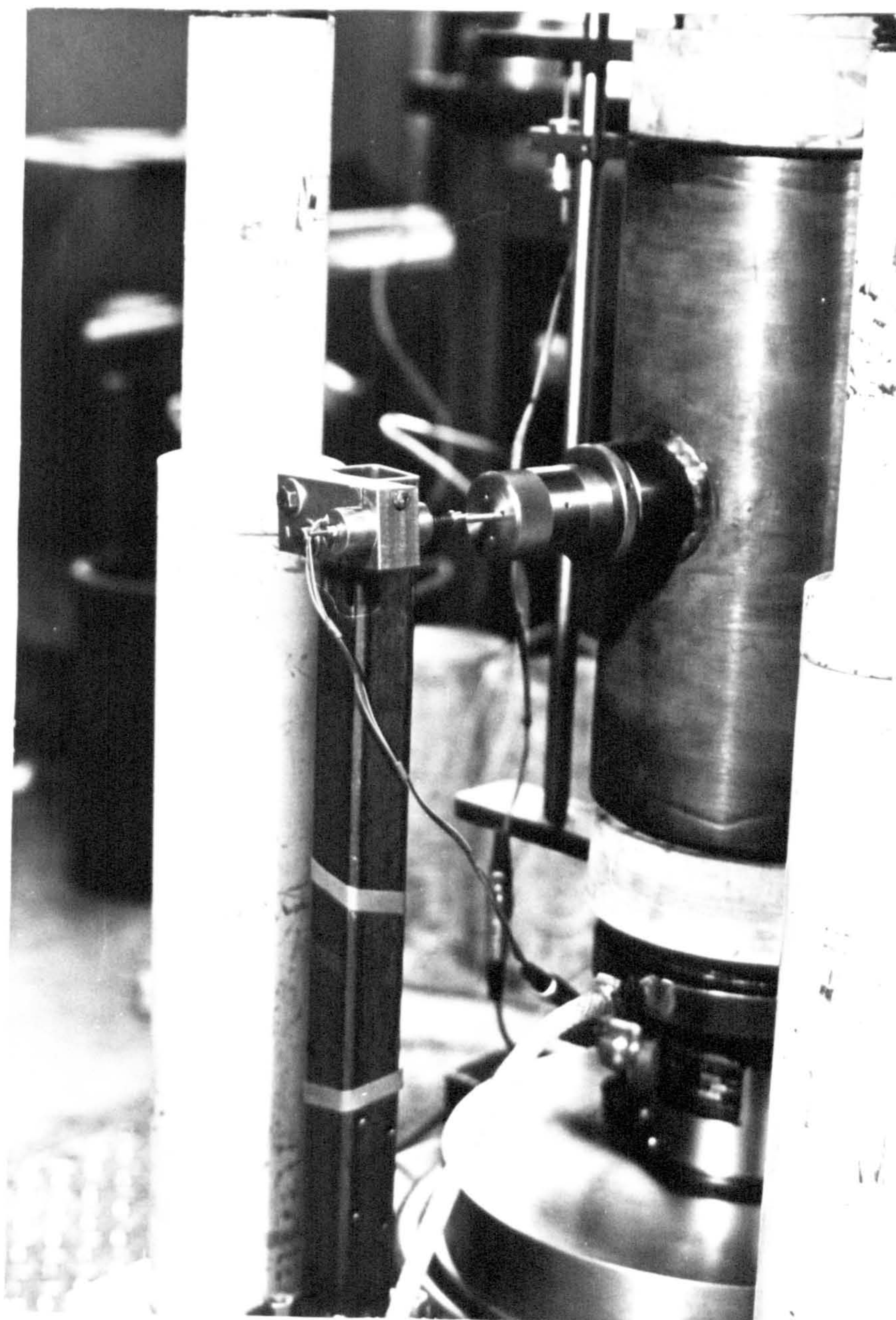


FIG. 3.17 ARRANGEMENT OF RADIAL MEASURING
SYSTEM

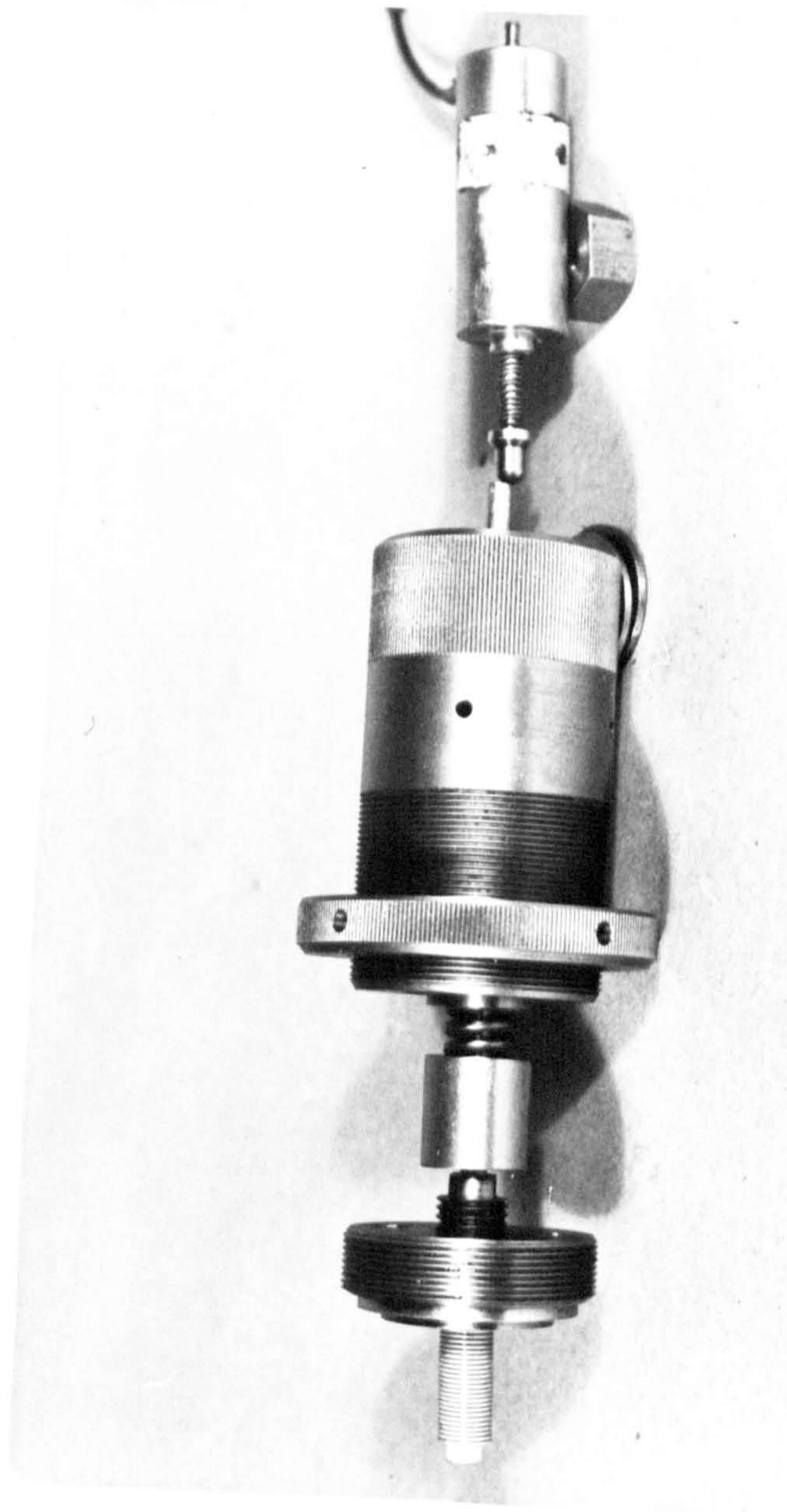


FIG. 318 RADIAL MEASURING DEVICE

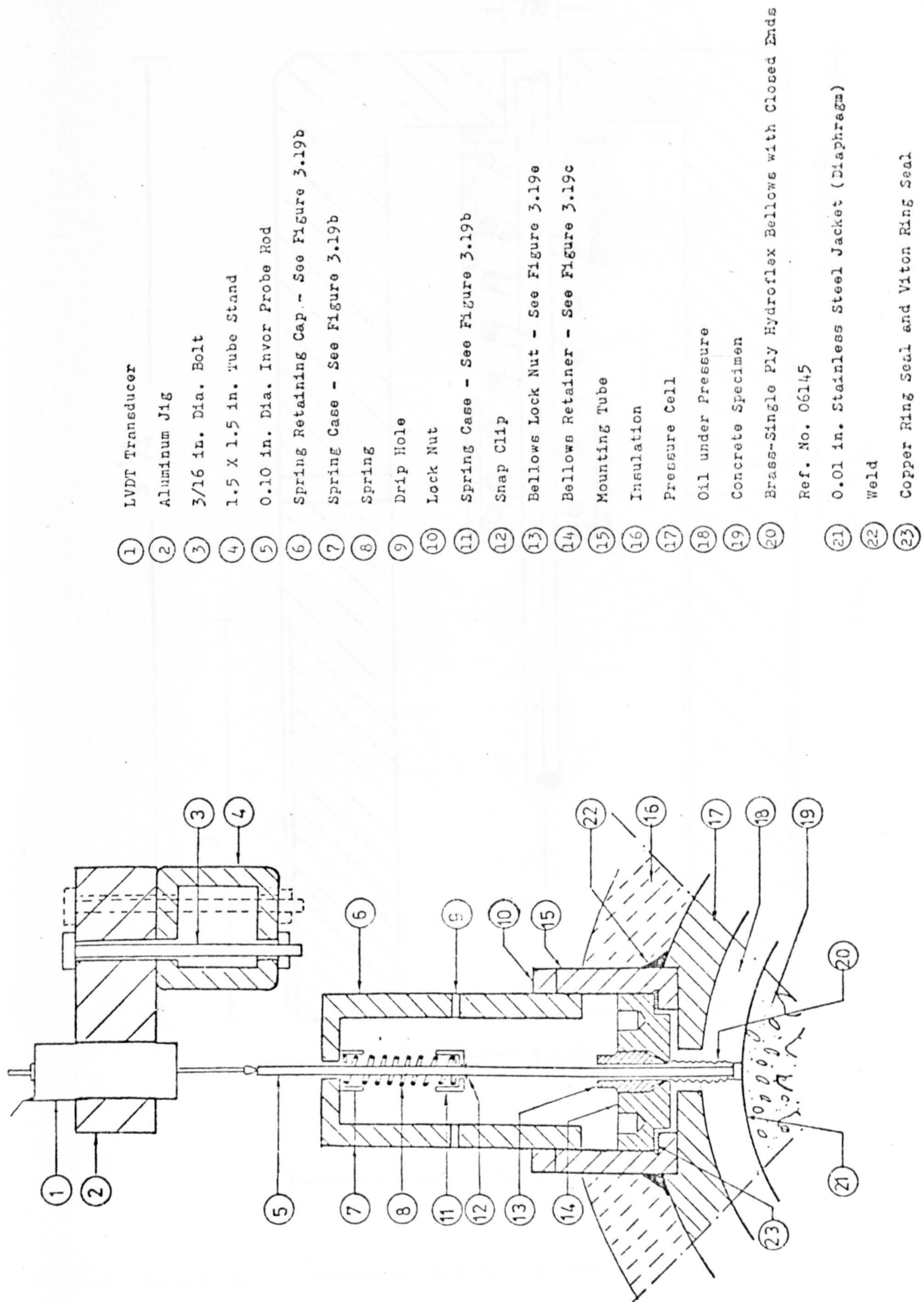
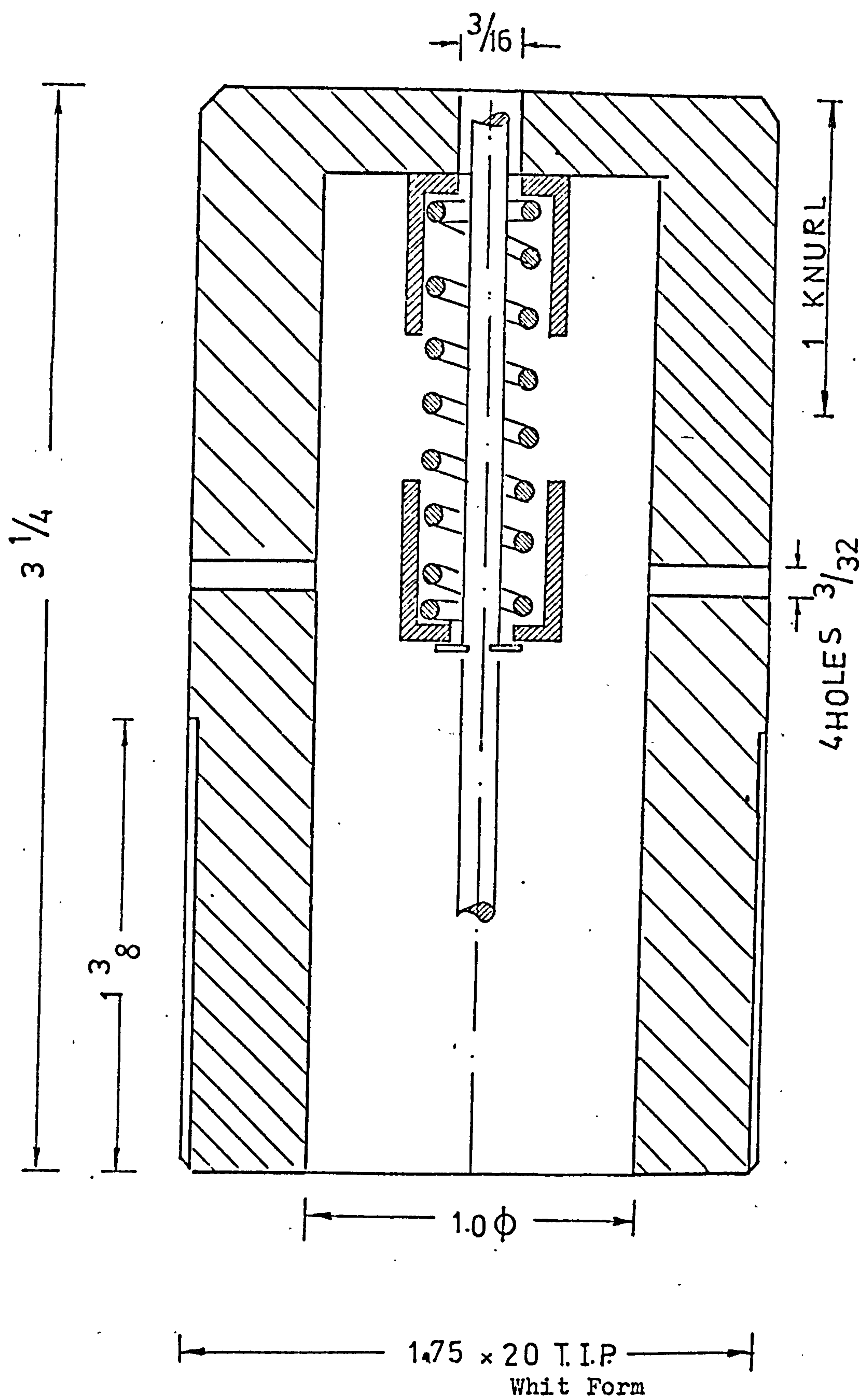
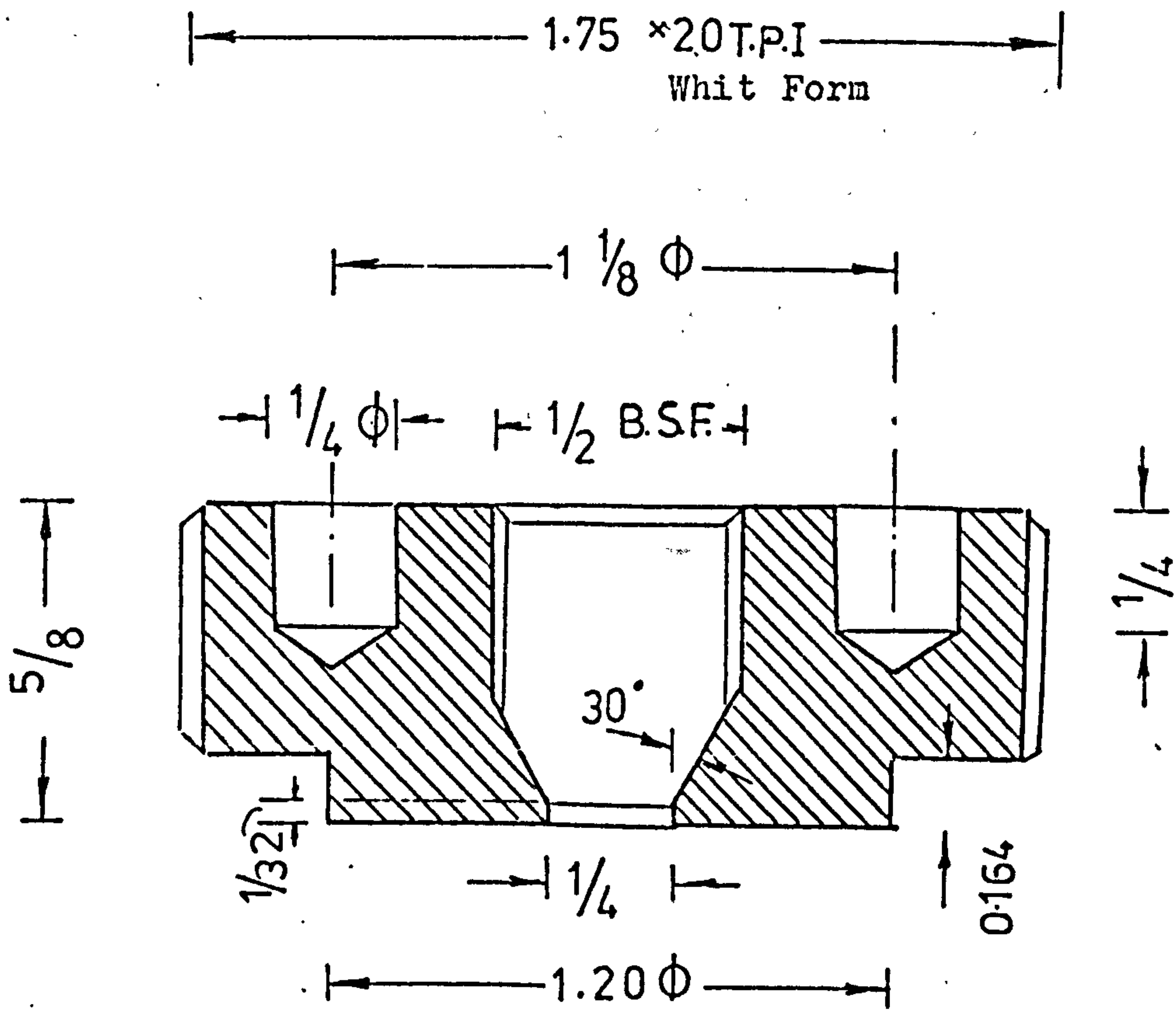


Figure 3.19a The Radial Measuring Device



Note: All Dimensions Are in Inches

Figure 3.19b Spring Retaining Cap Material: Mild Steel



Note: All Dimensions Are in Inches

Figure 3.19c Bellows Retainer Material: Mild Steel

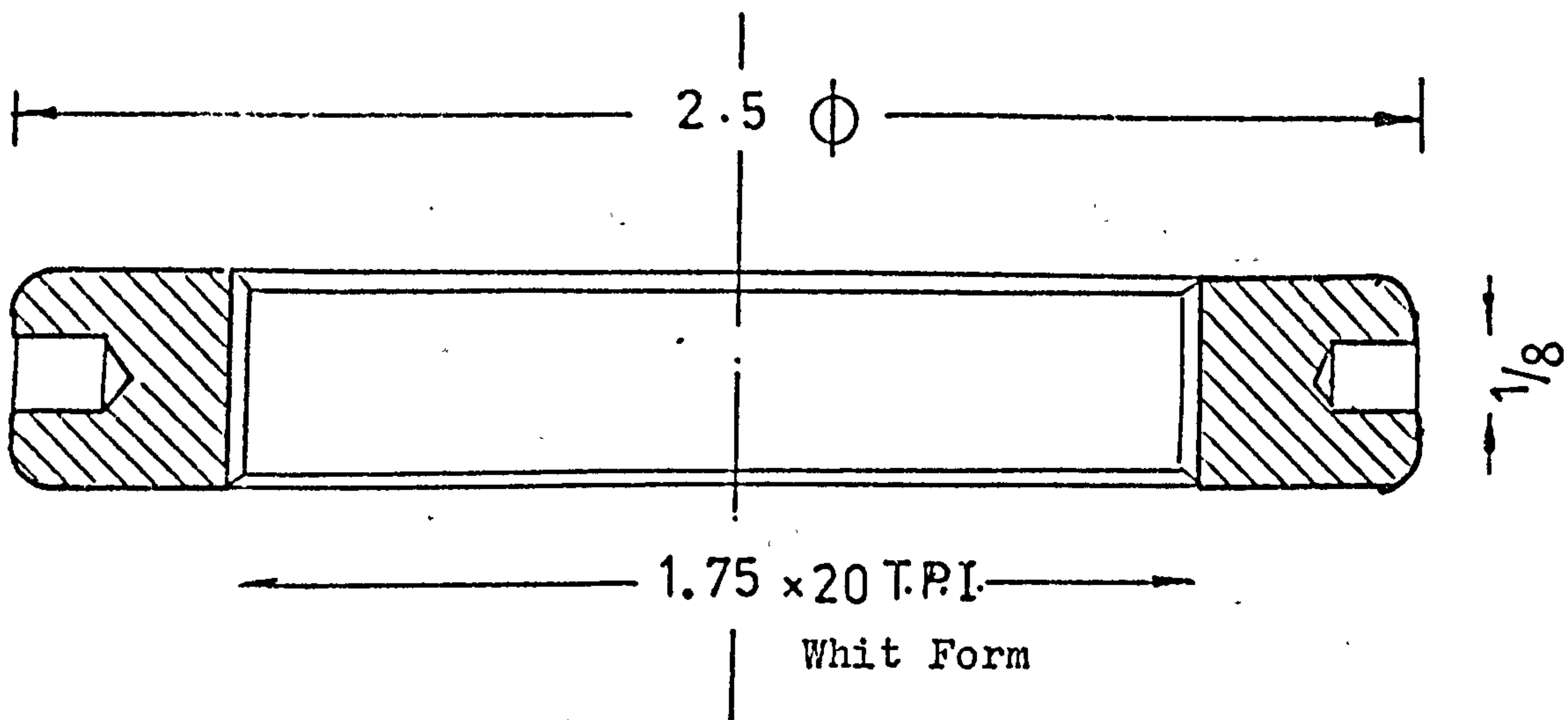


Figure 3.19d Lock Nut Material: Mild Steel

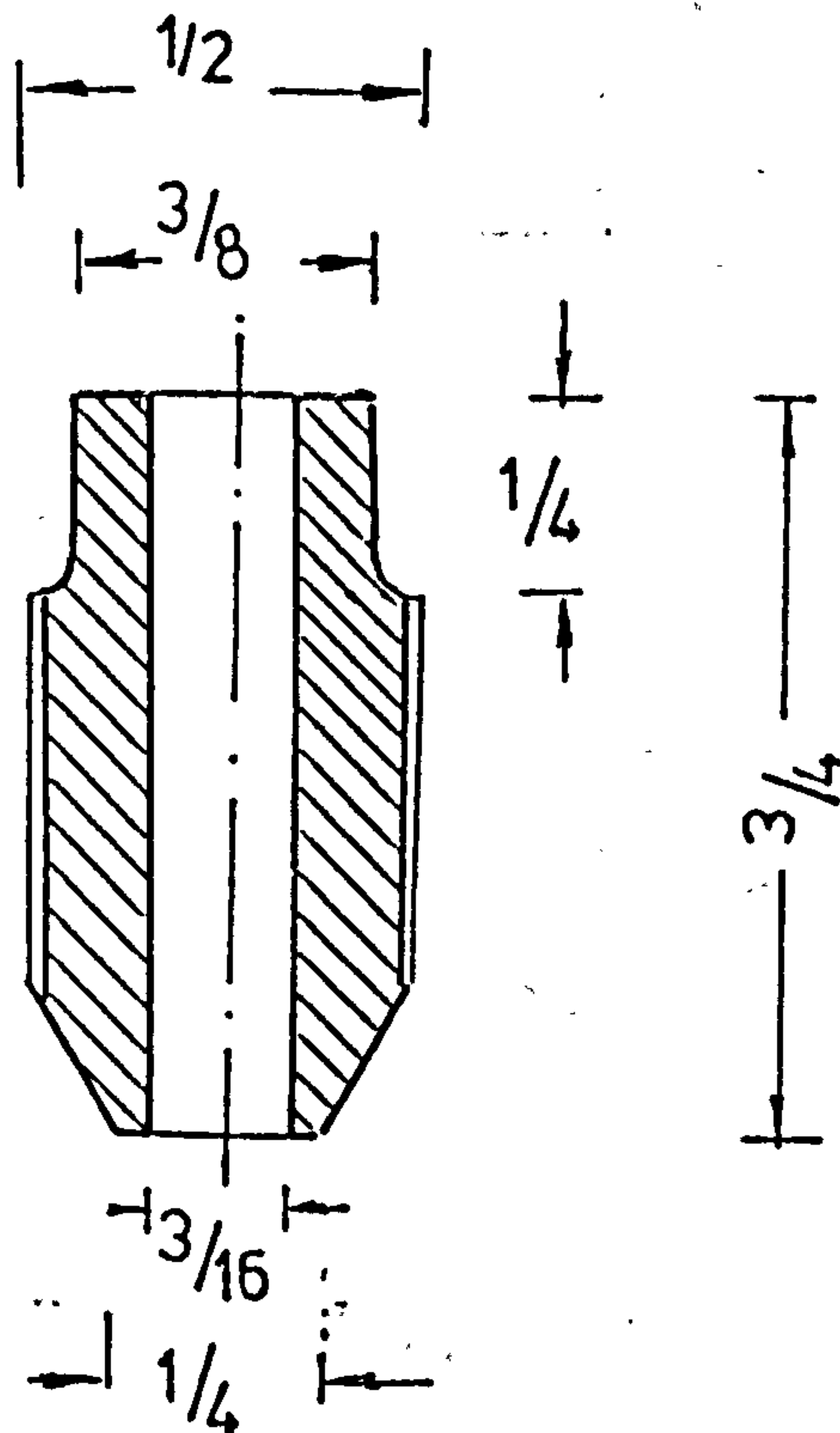


Figure 3.19e Bellows Lock Nut Material: H.T.S. Screw

Note: All Dimensions Are in Inches

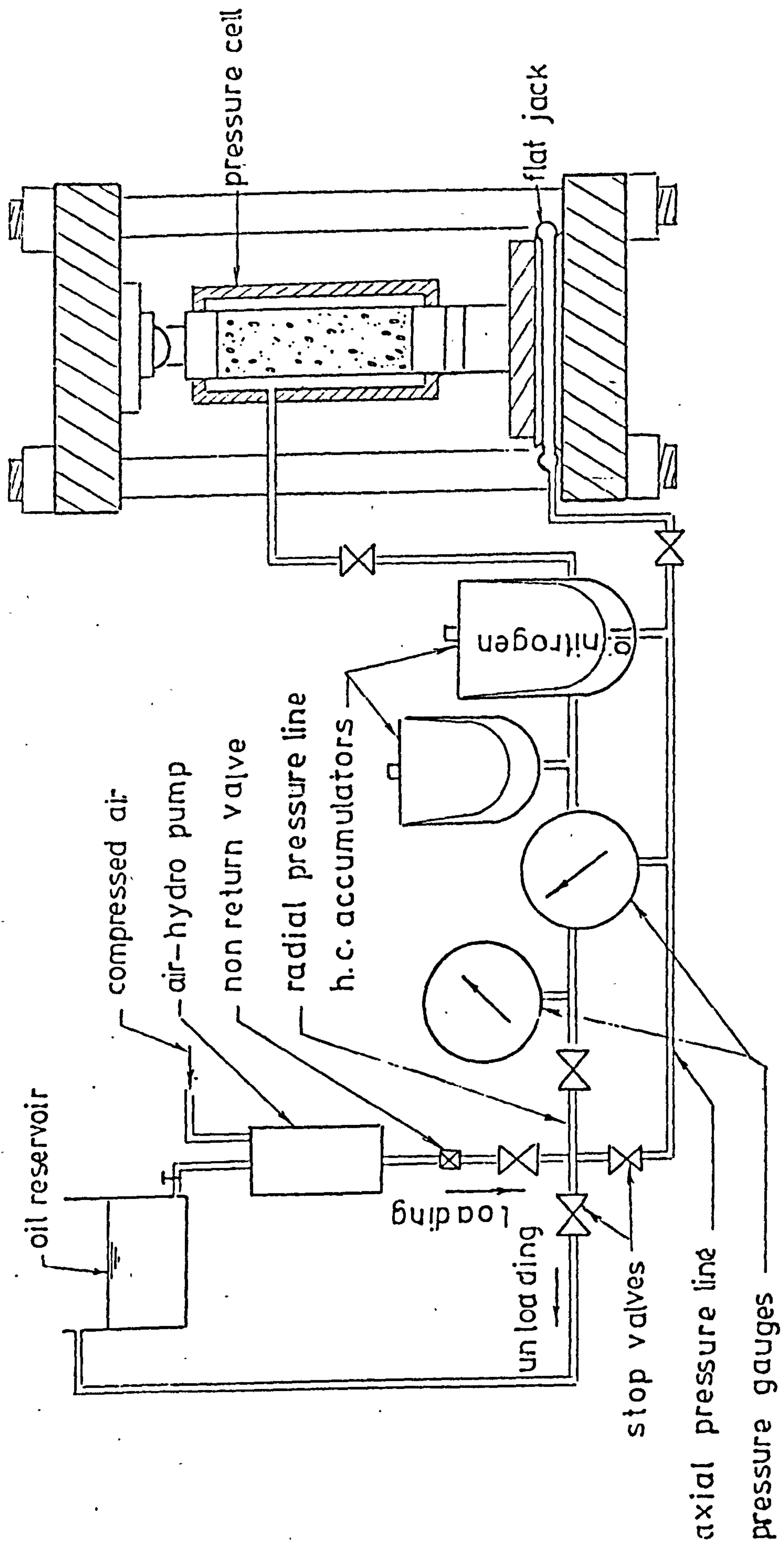
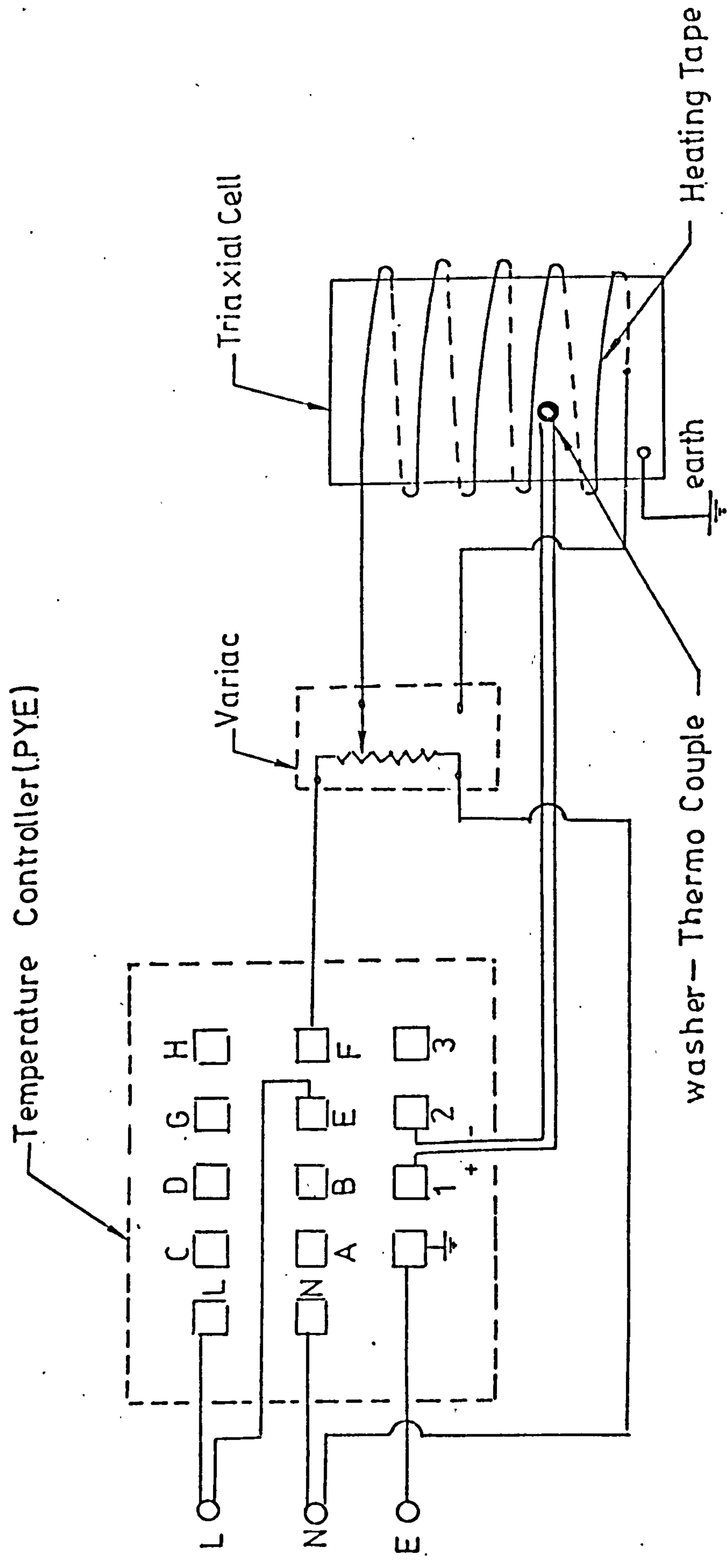


FIGURE 3.20 HYDRAULIC LOADING SYSTEM



Main Supply : 240 V.

Main Wiring : 240 V , 5 A.

Figure 3.21 Electrical Connections (heating system)

Chapter 4

INSTRUMENTATION

4.1 Introduction

The instrumentation is described in the following groups depending upon their function as mentioned below:

- i) Measurement of temperature,
- ii) Measurement of axial displacement,
- iii) Measurement of radial displacement,
- iv) Measurement of the applied load,
- v) Measurement of thermal expansion and shrinkage test.

The temperature at various locations on the pressure cell and also within the specimen itself was monitored by thermocouples. The radial and the axial displacements of the specimen were monitored by d.c. transducers. The applied axial load was recorded as an electrical output from a load cell placed between the loading platen and the ram and also from the pressure gauges.

Any equipment which could be influenced by temperature was placed well outside the pressure cell.

4.2 Measurement of Temperature

Temperature measurements are necessary to provide a record of thermal environment, to interpret displacement measurement,

and to obtain local temperature gradients from measurement at various points. For description of thermal environment, approximate values, say within $\pm 2^{\circ}\text{C}$ are satisfactory. This accuracy is also be sufficient for interpretation of displacement readings, provided the transducers are not under temperature influence. However, in order to separate the total strain into thermal mechanical strain components, it is necessary to have a more accurate measurement of temperature. Also, if the local temperature gradients are to be established reliably, the measurement of local temperature must be quite accurate.

While the requirements on the accuracy of thermal measurements vary with the purpose of these measurements, it was desired that the thermocouples to be embedded in the specimen should have the following characteristics:

1. the smallest possible size,
2. desired accuracy in the range of 20 to 250°C ,
3. fast response time,
4. stability for long time,
5. stability under compressive stresses up to 10,000 psi.

It was decided to use thermocoax thermocouples made to order. They are derived from thermocoax mineral insulated thermoelectric cable. The probe of thermocoax has two thermo-electric conductors consisting of chromel-alumel protected by a stainless steel sheath, the insulation between conductors and between conductors and sheath being highly compacted MgO powder. The

thermo-electric conductors are welded to one end to form a hot junction. The sheath is closed by argon arc welding, the hot junction remaining insulated from the sheath. The probe is extended by a flexible extension cable having conductors (with the same thermo-electric properties as the sensor) and a copper shielding braid extending the protection tube. The shielding braid is insulated and protected by an outer plastic cover.

The 6 in. length embedded within the specimen during casting was protected by 0.1 in. diameter stainless steel sheath. The cold junction of the thermocouples were connected to a multi-channel U.V. recorder. The outputs of these thermocouples were in the range of A.100 galvanometers, therefore, damping resistors were unnecessary.

Another type of thermocouples was made by twisting and then fusing the two leads of a copper-constantan 25 gauge wire, with extruded nylon insulation on each lead as well as on the overall wire. The hot junctions of the thermocouples were fixed on the pressure cell body by asbestos string, while the cold junctions connected to a multi-channel temperature indicator (Comark). The latter instrument was suitable only for the copper-constantan thermocouples, from which direct reading can be obtained.

4.3 The Measurements of the Deformations

4.3.1 Vertical Displacement

The vertical movement of the cylinder is measured by D.C. Linear variable differential transformers (LVDT), D-Series

Type D411-01, order made supplied by Electro Mechanisms Ltd.

The LVDT coils are wound with polyester covered copper wire on cylindrical ceramic or glass-fibre formers which offer good dimensional stability, a stiff structure and good resistance to thermal shocks.

The complete coil assembly is housed in cadmium plated mild steel, or ferromagnetic stainless steel case providing full electro static and electro magnetic shielding. The free, unguided moving core is manufactured from nickel-iron alloy and is heat treated to produce a uniform permeability.

Electrical connections to the LVDT are via colour coded, silver-plated, PTFE covered, copper wire leads, 12 in. long which are anchored within the transducer case in an epoxy resin seal and which emerge through one end plate.

This transducer was mounted on an arm attached to a rigid support. The transducer spindle was placed on a mild steel plate welded to the loading platen.

The electrical circuit is shown in figure (4.1). The transducer was energised by 24V D.C. Due to high resistance of the transducer $50k\Omega$, damping resistor was necessary to provide the galvanometer with a level frequency response. The parallel resistor R_2 can be determined from the equation.

$$R_2 = \frac{R_s + R_1}{R_s + R_1 - R_d}$$

where

R_1 is series resistance

R_2 is parallel resistance

R_s is transducer resistance

R_d is damping resistance

In this system, it is ensured that no voltage exceeding the safe maximum value for the galvanometer as well as for the U.V. recorder.

4.3.2 Radial Displacement

The radial displacement was measured by the same type of transducer used for vertical displacement with similar electrical circuit as well. Two transducers were used as shown in figure 3.8. Each was mounted on a cantilever against an Invar steel probe rod with the special arrangement described before.

4.4 Load Measurement

The radial pressure was monitored accurately by a Budenberg pressure gauge, it was possible to measure up to ± 5 psi.

The axial load was measured in two different ways as follows:

1. by noting the pressure gauge and the measurement of the gap occupied by the flat-jack. A characteristic of this type of jack is that the load-pressure relationship varies with the size of the gap occupied by the jack. Thus with contraction of the specimen, the load corresponding to a pressure gauge reading. See figure 4.2.

2. by order made load cell, capable of measuring compressive axial loads in the ranges from 0 - 100 tones to an accuracy of 0.25% throughout the whole range. It is made of EN30T steel with chromium plated outercasing to protect the inner sensitive element, and hermetically sealed to make it completely water proof.

The gauge pattern is particularly important as it partially governs the correction of the anomalous sensitivity of the cell when it experiences loads distributed non axially.

The strain gauges are wired to a central spider as shown in figure 4.3, the connections being made so that a complete resistance bridge is contained within the cell. Input and output connections are made by a 6-pin plessey connector which is mounted on the protective casing. As the complete resistance strain bridge is contained within the cell, its load dependent output is not disturbed by small variations in lead and contact resistance in the measuring circuit. Since the strain gauges comprising the four arms of the bridge are all attached to the inner wall of the cylinder which is isothermal surface, any changes of resistance caused by variations in temperature in one gauge are balanced by similar temperature-dependent variations of resistance in the corresponding gauge in the opposite arm and in this way, the cell is compensated for changes in ambient temperature.

Figure 4.3 shows schematic diagram of four arms bridge strain gauges A-A, D-D, where A represents the active strain gauge, and

D represents the Dummy strain gauge. Resistance R are for temperature compensation and non axial load correction. The cell supplied with 5 volts D.C. input voltage, the output was fed into the U.V. recorder through B.160 galvanometer.

4.5 Measurement of the Thermal Expansion and Shrinkage

An Ether furnace was used to provide thermal environment for the shrinkage test. This furnace is equipped with temperature controller of the Ether Transitrol to regulate the furnace temperature. See figure 4.4. it could maintain the furnace temperature at any level between 20°C and 1000°C to within $\pm 5^{\circ}\text{C}$ of the set temperature by regulating the power supply. The required temperature level could be set easily on the dial showing 20°C - 1000°C range at steps of 10°C . The thermocouple reading was indicated by a hand moving across the dial. When the moving hand reached the set temperature indicator, a relay switch operated by photo-sensitive cells, cut off the power supply to the furnace. Once the temperature dropped by a few degrees, the power supply was resumed. This process could be continued automatically to maintain the temperature at the desired level for as long as required.

The controller was also equipped with a programmer in the form of a cam disk attached to an arm connected to the set temperature indicator and could be operated to raise the temperature of the furnace at any desired rate by altering the cam profile. The cam disk itself was regulated by a motor through a suitable reduction gear system.

4.6 U.V. SE 2100 Electronic Recorder

The ultra violet U.V. light recorder type SE 2100/25 was used effectively, which is a direct writing oscillograph. The recorder utilises high-sensitivity mirror galvanometers (voltage range 0-29V) writing upon fast print-out paper to give, almost immediately, a record of correlated channels. See figure 4.5.

The principle of operation depends upon the incident light from an ultra-violet lamp being deflected by the movement of a small mirror attached to the galvanometer suspension. The movement is caused by the torque produced when current passes through the coil attached to the suspension. Recording is effected by focusing the reflected spot onto sensitised print-out paper which produces a high contrast recording within a few seconds under normal ambient light conditions.

Up to 25 channels can be recorded on a roll direct print-out paper, which is rolled emulsion side inwards. A viewing scale clearly marked at 2 mm and 1 cm intervals, coincident with grid lines, provides a simple and convenient method of setting galvanometers zeros. Traces can be deflected over 30 cm which is full width of the recording paper with non-linearity not greater than $\pm 1\%$ over 6 cm deflection. Two longitudinal reference traces are projected by datum units (dummy galvanometers), one situated at either end of the magnet block, and adjusted to any point over the width of the paper.

A three-phase synchronous motor, connected capacitively

in a delta configuration and run from a single-phase supply, drives a multi-speed gearbox via a system of non slip toothed belts, resulting in a paper acceleration time of less than 0.2 seconds. Selection of 15 speeds from 1.25 mm/s to 2000 mm/s prior or during a recording is achieved by a group of front panel operated bottoms. The speed was set on 5 mm/s.

4.7 Galvanometers

Three types of galvanometer were used A.100, A.35, which cover the range 0-3.5 mv, and B.160 which covers the range 0-14 mv; some of them were provided with damping resistors to prevent the voltage exceeding the safe maximum value for the galvanometers.

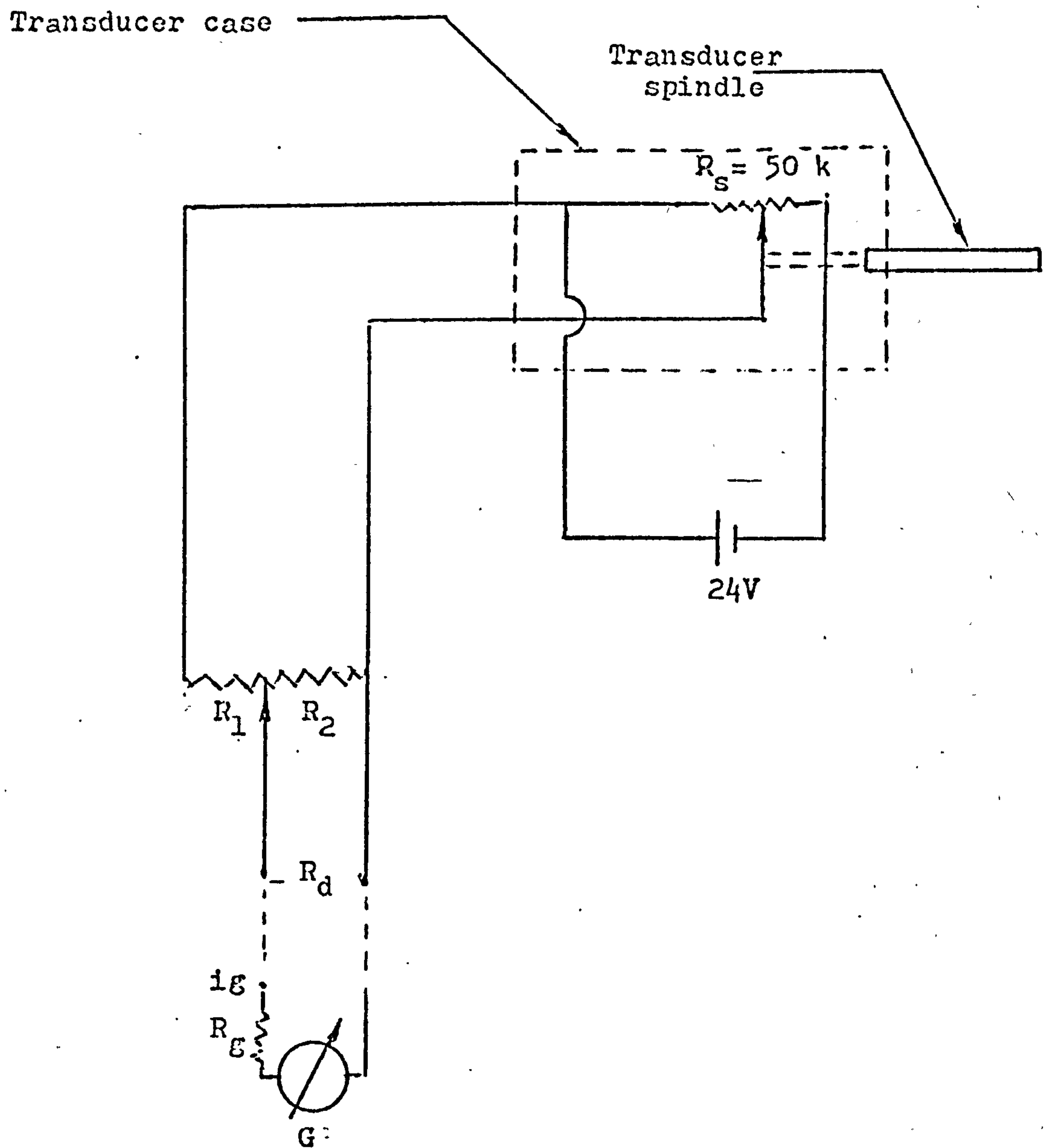
The light beam from the U.V. lamp is focused onto the galvanometer lens via the first focusing lens and the first deflecting mirror angled to the plane. Light reflected from the galvanometer mirror is passed through its plano-convex lens to the second deflecting mirror. A shutter control (TRACE INTENSITY) operated from the front panel of the U.V. recorder, controls the amount of light falling into the mirror and hence through the final focus lens onto the recording paper.

4.8 Power Packs

A minoreg unit of 325 type power pack was used to supply stable d.c. power to the transducers monitoring the axial and the radial displacement of the specimen. This unit was designed to provide stable supply when connected to a 50 cycles per second, 220 volts AC mains. The output range could be varied

between 0 - 25 volts with amperage varying between 0 - 5 amps.

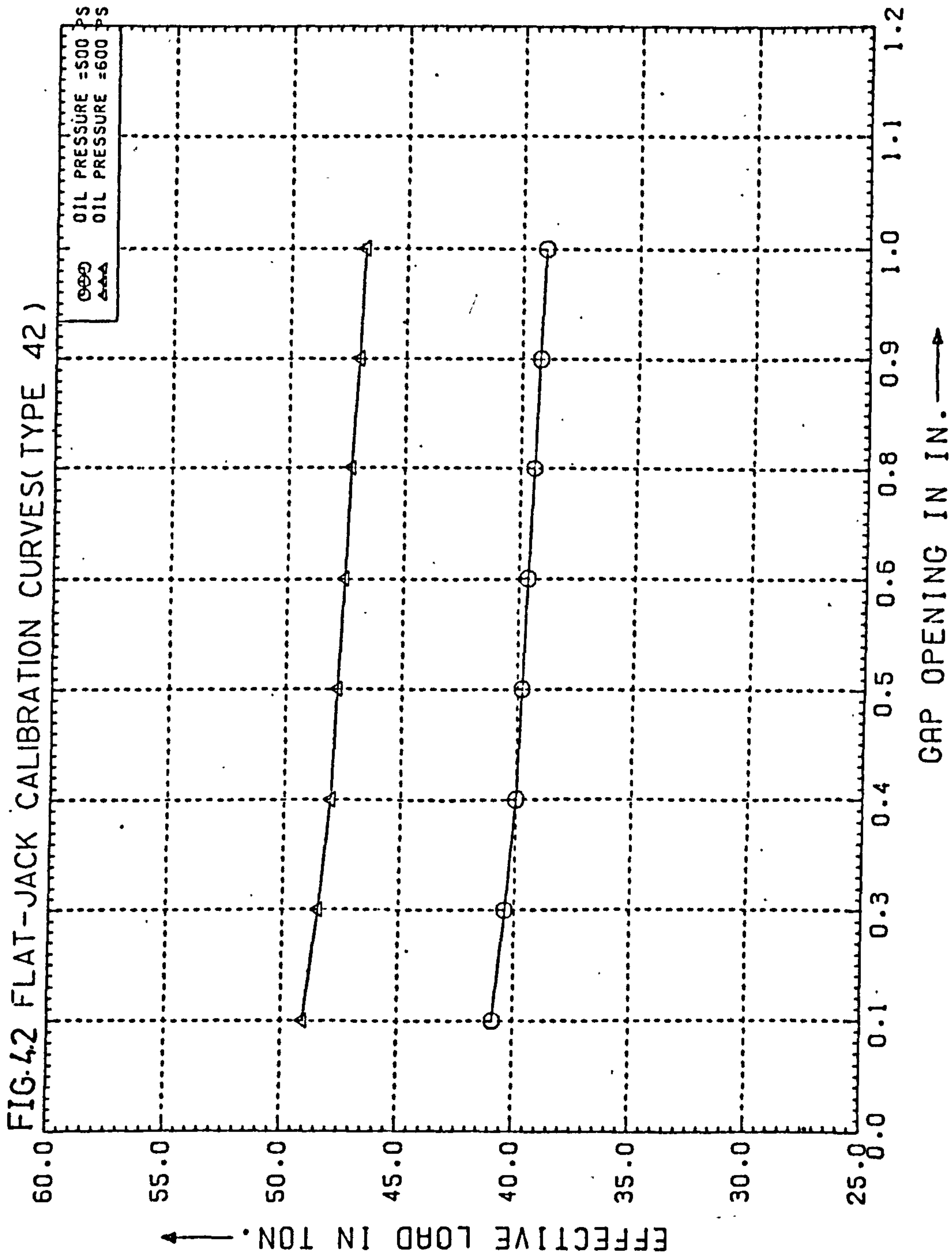
A separate power pack was used to supply the load cell, in order to avoid any undesirable electrical interference in the system, this unit is identical to the one mentioned above. See figure 4.5.

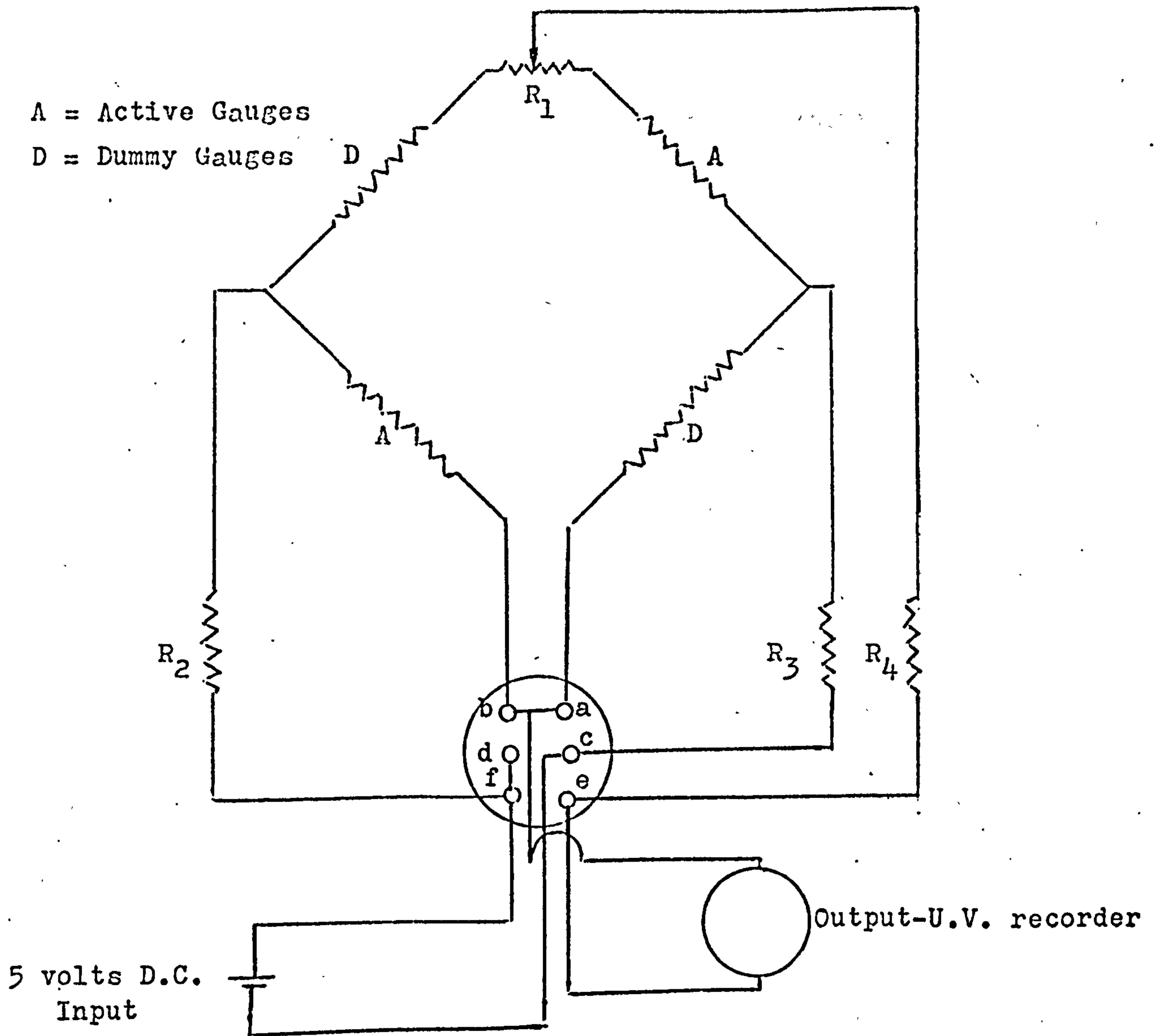


R_g = Galvanometer Resistance
 G = Galvanometer
 R_d = Damping Resistance
 R_1 = Series Resistance

R_2 = Parallel Resistance
 R_s = Transducer Resistance
 i_g = Galvanometer Current

Figure 4.1 Typical Electrical Connections for D.C. Transducer.





R_1, R_2, R_3, R_4 are for resistances for temperature compensation and non axial load correction.

Figure 4.3 Electrical Connection of Load Cell

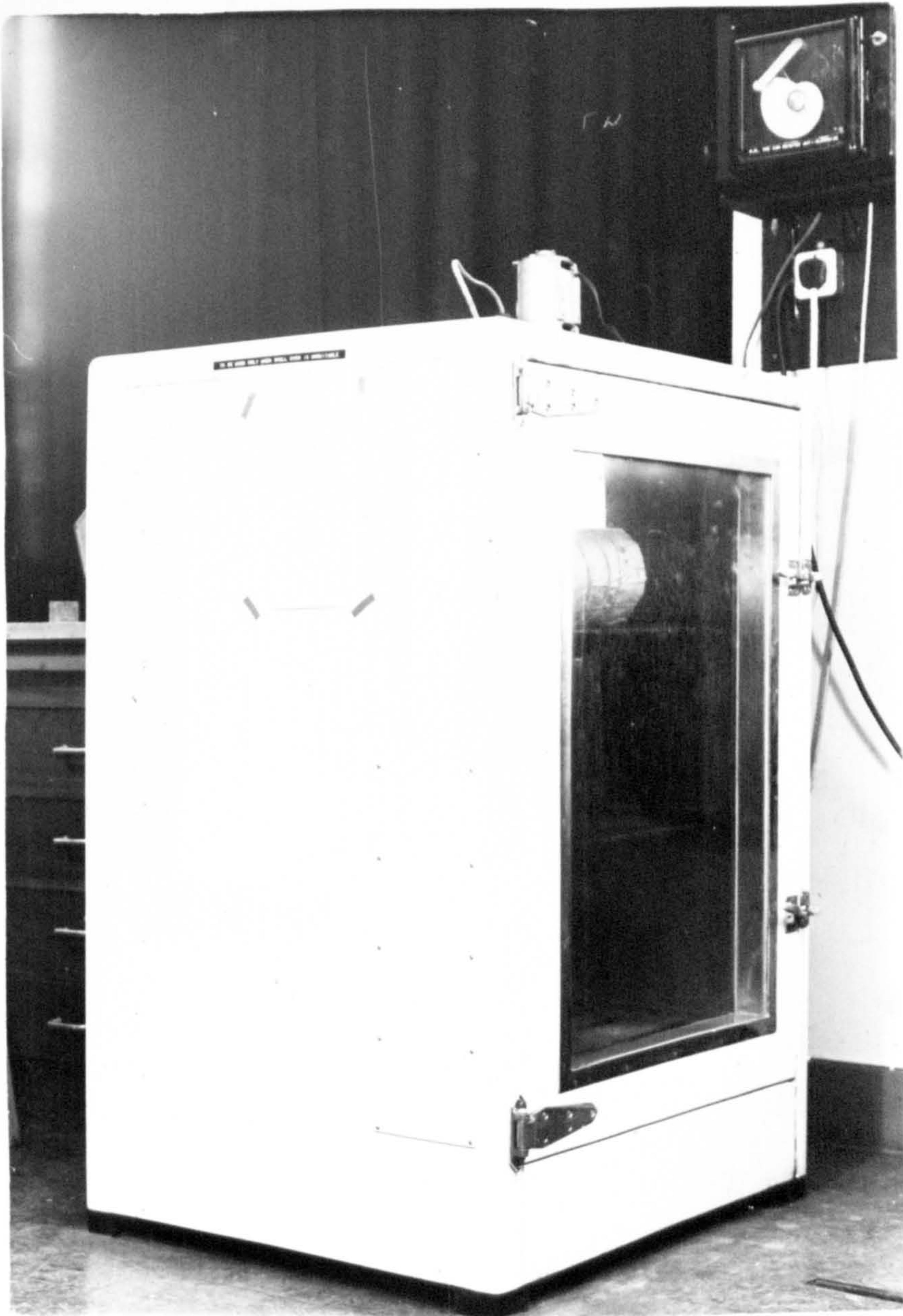


FIG. 44 ETHER FURNACE & SHRINKAGE
SPECIMEN

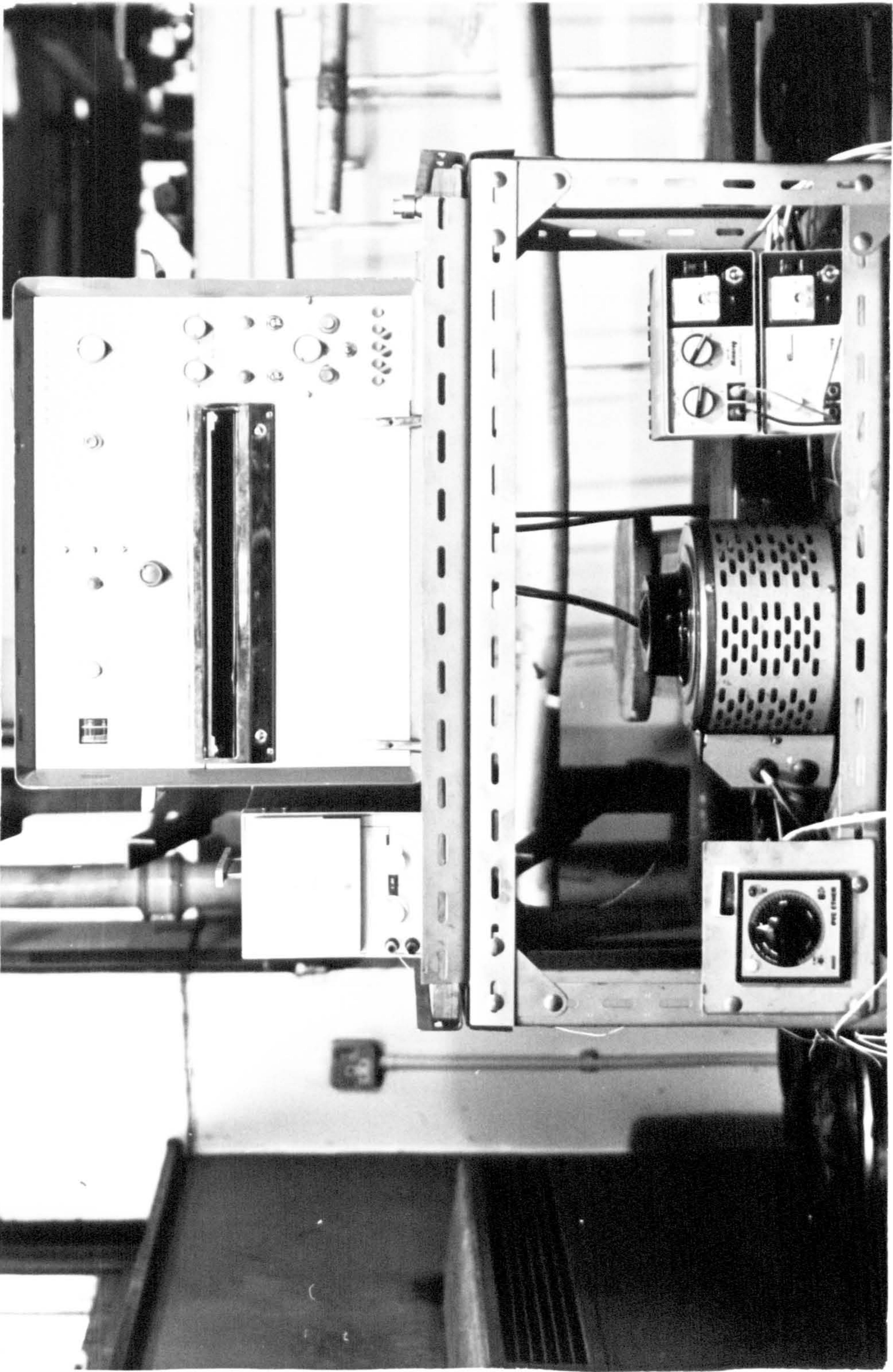


FIG. 4.5 CONTROL AND MONITORING UNITS

Chapter 5

TEST SPECIMENS

5.1 General Remarks

The purpose of the experimental project was to perform thermal creep, shrinkage and expansion tests on concrete cylinders of 6 in. diameter 12 in. long, at a time after casting, when the phenomenon of drying shrinkage, moisture loss and strength of concrete had ceased to affect the test results appreciably, at the same time it was desired to observe the performance of the apparatus and in particular the pressure cell. The specimens were ground and the voids filled, then polished and sprayed to fit into the pressure cell without difficulty.

The specimens were made of a good-quality structural concrete, to be tested later than six months after casting. A 28-day cube strength of 7400 psi. was anticipated

5.2 The Concrete Mix

One concrete mix was used throughout this investigation. Both the two sands and the coarse aggregate were irregular Thames Valley gravel. The relevant sieve sizes are entered in table 5.1, in which the mix proportions are listed.

The aggregates, both fine and coarse, were sieved, washed and hot air dried prior to storage in large bins. The bin dry aggregate contains 1.3 percent of moisture at the time of casting.

The absorption capacity of the bin dry aggregates was also determined and was used to establish the effective water-cement ratio of the mix.

The cement was ordinary blue circle Portland cement. The cement was stored in two 4000 kgm. capacity dry cement silos and was well blended. The quantity was sufficient to supply the laboratory requirements for about four months which was within the casting period of the specimens tested.

Table 5.1 Mix Proportions

Particulars	Weight
Ordinary Portland Cement	1.00
Fine sand (100-25 B.S.)	0.58
Coarse sand (25 B.S. - 3/16 in.)	1.43
Coarse aggregate (3/16-3/8 in.)	1.32
Coarse aggregate (3/8-3/4 in.)	2.61
Water	0.50

5.3 Mixing Procedure

The concrete mix was manufactured in two batches. First batch manufactured in 0.05 m^3 . Capacity flat bottomed concrete mixing machine with counter rotating blades (Eirich pan mixer). The required quantities of materials (for 0.043 cubic meter of wet concrete) were weighed. The aggregate and cement were dry mixed for 0.5 minute. Then, the required water was added and the mixing machine was run for further 3.0 minutes.

Since the amount of material for the second batch was larger than 0.05 cubic meter, the mixing was done in 0.1133 cubic meter capacity flat bottomed concrete mixing machine with counter rotating blades. The required quantities of materials (for 0.055 cubic meter of wet concrete) were weigh batched by means of a 100-kgm capacity Pfister Waagen batcher. The aggregates and cement were then directly discharged into the pan of the mixing machine. The constituents were dry mixed for 0.5 minute. Then, the required water was added and the mixing machine run for further 3.0 minutes.

At the completion of the mixing, the entire content was discharged into a wheel barrow and brought near the casting place where tests on the wet mix were performed prior to casting.

5.4 Casting of Specimens

Specimens for thermal creep test, were cast in the first batch which consisted of six cylinders together with control specimens consisting of six, 4 inch cubes, the cylindrical specimens were provided with six thermo couples embedded in each of them.

The second batch specimens consisted of six cylindrical specimens together with twelve, 4 inch cubes, the cubes were used as control specimens, whereas the cylindrical specimen were used in thermal shrinkage and thermal expansion tests. The time between the two consecutive casting was fourteen days so that the specimens could be tested at the desired ages.

The concrete in the cylindrical mould was compacted on a vibrating table with moulds filled to a third of the volume in three stages. The control specimens were similarly filled and compacted in three layers by using the same vibrating table. The compaction was carried out with the frequency of vibration held at around 3000 cycles per minute.

After compaction, the top surfaces were trowelled to a smooth finish. The specimens were then left wrapped around with polythene sheets. One day after casting, the specimens were demoulded, identified, and were left for curing.

5.5 Curing of the Specimens

One curing regime was adopted throughout this investigation. The specimens were water cured, then submerged in watertank after demoulding and left there for seven days. They were then stored in the laboratory at 20°C and 50 percent relative humidity until they were required for testing.

5.6 Surface Preparation

Prior to triaxial thermal creep testing, and after the specimens have been in air for some time, the plane surfaces of the cylinders were ground accurately plane and parallel using a specially adapted surface grinding machine fitted with a diamond-impregnated lens polishing cupwheel. The curved surfaces of the cylinders are accurately ground to a nominal 5.935 in. diameter over the full length, using a lathe fitted with a tool-post grinder and diamond-impregnated grinding wheel. The grinding over the full length required a special designed fitting

discs as shown in figure (5.1), to hold the cylinder with the grinding machine while the grinding process going on. The grinding process was carried out using water as a coolant in sufficient quantities such that no significant thermal gradients were induced which would be liable to affect the properties of the material.

Surface air voids which had been exposed after grinding, were filled with ceramic cement 'Autostic', this filler has the advantage of high quality performance at elevated temperatures. Any surplus hardened filler is removed by abrasion, to ensure that the voids contain ceramic cement are only up to the surface of the specimen.

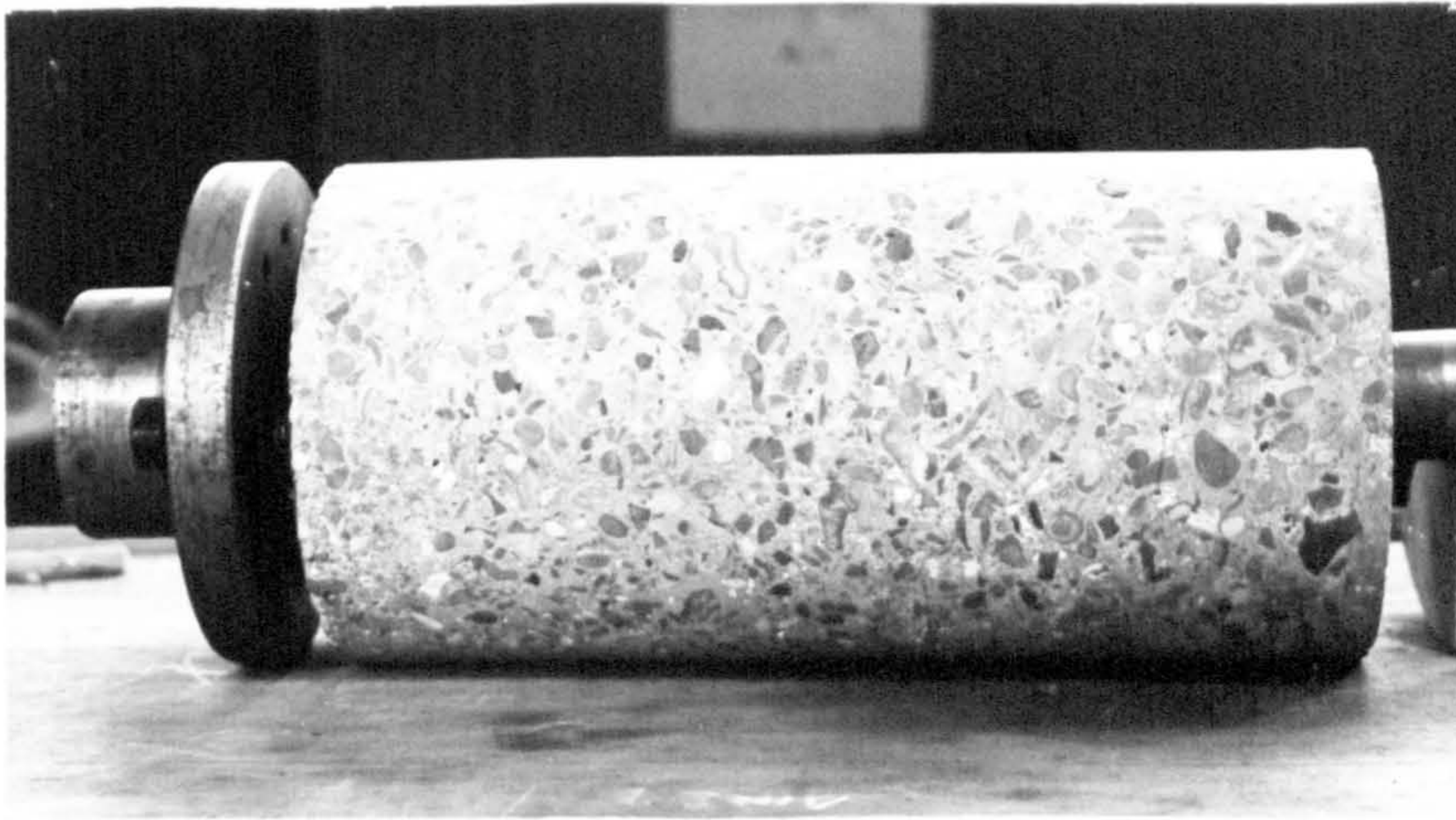


FIG . 5.1 ARRANGEMENT OF THE DESIGNED
FITTING DISCS

Chapter 6

DESCRIPTION OF TESTS

6.1 General Remarks

The test carried out, can be classified under:

- i) Preliminary tests,
- ii) Calibration tests,
- iii) Main tests.

Before the main tests were started, a series of preliminary tests were carried out to check the performance of the test rig with particular attention to the heating and the displacement monitoring equipment. This was followed by calibration of the transducers, the load cell and the thermocouples.

6.2 Preliminary Tests

The preliminary tests indicated that during heating, the required temperatures are influenced by the voltage supply and ambient temperature, the difference in temperatures between the specimen surfaces and centre nearly disappears after twenty hours of the heating. Therefore, provided that the ambient temperature is controlled, the graph in figure (6.1) which is derived experimentally can forecast the temperature in the specimen from the voltage being supplied to the heating tape.

The syndanyo discs are too soft as intermediate material between the specimen and platens, when an external measuring system is adopted

There was some concern about the oil (Earo Turbine Oil) being used, in particular the flash point, however, preliminary tests proved that fire risk was remote due to two reasons;

1. The oil will not ignite even if the flash point is reached.

2. The safety measures included in the apparatus design are unlikely to bring the oil temperature near the flash point.

6.3 Calibration Tests

Individual calibrations of each different unit as well as the combined calibration of the entire system with all units connected together were carried out.

6.3.1 Calibration of the Vertical Transducer

The transducer was mounted on a jig, while the travel of the spindle was controlled by micrometer head. The input power was 24 volts d.c. The transducer connected to damping variable resistor and the output was monitored on the recording paper of the u.v. recorder.

As displacements exceeding 0.05 in. were not expected to develop during high temperature creep tests, the transducer was calibrated within the range of 0.0 to 0.05 in. The calibration was performed in steps of 0.005 in. for the entire range. The traces were projected over 25 cm. of the recording paper with non linearity not greater than $\pm 1\%$ over 6 cm. deflection. The projected range can be varied either by turning the resistor

knob or replacing the galvanometer concerned, however, the accuracy obtained from this arrangement was ± 8 micro strain, figure (6.2) shows the calibration results which seem to be linear within the range required, also it should be noted that the transducer has to be balanced before carrying out the calibration test, this was done by connecting the transducer to a digital voltmeter and moving the spindle in or out until zero output is obtained, and the spindle length should not be increased beyond that, thus optimum use of the transducer linearity has been achieved.

6.3.2 Calibration of Horizontal Transducers

The horizontal transducers were used to monitor the radial displacements of the specimen which have been transmitted by two Invar steel probe rods at the middle height of the specimen passing through the pressure cell by a specially designed compartment (see 3.4.2). The horizontal transducers with a power input of 24 volts d.c. were each calibrated individually by means of a micrometer similar to the arrangement described previously in steps of 0.001 inch throughout the range of 0.0 to 0.01 with the output being recorded on 25 cm. of the sensitized print out paper of u.v. recorder.

The correlation between the transducers displacement and the deflected traces relevant were shown in figure (6.3), also figure (6.4) shows the instrumentation used in this calibration. The accuracy produced by this procedure was ± 10 micro-strain.

6.3.3 Calibration of Load Cell

The load cell was calibrated on its own by 150 tons capacity, Avery compression machine, a separate power pack provided the load cell with 5 volts d.c. input. The load cell output was connected to the u.v. recorder and the corresponding galvanometer was B. 160, which accommodated the output of the load cell without needing to damp the resistor. The load cell was placed between the platens of the testing machine. The load was increased in steps of 5000 lbs. upto 100 tons and then decreased in steps of 5000 lbs. The recorded output was almost linear, figure (6.4) shows the correlation between the applied load and the relevant deflected trace.

6.3.4 Thermocouples Calibration

A thermocoax-thermocouple was calibrated in two ways; firstly, by submerging it in water together with a copper-constantan thermocouple, the other ends of each thermocouple were connected to different terminals. The thermocoax was connected to the u.v. recorder and the copper-constantan was connected to the temperature indicator (Comark) which gives direct readings. The water was heated gradually until boiling, then left to cool down, the output of both thermocouples were recorded simultaneously. Secondly, both thermocouples were placed inside electrical oven and the temperature raised to 200°C , and again the output of both thermocouples recorded continuously, the results of these calibrations were drawn in figure (6.5), which shows that the temperature varies linearly with the relevant deflected trace on the recording paper of the u.v. recorder.

6.4 The Main Tests

Two series of tests were carried out, the triaxial thermal creep tests as listed in table 6.1, and the thermal shrinkage-expansion as listed in table 6.2.

6.4.1 Triaxial Thermal Creep Tests

Before prior testing, each specimen was sprayed by 'Rocol' which formed an invisible lubricant film on specimen surfaces, then placed vertically into the pressure cell by tapping gently on it's upper plane surface until it touches the lower loading platen. The thermocouple lead passed through a hole in the upper loading platen and connected to the u.v. recorder. The upper loading platen, sindanyo disc and spherical seating were assembled carefully to preserve vertical alignment by means of a locating pin and the male part of the spherical seating, which was screwed to the centre of the upper plate.

The temperature was brought to the desired level 'prior to loading' within twenty-four hours or even longer, which depends upon the rate of heating. In most cases, the rate of heating was around 7°C/h . The moisture was allowed to escape freely and no attempt was made to seal the specimens.

The loading path used for the tests is shown in figure (6.6). It is achieved by increasing the axial load to the required value. After reaching the required value, the radial pressure was increased to 500 psi. Both axial load and radial pressure were kept constant by the design of the pressure circuit (see 3.4.4). The maintenance of a constant loading rate

was difficult since the accumulator takes a number of minutes to reach the required loading pressure which results in relatively large increases of the specimen strain during loading.

The radial displacements, the axial displacements and the temperature were measured and recorded continuously throughout the ten days period of the test.

The test was terminated by removing the radial pressure slowly. Then the axial load was removed gradually. The thermal system was switched off and the pressure cell left to cool naturally. This procedure has evolved so that the specimens can be removed from the cell without damage to the diaphragm.

After dismantling the cell and the platen assembly, the specimen is removed using an extraction device specially developed to avoid damaging the specimen and the diaphragm.

The difficulties encountered in producing a constant loading rate may be overcome by raising the oil pressure 'prior to loading', in each pressure line, to a value higher than the required pressure. After that, the loading rate can be controlled merely by a control valve and an unloading valve in each line independently. (see figure 6.7).

6.4.2 The Residual Strength

After each triaxial creep test had been carried out, the specimen was taken out of the pressure cell and tested

uniaxially by a 300 tons Amesler compression machine, the residual strength obtained from this test was compared with the ultimate strength obtained from a cylinder which had not been stressed or heated.

6.4.3 Thermal Shrinkage and Expansion

It was impossible to carry out creep and shrinkage tests simultaneously on the same specimen, therefore a separate set of the tests were designed to investigate coefficient of thermal expansion and thermal shrinkage, the latter was defined as the increase in drying shrinkage due to elevated temperature. Conditions of these tests are listed in table 6.2, each specimen was cured in similar conditions to thermal creep specimens having the same age.

The specimens were 6 in. diameter 12 in. long, provided with stainless steel Demec Pips for use with an 8 in. gauge length demountable extensometer (Demec). The specimens were enclosed in 0.005 in. stainless steel jacket and lagged with 0.5 in. layers of aluminised glass cloth insulation strips.

The thermal environment was provided by an "Ether" furnace which was equipped with an "Ether" Transitol Controller. The specimens were heated at a rate of 7°C per hour to the required temperature and kept constant for ten days.

Complete sets of strain readings were taken before and after heating every day throughout the testing period. The furnace door had to be opened for strain measurements as this

operation took nearly 30 seconds only, therefore, slight change in the environment surrounding the specimen might occurred but the thermocouples attached to the specimen did not show significant change in temperature due to insulation. After ten days the furnace was switched off and the specimen left to cool naturally.

The thermal expansion was determined from the initial and final temperature strain reading on the first day, whereas shrinkage was determined from measurements after 24 hours of heating onward.

Table 6.1 Test for Creep Strain under Multiaxial Stress

Test No	Nominal Temperature °C	Stress lb/in ²		Time (Days)
		Axial	Radial	
1	125	0.3 σ_u	500	10
2	150	"	"	"
3	175	"	"	"
4	125	0.5 σ_u	500	10
5	150	"	"	"
6	175	"	"	"

Table 6.2 Test for Thermal Shrinkage of Concrete

Test No	Nominal Temperature °C	Stress lb/in ²	Time (Days)
1	125	0	10
2	150	0	"
3	175	0	"
4	125	"	"
5	150	"	"
6	175	"	"

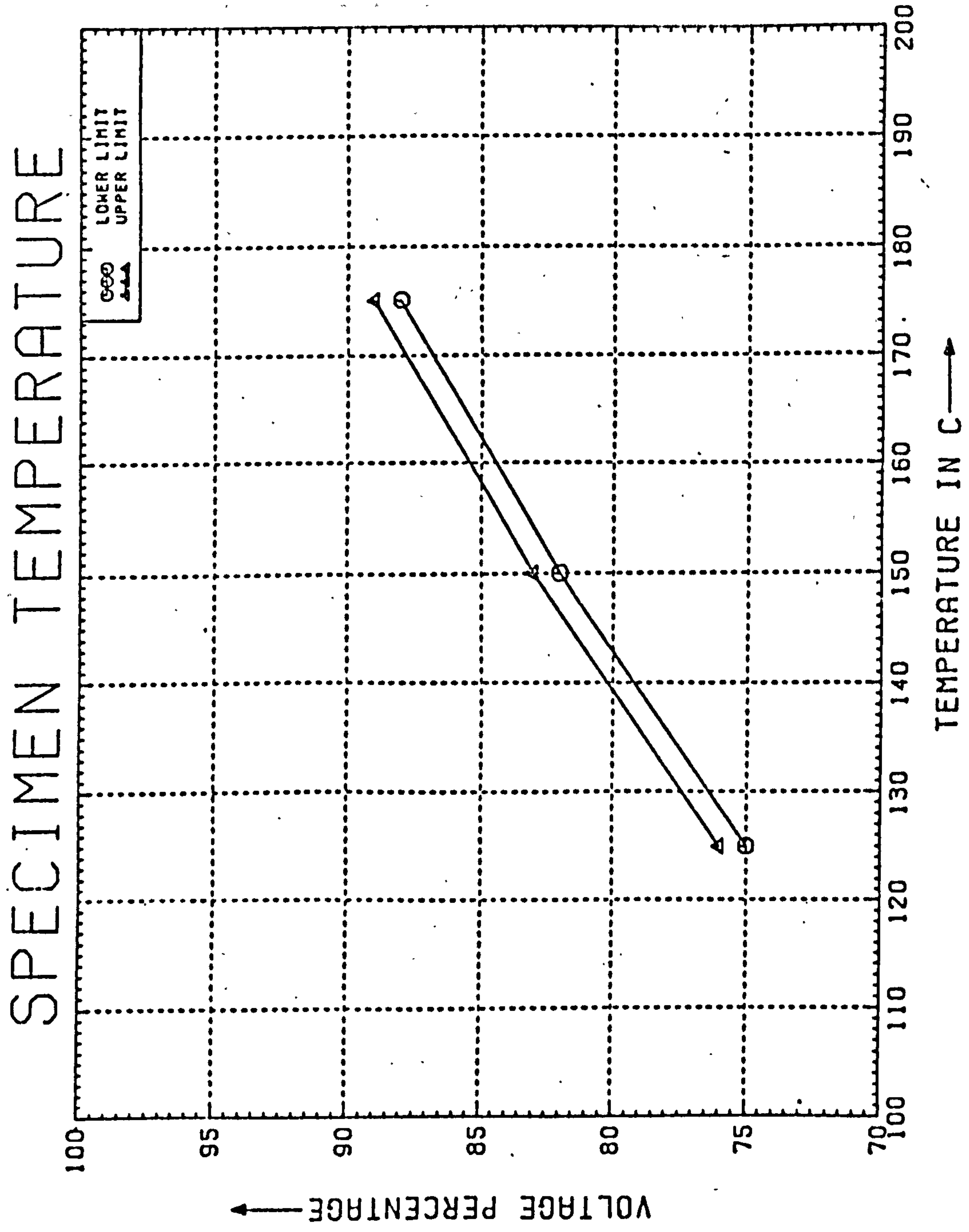


FIG. 6.1

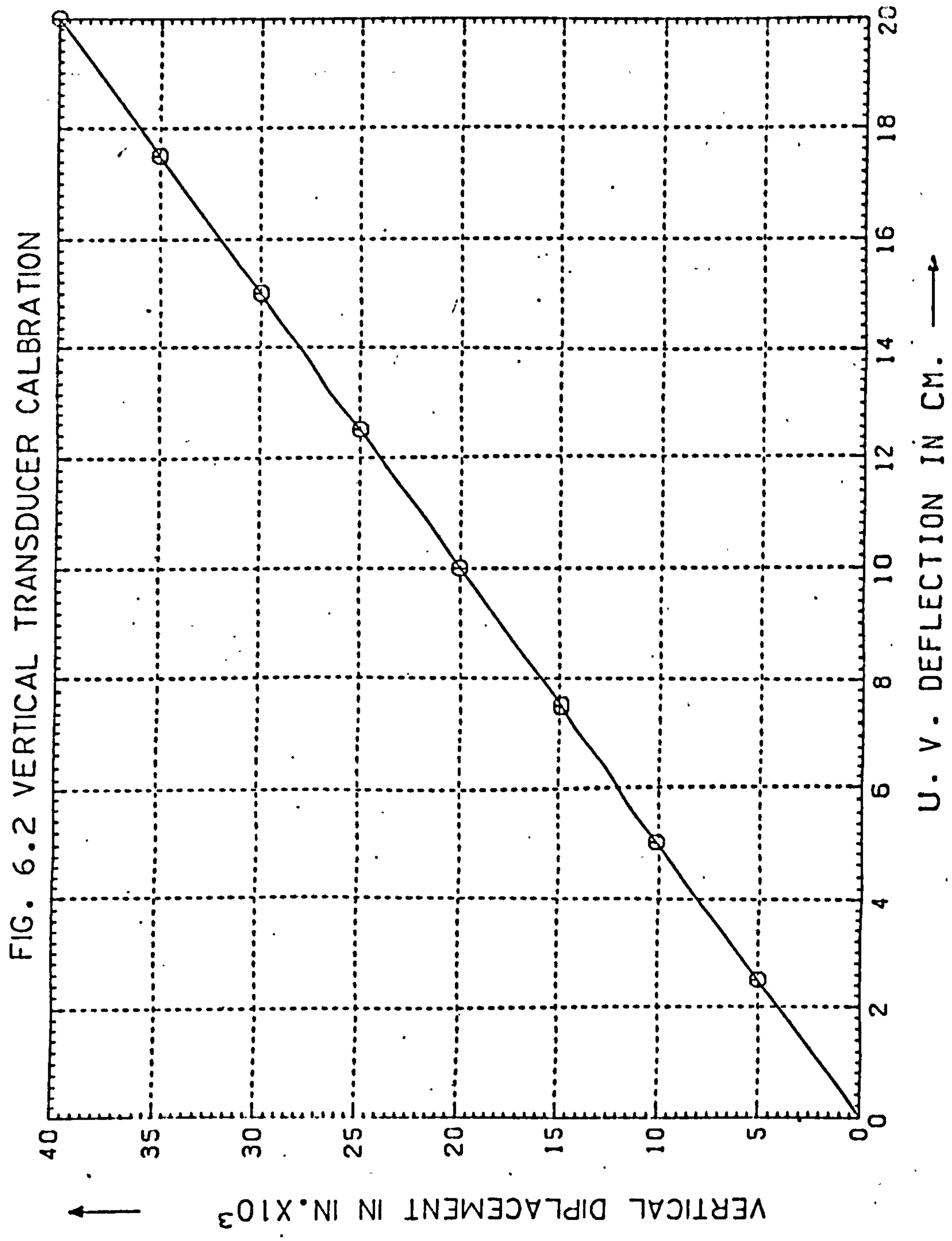
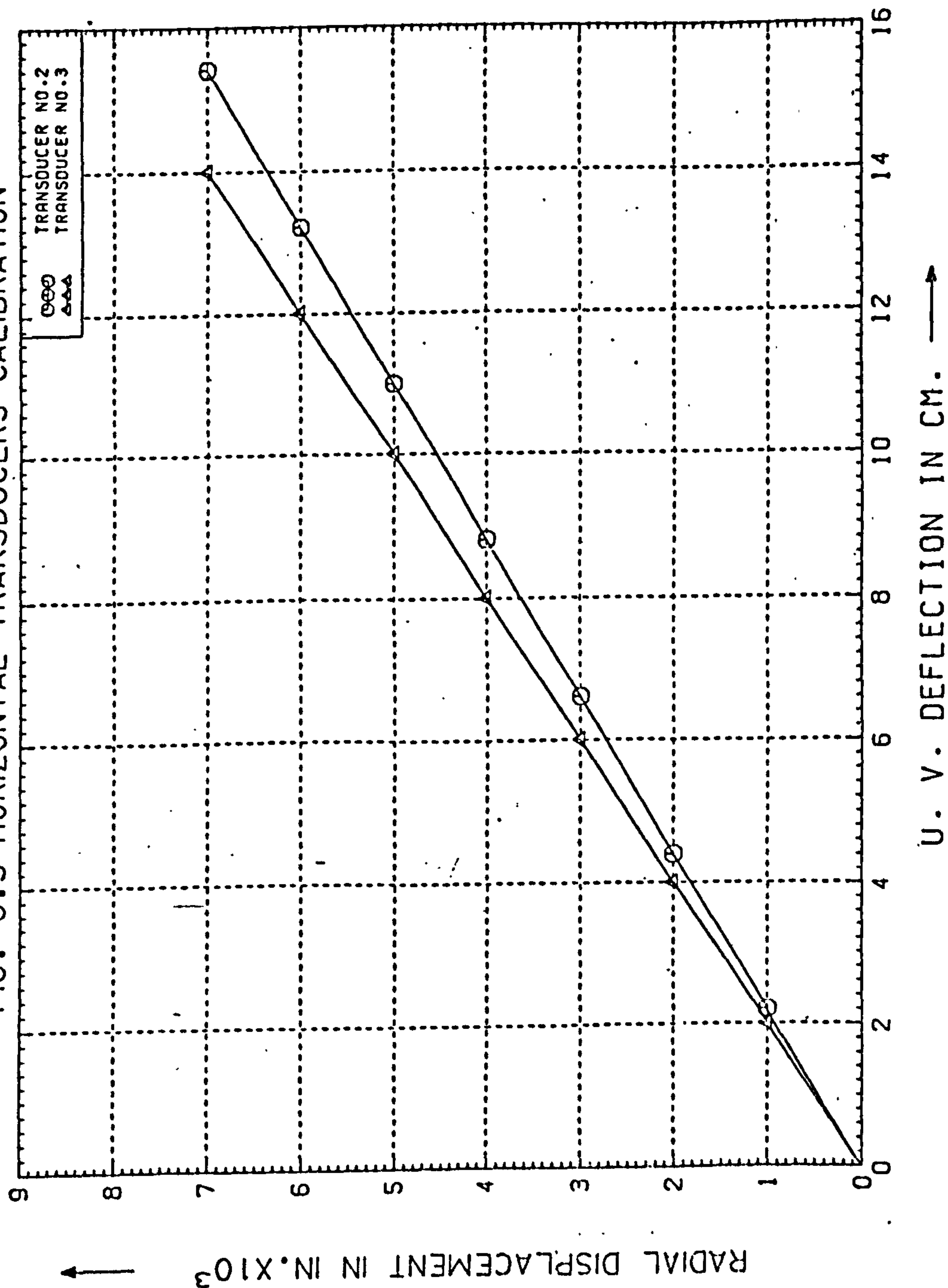
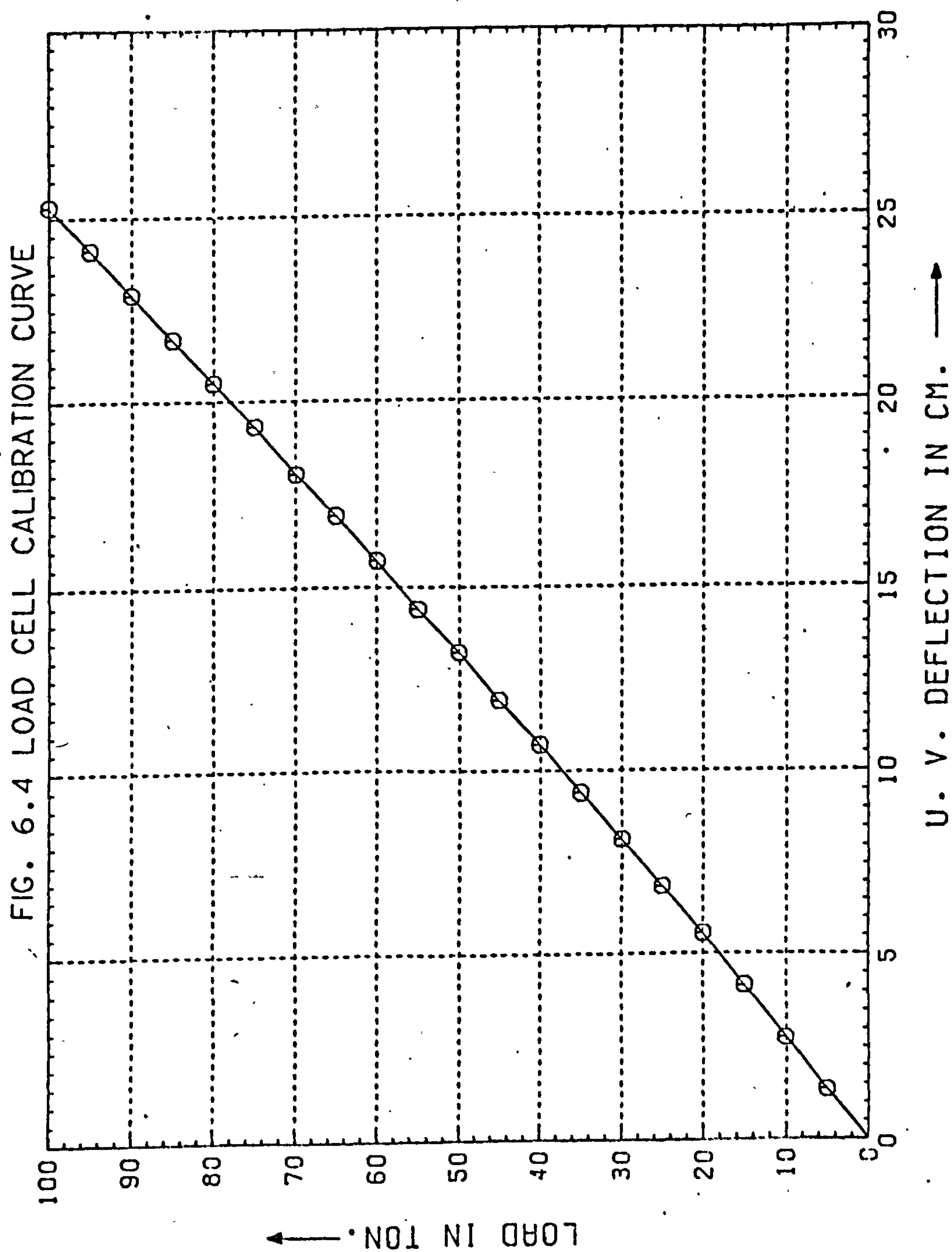
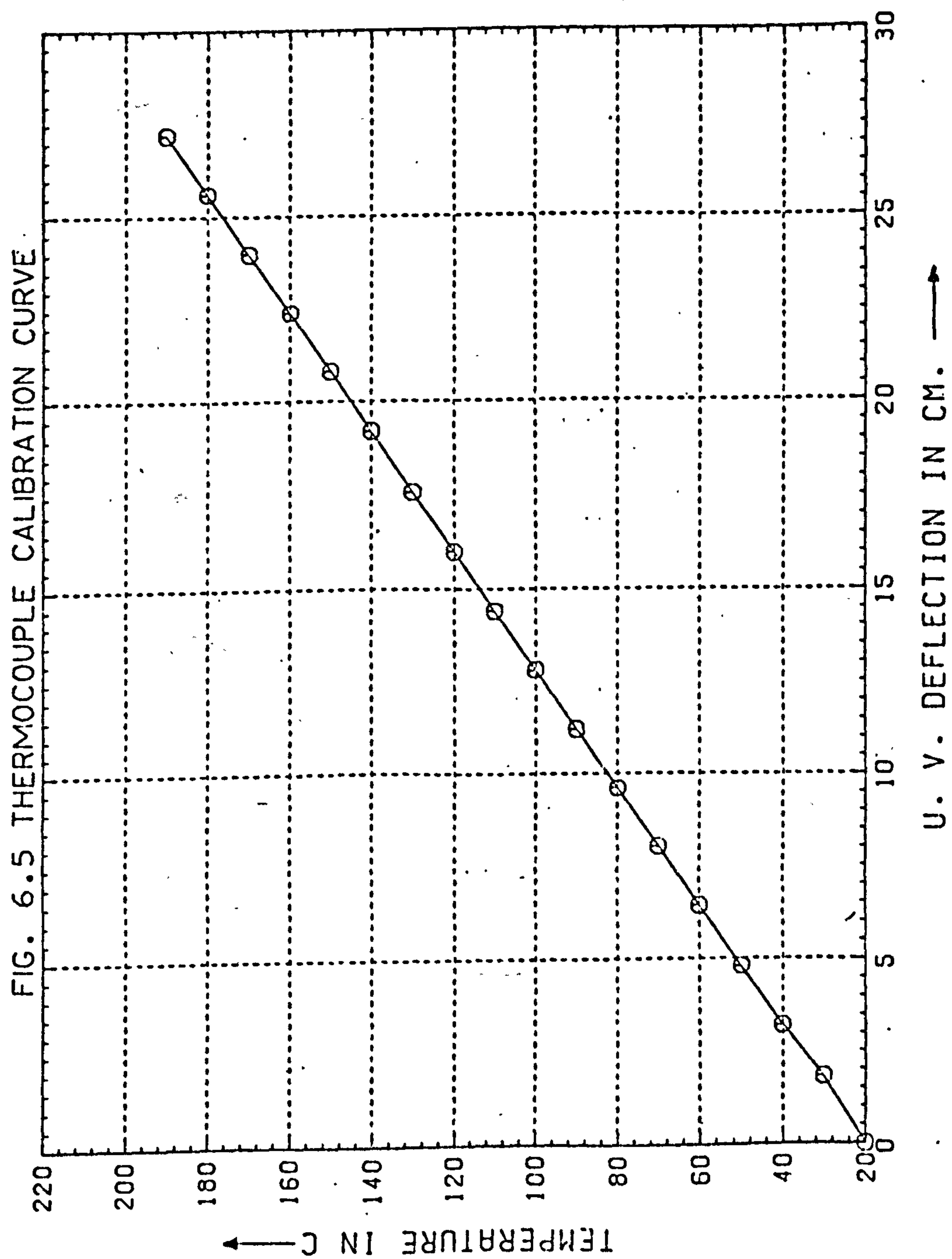


FIG. 6.3 HORIZONTAL TRANSDUCERS CALIBRATION







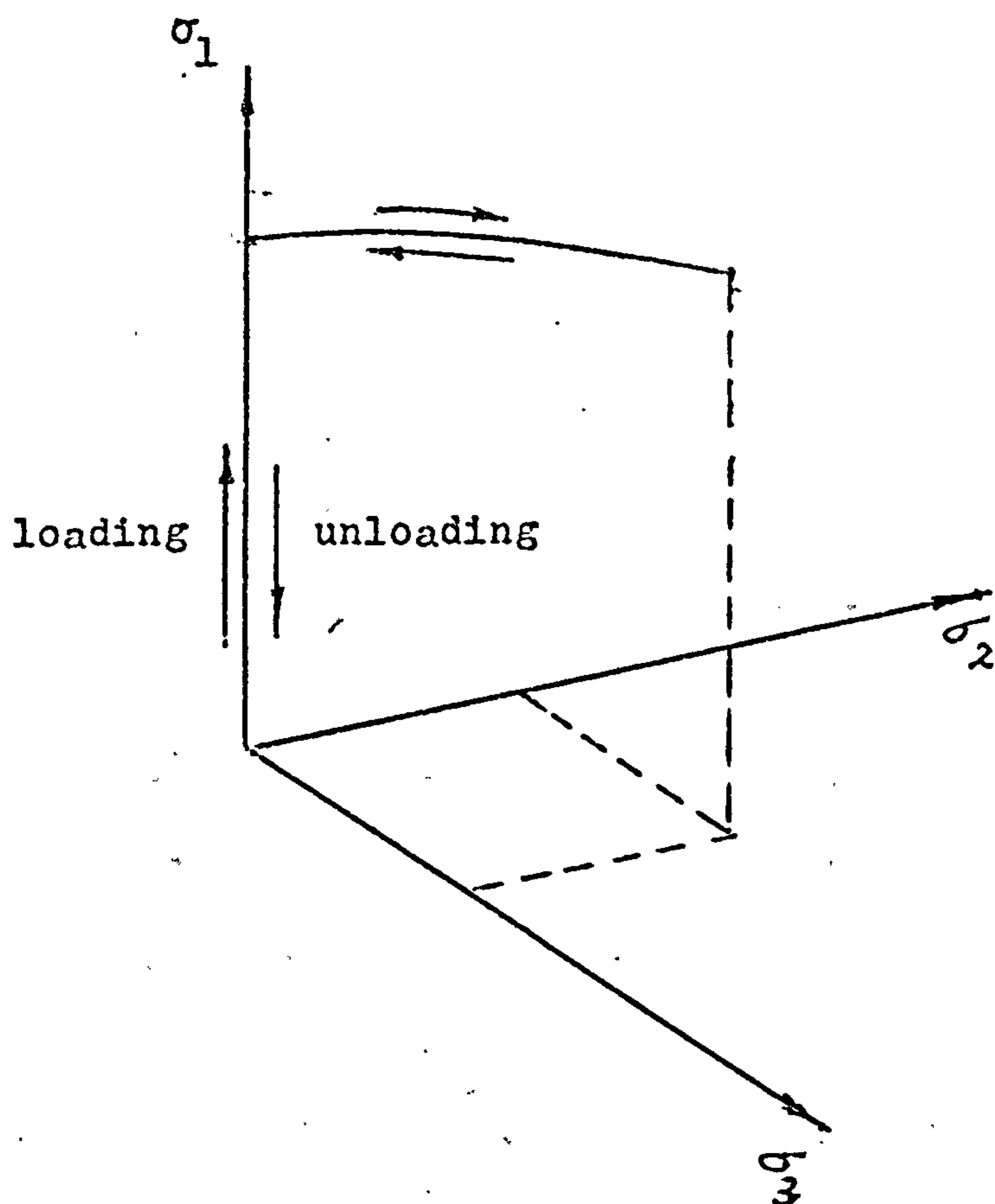
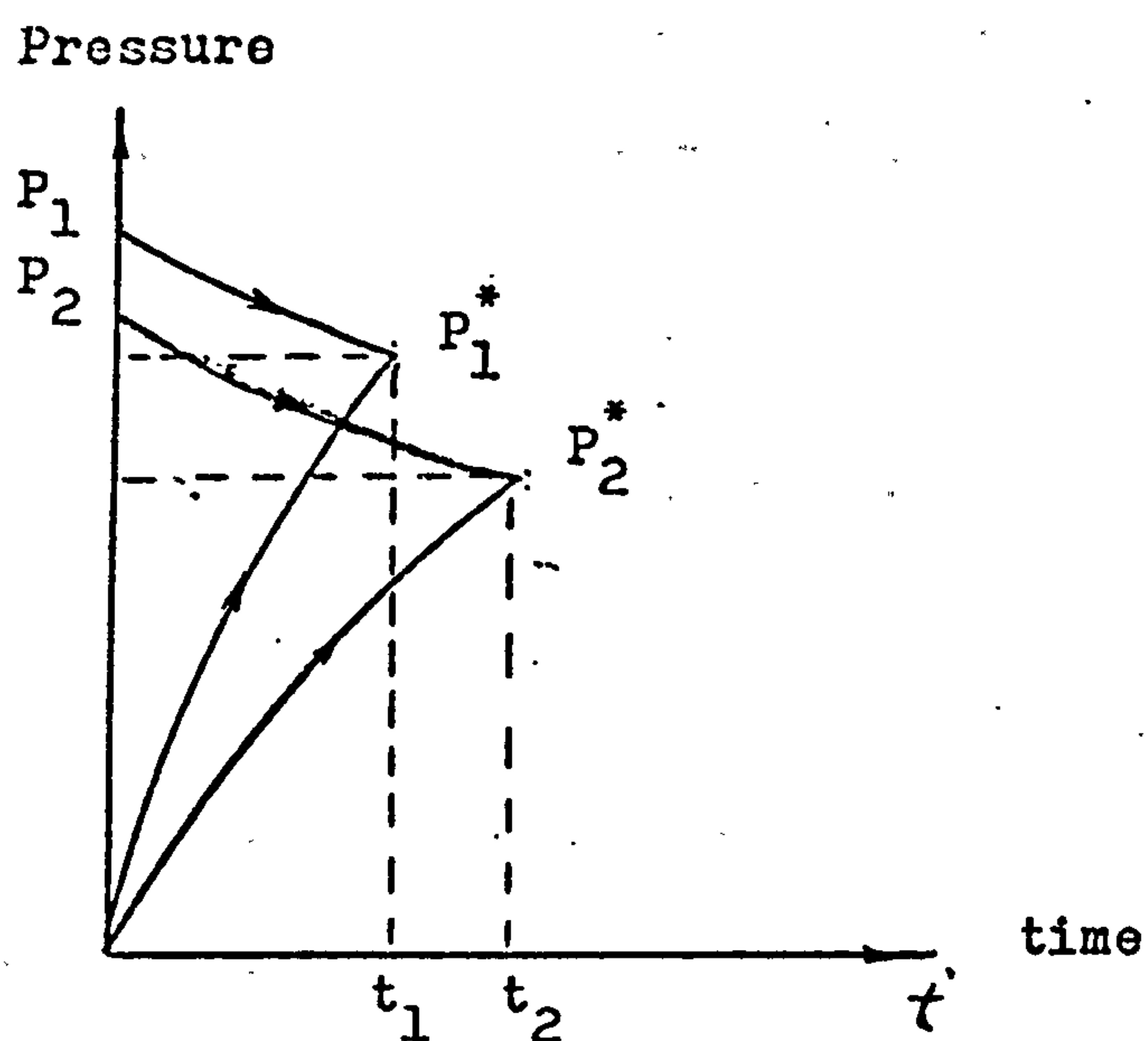


Figure 6.6 Loading Path



- P_1 is the pressure in the axial loading line.
- P_1^* is the required pressure in the flat-jack.
- P_2^* is the pressure in the radial loading line.
- P_2 is the required pressure in the pressure cell.
- t_1 is the required time to produce P_1^* .
- t_2 is the required time to produce P_2^* .

Figure 6.7 Pressurising Procedure

Chapter 7

TEST RESULTS AND DISCUSSION

7.1 Series 1

Figures (7.1) and (7.2), show the measured total deformation "elastic and creep" values plotted against time at the three temperature levels and at the two stress levels. It is also apparent that the creep magnitude is dependent on the test temperature.

Figure (7.1) shows the creep test at 175°C, in which the confining pressure was released after four days as a result of bellows failure and the test went on with the axial stress. A similar incident can be seen in figure (7.2) at 150°C, in which the test was terminated after five days.

Corrections were also made for the shrinkage in the loaded specimens where similar water loss had occurred. See figure (7.9).

The residual uniaxial strength of loaded and unloaded specimens are shown in figure (7.6), as a percentage of the ultimate uniaxial cold strength ($f_{cy} = 81 \text{ } \overset{n}{\text{ton}}\text{es}$). The results generally show a decrease in strength with an increase in temperature.

The X-ray diffraction photographs show definite differences between three samples representing various conditions, in which sample No.1 was unheated and unstressed, sample No.2 was

heated and unstressed and sample No. 3 was heated and stressed. It emerges that sample No. 3 contains a considerable amount of free hydrated lime, $C_a(OH)_2$, which is not present in either of the other two to a detachable extent. The other two samples, No. 1 and No. 2, contain some calcium carbonate, which is also present to a smaller extent in No. 3. These chemical changes probably have some influence on both strength and creep of concrete.

The following relations are proposed to describe the thermal creep under multiaxial state of stresses, provided that stress does not exceed 2000 psi. See fig. 7.9.

$$\varepsilon_{ca} = \frac{1}{10} t^{0.31} \sigma_a \left[\left(\frac{|125 - T|}{1000} \times t \right)^{0.6} \pm 1 \right] \quad (7.1.1)$$

where

ε_{ca} is the axial creep strain in micro strain,

ε_{cr} is the radial creep strain in micro strain,

t is the loading time in days,

$$\sigma_a = \sigma_1 - \nu(\sigma_2 + \sigma_3), \quad (7.1.2)$$

$\sigma_1, \sigma_2, \sigma_3$ are the principal stresses in psi,

T is the working temperature above 100°C .

Sign is + if $T \geq 125^\circ\text{C}$

- if $T < 125^\circ\text{C}$

$$\sigma_r = \sigma_2 - \nu(\sigma_1 + \sigma_3) \quad (7.1.3)$$

ν is Poisson's ratio.

for cylinder $\sigma_2 = \sigma_3$

To facilitate comparison, the results described by various investigators together with the results obtained here are expressed in terms of specific creep and subsequently compiled in table 7.2, and plotted in figure 7.7. Therefore, it is necessary to introduce the following definition for specific creep under multiaxial state of stresses at specified temperature;

$$\text{Specific Creep} = \frac{\text{Creep Strain}}{\text{Effective Stress}} \quad (7.1.4)$$

Where,

effective stress is defined in (7.1.2) and (7.1.3)

It is apparent from figure (7.7) that our results are about 30% less than those of Cruz (129), Gross (39) and Schneider (40). The main reasons could be; different environment, specimen size and the most important is the influence of radial pressure in reducing the creep, whereas Marechal data represents a low limit due to the fact that his specimens were predried in the oven for one month, prior testing, so that the moisture had been almost completely removed and this reduces creep.

The Modulus of elasticity and the creep Poisson's ratio are derived from the theory of elasticity as follows;

$$\epsilon_a = \frac{1}{E} [\sigma_1 - 2\nu(\sigma_2)], \quad \sigma_2 = \sigma_3 \quad (7.1.5)$$

$$\epsilon_r = \frac{1}{E} [\sigma_2 - \nu(\sigma_2 + \sigma_1)] \quad (7.1.6)$$

dividing ϵ_r by ϵ_a we obtain,

$$\frac{\epsilon_r}{\epsilon_a} = \lambda = \left[\frac{\sigma_2 - \nu(\sigma_2 + \sigma_1)}{\sigma_1 - 2\nu\sigma_2} \right]$$

from which we obtain

$$\nu = \frac{\sigma_1 \lambda - \sigma_2}{2 \sigma_2 \lambda - (\sigma_1 + \sigma_2)} \quad (7.1.7)$$

having determined ν from (7.1.7), E then could be determined from either (7.1.5) or (7.1.6) as follows;

$$E = \frac{1}{\epsilon_a} (\sigma_1 - 2 \nu \sigma_2)$$

The secant Modulus of elasticity and the secant elastic Poisson's ratio are listed in table (7.1). It seems that the value of Modulus of elasticity has been reduced with increasing temperature from its value 4.46×10^{-6} lb/in² at room temperature.

Equation (7.1.8) is proposed to describe the change in the secant Modulus of elasticity due to temperature effects within the tests range

$$E_T = E_{20} \left[1 - \left(\frac{T - 125}{1000} \right)^{0.48} \right] \quad (7.1.8)$$

where

E_T is the value of secant Modulus of elasticity at temperature T .

E_{20} is the value of secant Modulus of elasticity at temperature 20°C.

If creep strains, instead of instantaneous elastic strains, are used for the axial and the radial directions in the equation (7.1.7), a form of creep Poisson's ratio is determined for cases in which the radial stress is zero, i.e., uniaxial

loading, this equation reduces to the conventional definition of Poisson's ratio.

It was found that creep strain was proportional to stress for the concrete and conditions of this investigation with the creep proportional being approximately 2000 psi. The 7.1.7 equation assumes that strains in a particular direction that are caused by multiaxial loadings may be superposed. The principle of superposition has been used by other investigators (1, 2, 39, 42), but Gopalakrishnan et al (130) stated that creep strains cannot be simply superposed under multiaxial compression; nevertheless they used an expression similar to the one above to calculate an effective creep Poisson's ratio. The data obtained in this investigation were inconclusive with regard to the validity of applying superposition up to 3200 psi.

Creep Poisson's ratio values, calculated for air-dried specimens, loaded at 125°, 150° and 175°C are shown in figures (7.4) and (7.5).

There is no significant effect caused by temperature, although it appears that creep Poisson's ratio is slightly higher for specimens loaded at 175°C in the first three days, whereas for a longer period this slight increase disappears. However, the increase in the axial stress within the test limit also does not increase the creep Poisson's ratio.

7.2 Series II

Table 7.3 lists the coefficient of thermal expansion

for the concrete used in this investigation.

Figure (7.9) shows the measured shrinkage within the ten days period. The values are averaged on the two specimens and used to correct the measured thermal creep.

The results of this series together with the steady state thermal creep, could be used to estimate the transient thermal creep within approximately the test range.

The simple subtraction of the thermal creep and the thermal shrinkage may imply a slight error due to the fact that the latter induces stress which tends to increase creep.

The thermal shrinkage within the first day of heating can be estimated from the following;

$$\epsilon_{tr} = \epsilon_{ts} - \epsilon_{st}$$

where

ϵ_{ts} is the total thermal shrinkage within 11 days.

ϵ_{tr} is the transient thermal shrinkage prior reaching the steady state

ϵ_{st} is the measured steady state thermal shrinkage and this is shown in figure 7.9.

The values ϵ_{ts} are the difference between two readings at normal temperature, prior and after heating, these values are given in table 7.4. The influence of shrinkage on the co-

efficient of thermal expansion is to reduce it (see table 7.5)

7.3 Concluding Remarks

The following remarks only apply to the particular concrete examined which was loaded triaxially in compression and air dried.

1- Creep strain of concrete subjected to multiaxial compressive stress increase with temperature. This increase is non-linear within the range of the experiments.

2- Creep strain is linearly proportional to the effective stress up to 2000 lb/in².

3- The radial pressure reduces the creep in the axial direction.

4- The residual uniaxial strength and the Modulus of elasticity are reduced with increasing temperature.

5- The X-ray diffraction photographs reveal chemical changes in the thermal creep specimen.

6- Poisson's ratio for creep is similar in magnitude to elastic Poisson's ratio.

7- Creep Poisson's ratio did not change significantly with temperature or time of testing.

Table 7.1 Secant Modulus of Elasticity and Poisson's Ratio

Test No.	T. °C	σ_1 psi	σ_2 psi	$\epsilon_1 \times 10^6$	$\epsilon_2 \times 10^6$	$E \times 10^{-6}$	ν
1	125	1900	500	531	80	3.01	0.30
2	150	3160	500	1078	180	2.69	0.26
3	125	3160	500	924	200	3.09	0.30
4	150	1900	500	597	160	2.56	0.37
5	175	1900	500	595	85	2.68	0.30
6	175	3160	500	1140	280	2.49	0.32

Table 7.2

Investigator	Concrete Strength	Type of Test	Stress/Strength Ratio	Loading Time	Aggregate Type
Gross (39)	5611 psi	Steady State	0.2	7 days	quartz
Shneider et al (40)	7800	Transient	0.3	5 days	quartz
Marechal (29)	7000 psi	Steady State	0.14	5 days	quartz
Cruz (137)	4000	Steady State	0.54	.5 hours	lime stone

Table 7.3 Thermal Expansion on the First Day

Test No.	Coef. of Thermal Expansion $\times 10^6 / ^\circ\text{C}$
1	12.6
2	11.4
3	10.35
4	12.6
5	10.12
6	11.6

Table 7.4 Transient Thermal Shrinkage

Test No.	Shrinkage $\times 10^6$
1	108
2	112
3	105
4	115
5	117
6	114

Table 7.5 Thermal Expansion after 10 Days

Test No.	Coef. of Thermal Expansion $\times 10^6 / ^\circ\text{C}$
1	12.3
2	11.02
3	10.02
4	12.22
5	9.7
6	11.18

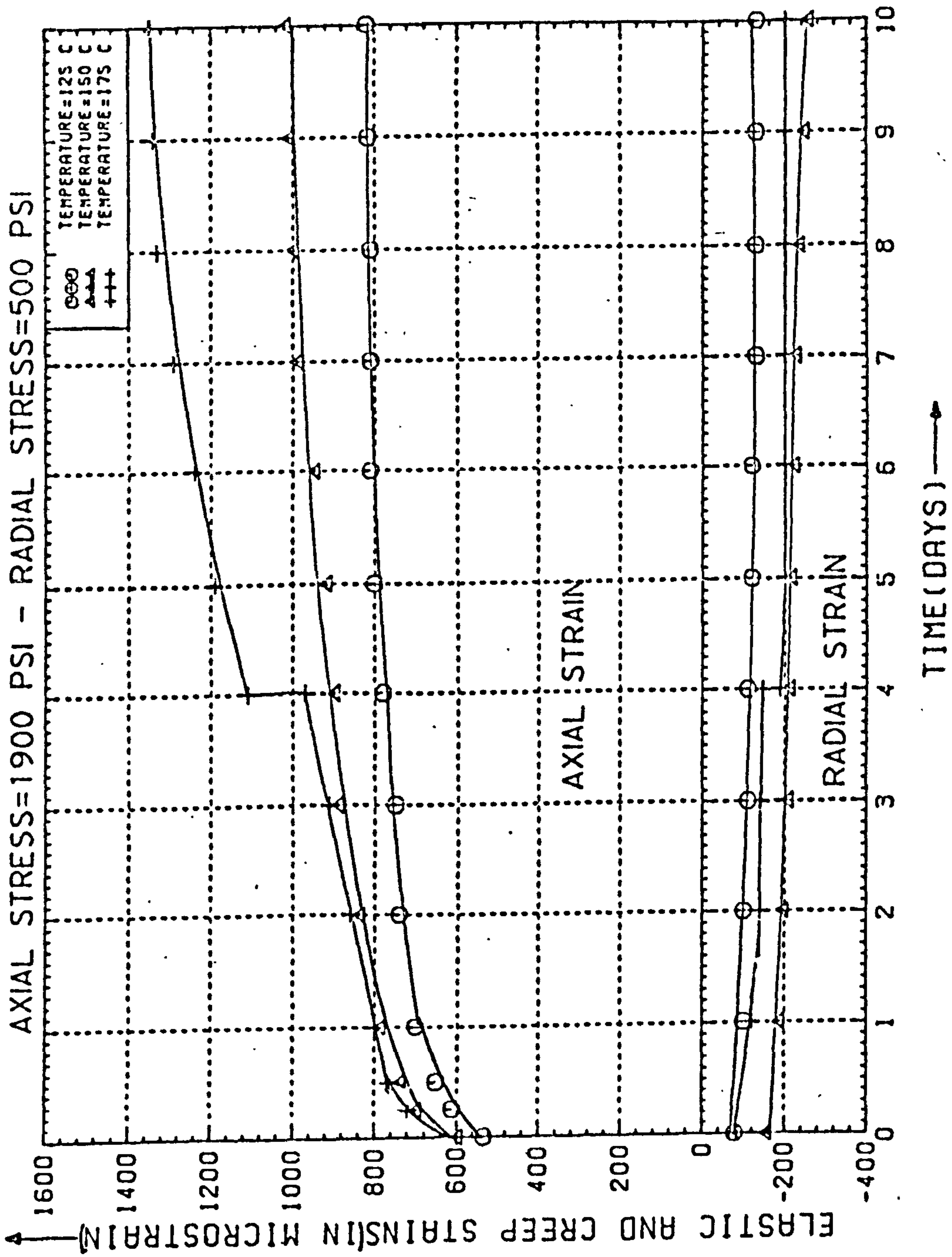


FIG.7.1 ELASTIC AND CREEP STRAINS FOR TEN DAYS PERIOD

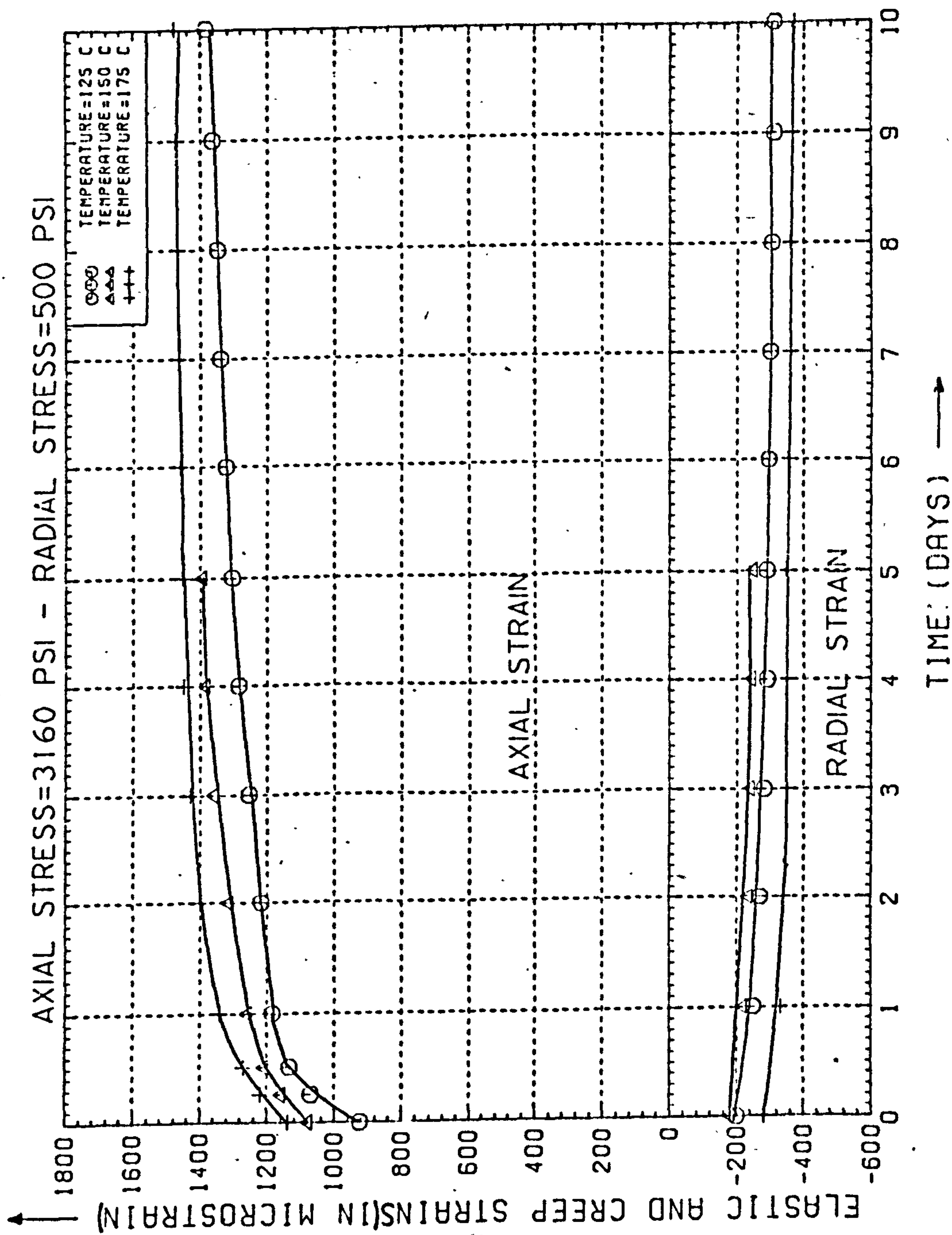


FIG.7.2 ELASTIC AND CREEP STRAINS FOR TEN DAYS PERIOD

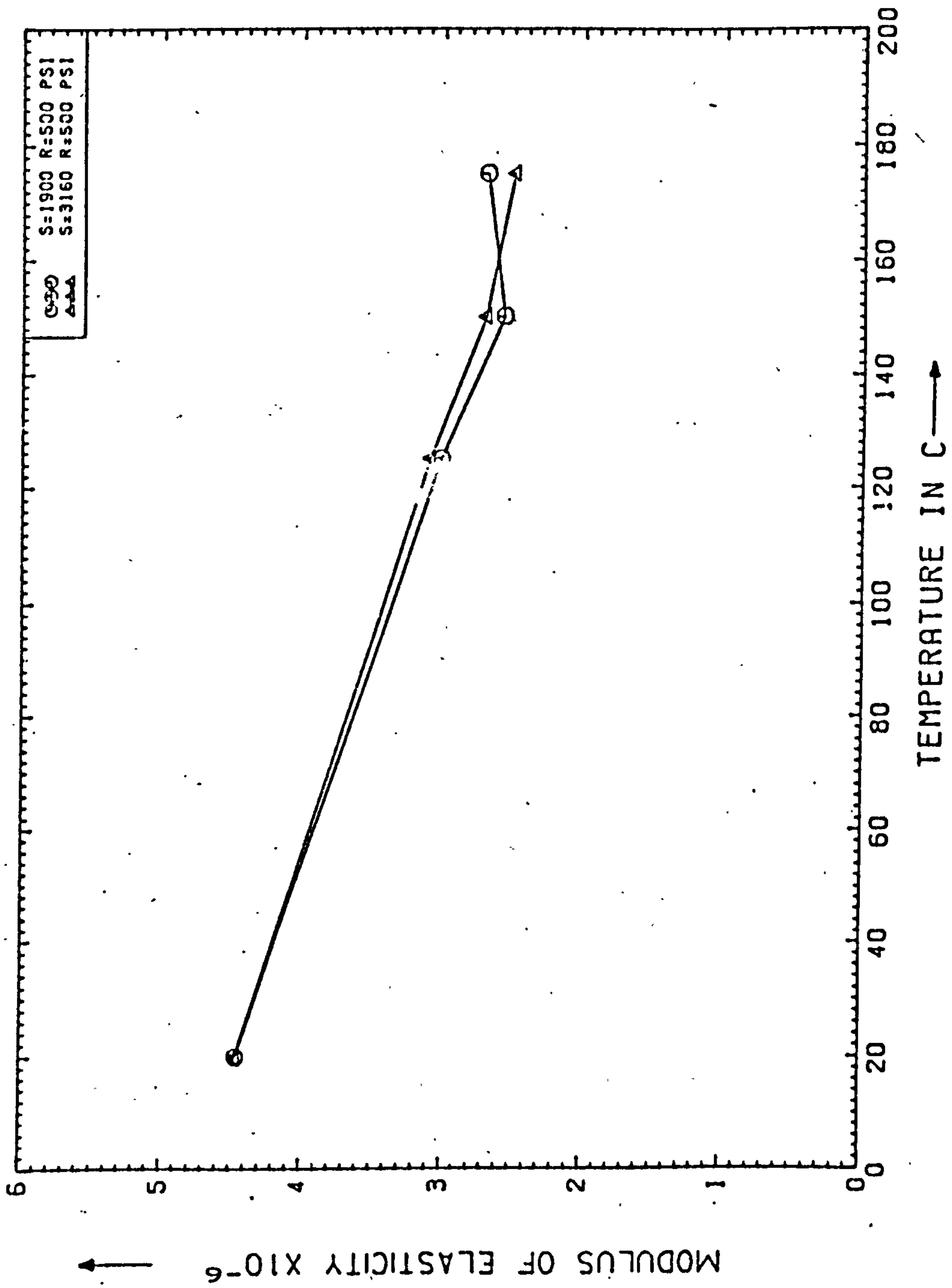


FIG-7.3a MODULUS OF ELASTICITY

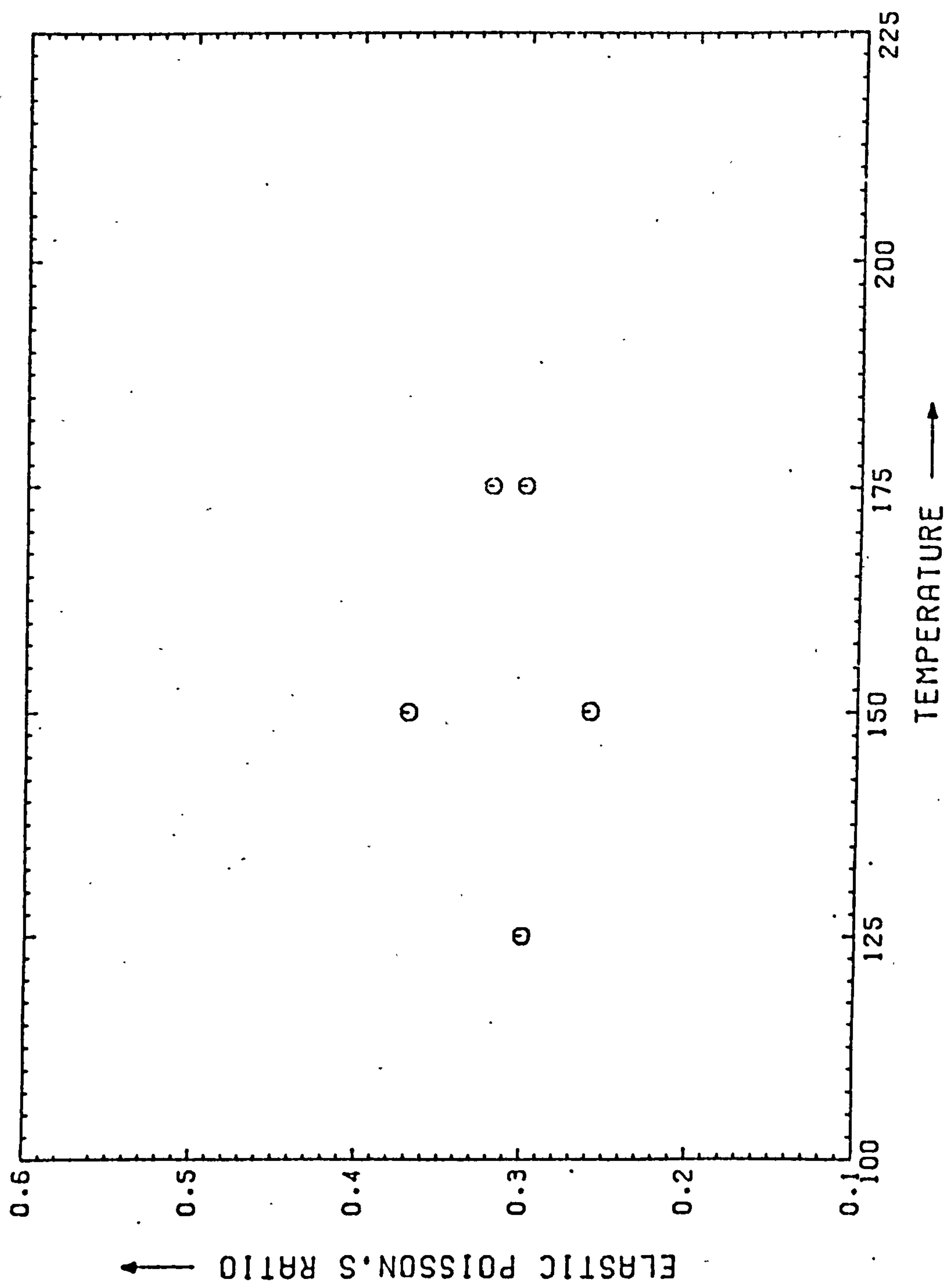


FIG. 73b ELASTIC POISSON'S RATIO

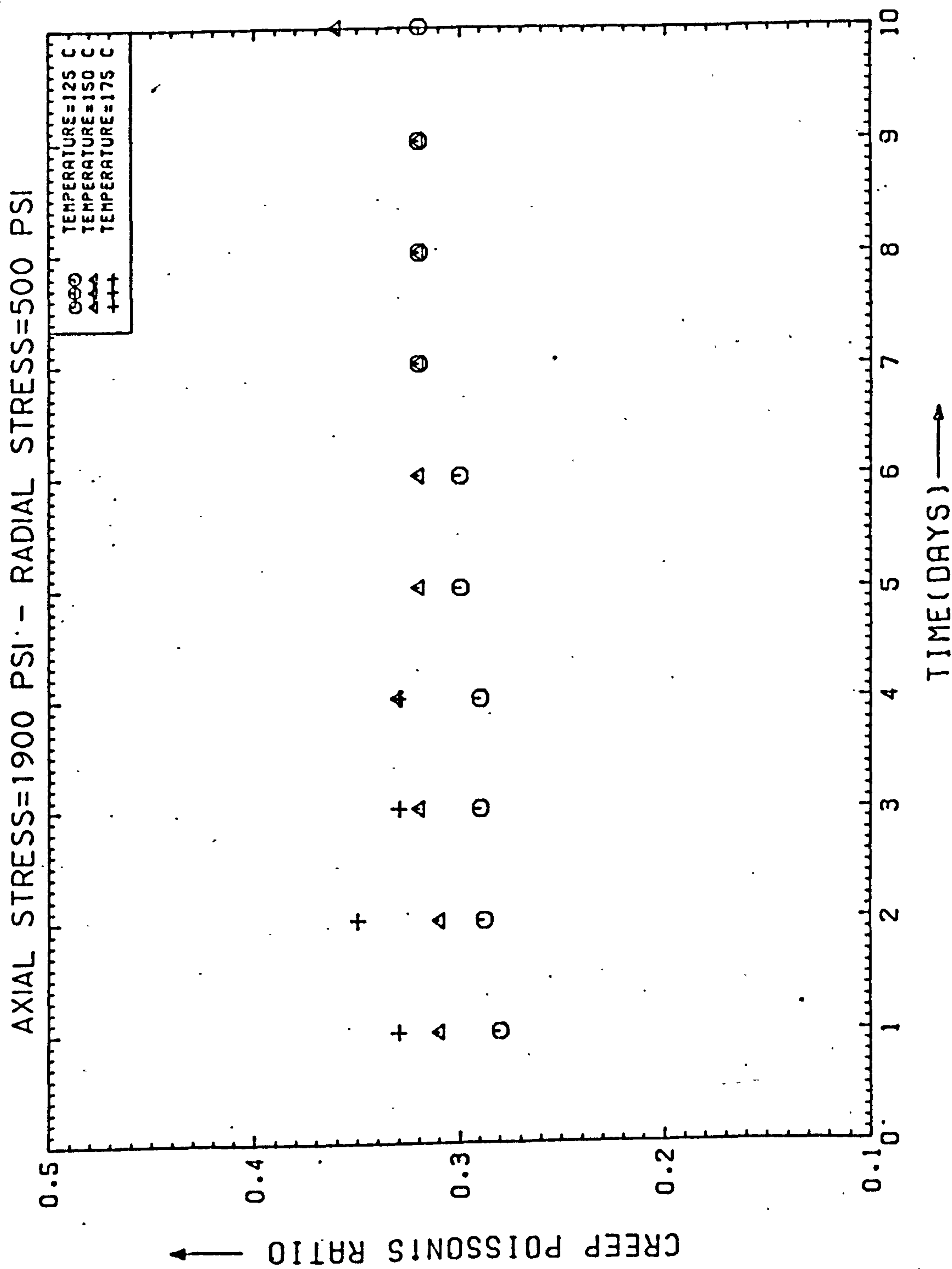


FIG. 7.4 CREEP POISSON'S RATIO

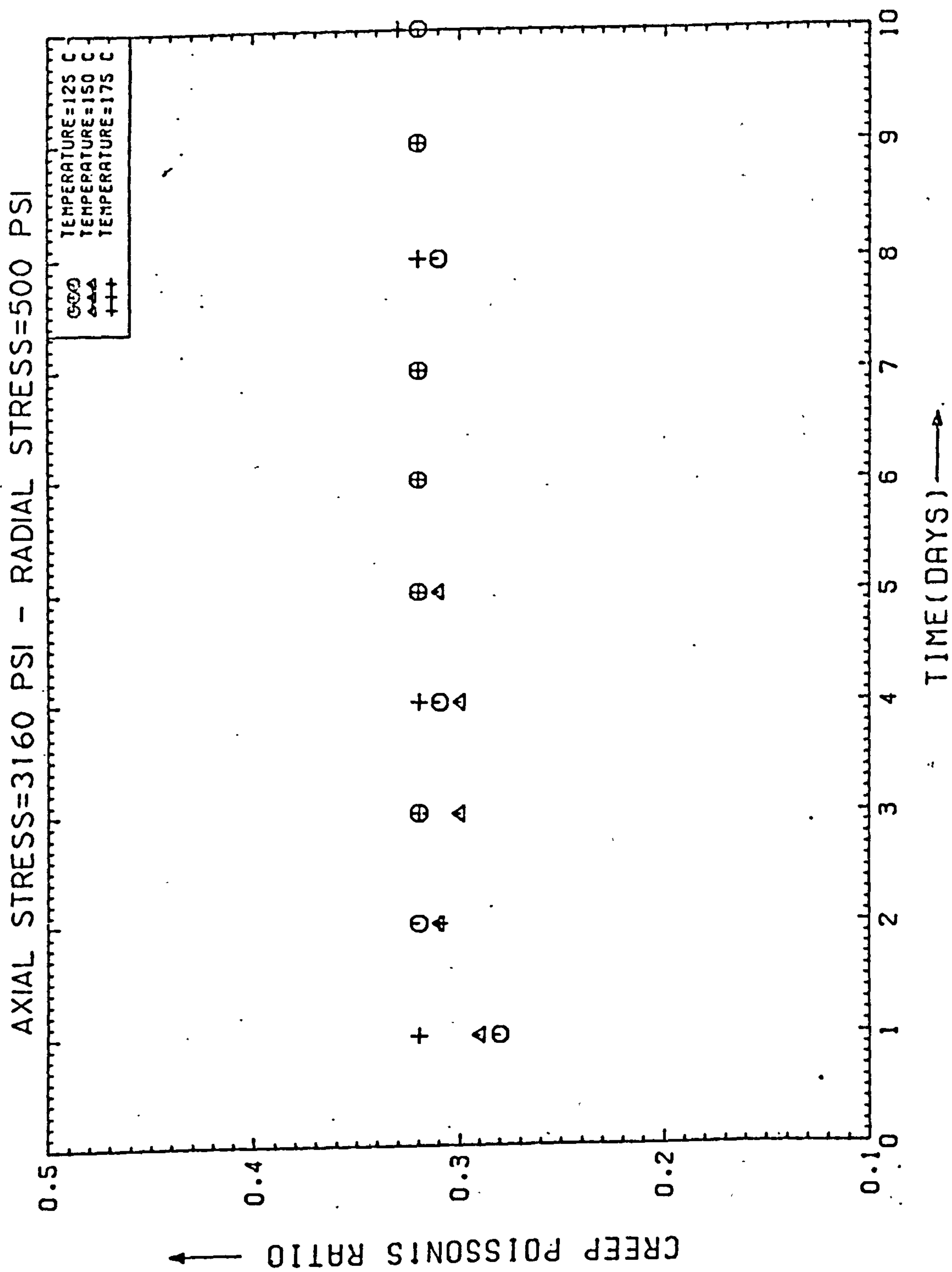


FIG.7.5 CREEP POISSON'S RATIO

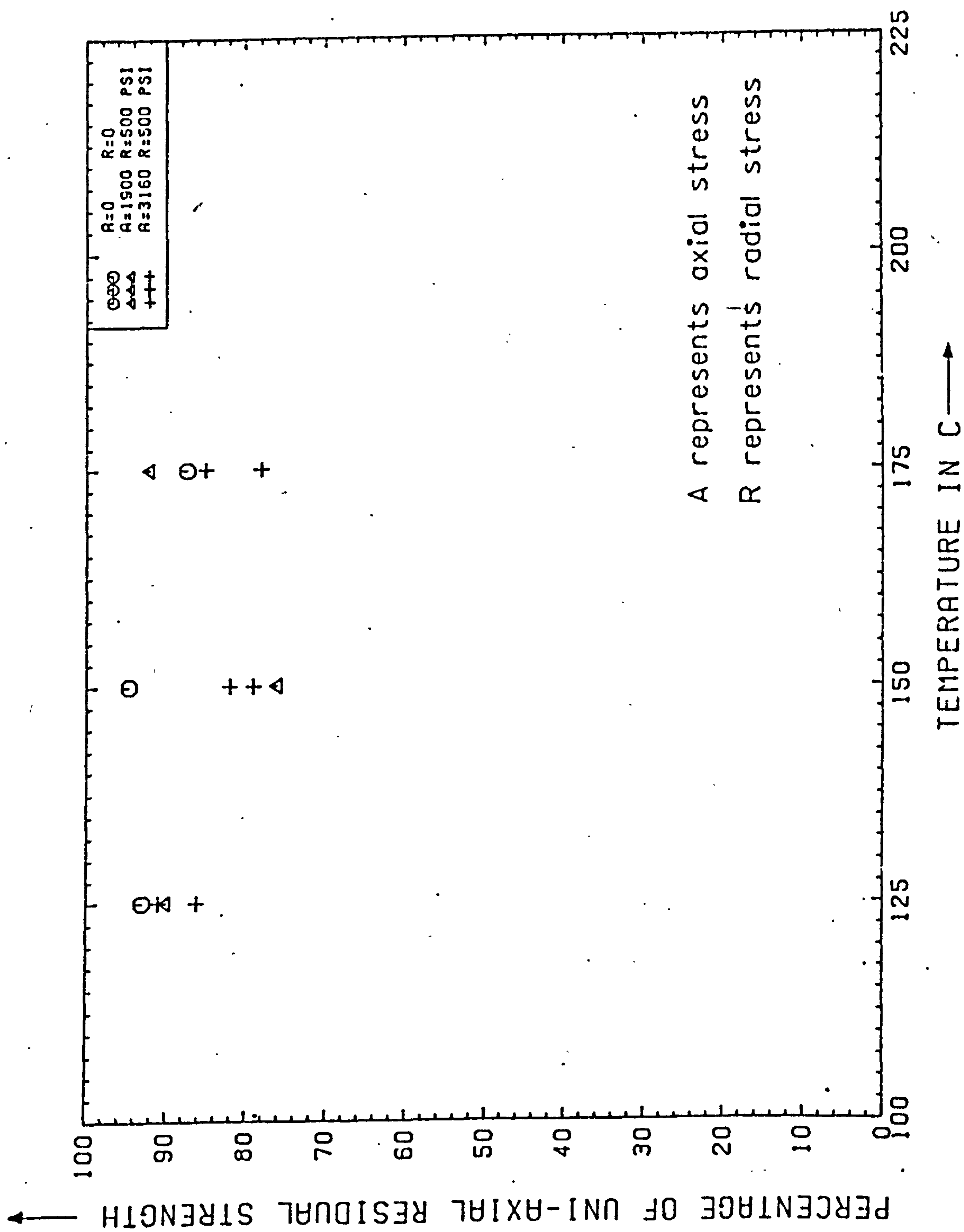


FIG. 7.6 UNIAXIAL RESIDUAL STRENGTH

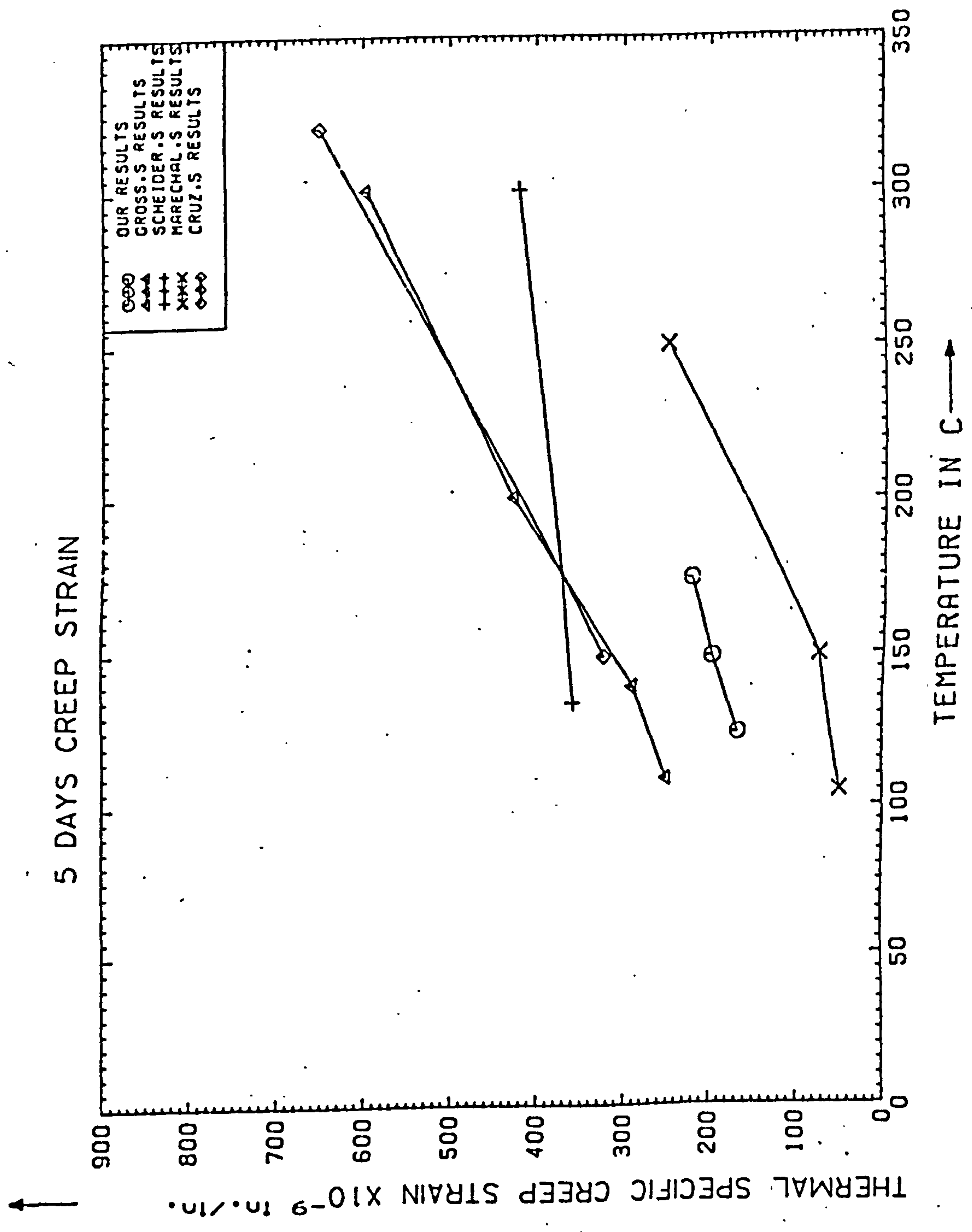


FIG. 7.7 THERMAL SPECIFIC CREEP STRAIN

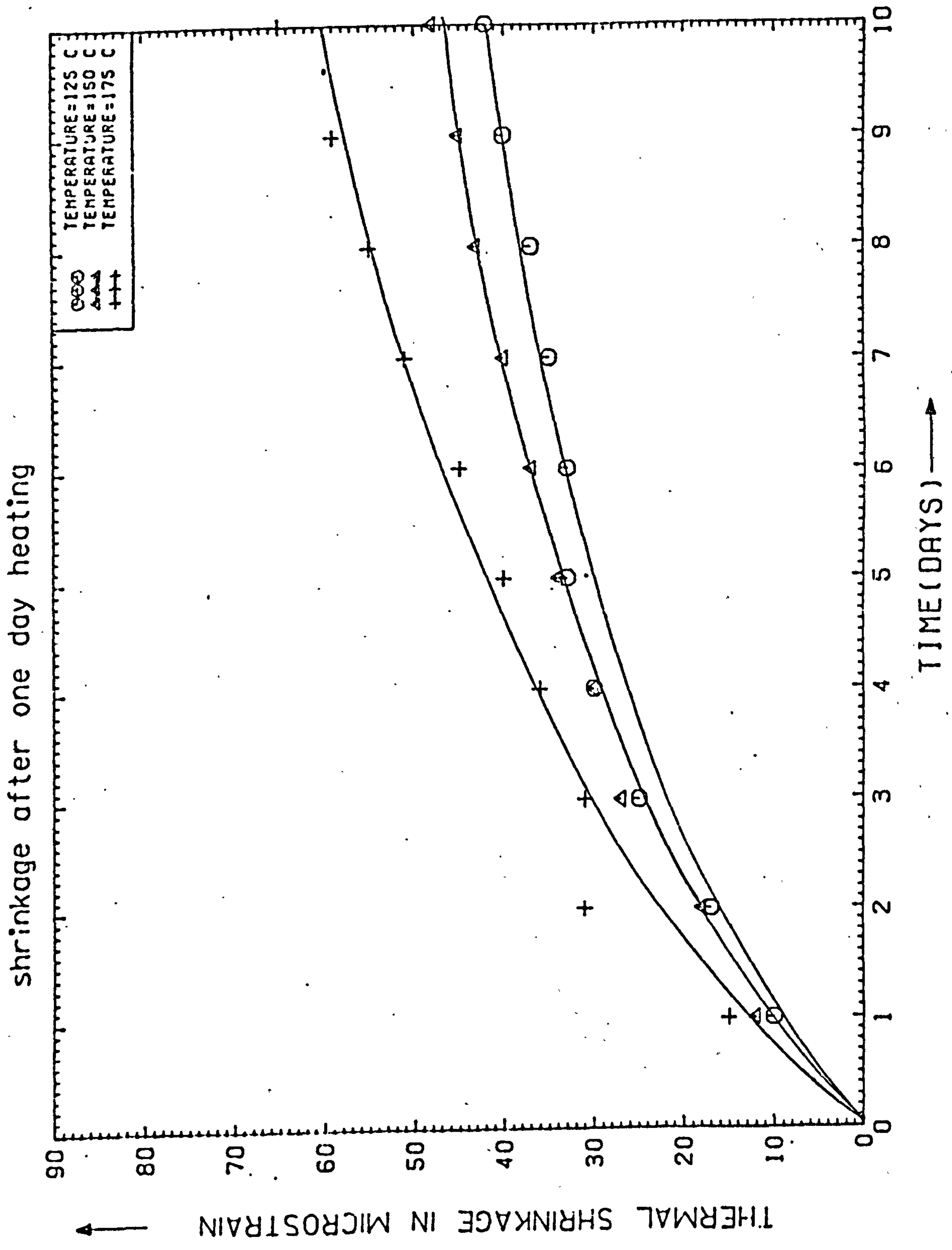


FIG.7.8 THERMAL SHRINKAGE FOR TEN DAYS PERIOD

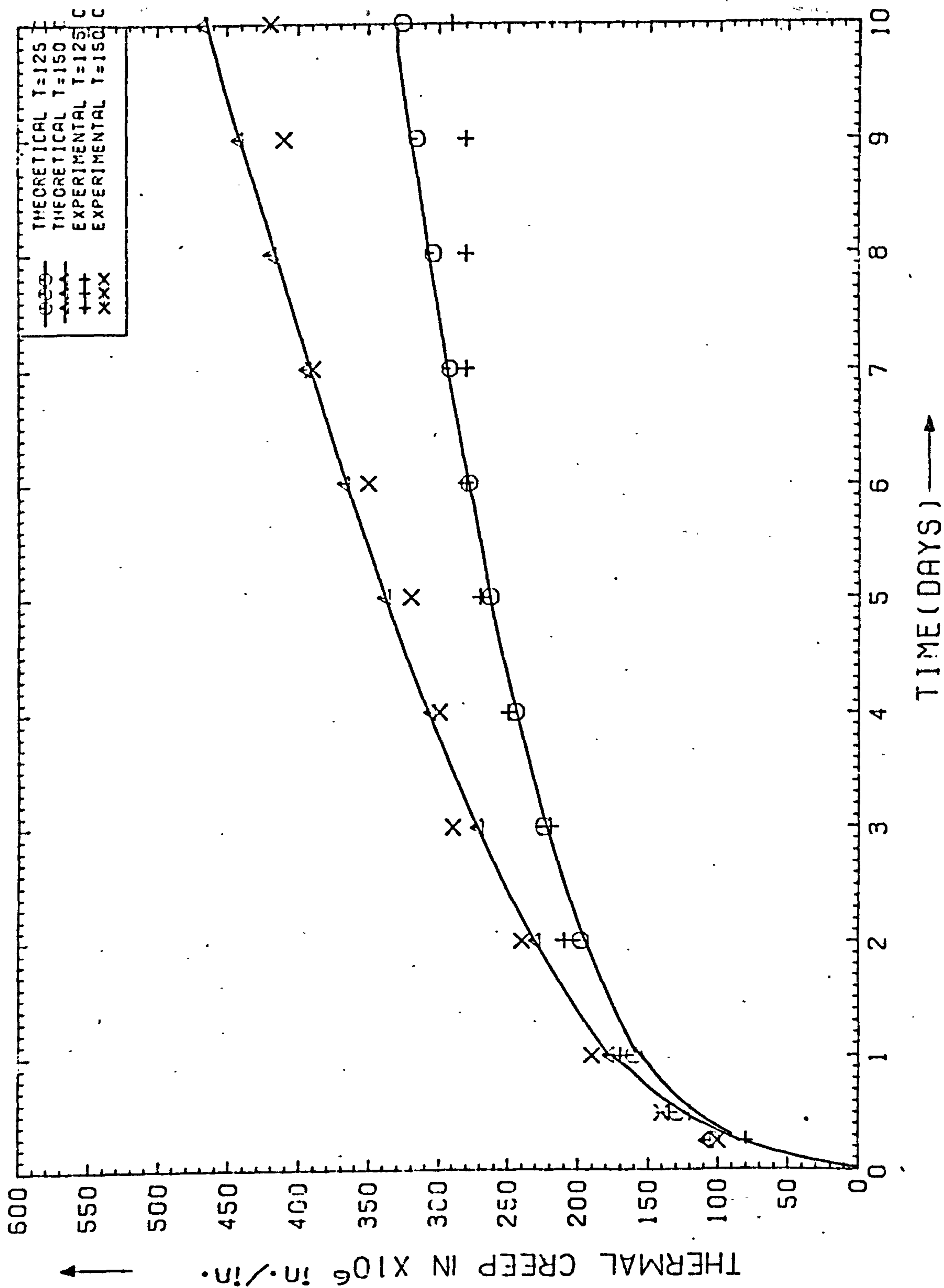


FIG 7.9 ANALYTICAL & EXPERIMENTAL RESULTS COMPARISON



FIG 7.10 SAMPLES OF TESTED SPECIMENS

Chapter 8

FINITE ELEMENT ANALYSIS OF THERMAL CREEP

8.1 Introduction

Reactor components are exposed to severe environmental conditions under which they are required to perform a complex pattern of operating schedules. The mechanical behaviour is strongly affected by these conditions, for example, most material properties exhibit at elevated temperatures pronounced non-linearity and strain rate sensitivity.

The question arises then how creep data, obtained from constant stress and temperature experiments, could be used for the case of time-varying stresses and temperatures. Different cumulative creep theories have been proposed; however, only the well-known strain-hardening theory is considered here.

In general, the mechanical and thermal effects are ^{not} coupled. Therefore, it is naturally possible to take account of this uncoupling. Thus, the problem of determining the response of non elastic structure to mechanical and thermal loads may be divided into two distinct problems.

- 1) Transient heat conduction;
- 2) Stress distribution due to thermal, mechanical loading and creep.

Once the first problem is solved, the second one may be

treated by methods discussed in section 8.3.

8.2 Transient Heat Conduction

The analysis of heat conduction problems is based on the concept of conservation of energy and an appropriate heat conduction law. For the complex problems arising in real life, solution of heat diffusion equation together with its boundary and initial conditions usually has to be carried out numerically. As in structural analysis, we may proceed either from the governing differential equations or from integral relations. The differential approach leads directly to finite difference methods, where for transient problems finite differences are also introduced in the time domain. The integral relation is set up as a variational principle. From the variational point of view finite element procedures for the analysis of transient or steady-state heat conduction are derived in much the same manner as in stress analysis.

The general 'quasi-harmonic' equation governing the heat diffusion can be written as (ref. 129)

$$\frac{d}{dx} \left(k_x \frac{d\theta}{dx} \right) + \frac{d}{dy} \left(k_y \frac{d\theta}{dy} \right) + \frac{d}{dz} \left(k_z \frac{d\theta}{dz} \right) + Q_e - C \frac{d\theta}{dt} = 0. \quad (8.2.1)$$

where

θ is the temperature

k_x, k_y, k_z are the material thermal conductivity coefficients.

Q_e is the rate of heat generation.

C is the specific heat per unit volume.

In the above, quite generally, all the parameters may be prescribed function of time and temperature.

$$k_x = k_x(t, \theta). \quad Q_e = Q_e(t, \theta), \text{ etc...}$$

Equation 8.1 together with the boundary conditions specifies the problem in a unique manner. However, an alternative is possible with the aid of the calculus of variations. The well-known Euler theorem (130) states that if the integral

$$X(u) = \int \int \int_V f(x, y, z, \theta, \frac{d\theta}{dx}, \frac{d\theta}{dy}, \frac{d\theta}{dz}) dx dy dz \quad (8.2.2)$$

is to be minimized over a bounded region V , then the necessary and sufficient condition for this minimum to be reached is that the unknown function $\theta(x, y, z)$ should satisfy the following differential equation

$$\frac{d}{dx} \left\{ \frac{df}{d(d\theta/dx)} \right\} + \frac{d}{dy} \left\{ \frac{df}{d(d\theta/dy)} \right\} + \frac{d}{dz} \left\{ \frac{df}{d(d\theta/dz)} \right\} - \frac{df}{d\theta} = 0 \quad (8.2.3)$$

within the same region, provided θ satisfies the same boundary conditions. This leads to the volume integral given below and taken over the whole region, should be minimized

$$X = \int \int \int \left[\frac{1}{2} \left\{ k_x \left(\frac{d\theta}{dx} \right)^2 + k_y \left(\frac{d\theta}{dy} \right)^2 + k_z \left(\frac{d\theta}{dz} \right)^2 \right\} - Q\theta \right] dx dy dz \quad (8.2.4)$$

$$\text{where } Q = Q_e - C \frac{d\theta}{dt}$$

subject to θ obeying the same boundary conditions.

If the unknown θ is defined, element by element, as

$$\theta = [N_1, N_j, \dots] \begin{Bmatrix} \theta_1 \\ \theta_j \\ \vdots \end{Bmatrix} = [N] \{\theta\}^e \quad (8.2.5)$$

where θ_1 , etc., are the nodal temperature, approximate minimization can be carried out (131).

Proceeding generally, we evaluate the element contribution

using the definitions of Eqs. (8.2.4) and (8.2.5).

For any node we can write, by differentiating Eq. (8.2.4)

$$\frac{dX^e}{d\theta_i} = \int_{ve} \left\{ k_x \frac{d\theta}{dx} \frac{d}{d\theta_i} \left(\frac{d\theta}{dx} \right) + k_y \frac{d\theta}{dy} \frac{d}{d\theta_i} \left(\frac{d\theta}{dy} \right) + k_z \frac{d\theta}{dz} \frac{d}{d\theta_i} \left(\frac{d\theta}{dz} \right) - q \frac{d\theta}{d\theta_i} \right\} dx dy dz \quad (8.2.6)$$

Noting that

$$\frac{d\theta}{dx} = \frac{dN_1}{dx}, \quad \frac{d\theta}{dx} = \frac{dN_j}{dx}, \quad \dots \quad \{\theta\}^e, \text{ etc.}$$

and that

$$\frac{d}{d\theta_i} \frac{d\theta}{dx} = \frac{dN_1}{dx}$$

$$\frac{d\theta}{d\theta_i} = N_1, \text{ etc.}$$

we have immediately for the whole element

$$\frac{dX^e}{d\{\theta\}^e} = [k]^e \{\theta\}^e + \{F\}^e \quad (8.2.7)$$

in which the 'stiffness matrix', k is obtained from Eqs.

(8.6) and (8.7)

$$k_{ij}^e = \int_{ve} \left\{ k_x \frac{dN_i}{dx} \frac{dN_j}{dx} + k_y \frac{dN_i}{dy} \frac{dN_j}{dy} + k_z \frac{dN_i}{dz} \frac{dN_j}{dz} \right\} dx dy dz \quad (8.2.8)$$

and with $dV = dx dy dz$

$$F_1^e = - \int_{ve} q N_1 dV + \int_{se} q N_1 dS \quad (8.2.9)$$

Assembly of the whole set of minimizing equations follows the usual rules. Thus for the whole region we have

$$\frac{dX}{d\{\theta\}} = 0 = [K]\{\theta\} + \{F\} \quad (8.2.10)$$

with

$$K_{ij} = \sum k_{ij}^e \quad F_i = \sum F_i^e$$

and summation over all elements.

For axisymmetric case Eq. 8.2.8 reduces to

$$K_{ij}^e = \int_{V^e} \left\{ \left(\frac{1}{r} \cdot \frac{dN_i}{dr} \cdot (k_r \cdot r \cdot \frac{dN_j}{dr}) + \frac{dN_i}{dz} k_z \cdot \frac{dN_j}{dz} \right) \right\} dV \quad (8.2.11)$$

The properties of concrete relevant to determination of heat dissipation are;

1. the thermal conductivity for the determination of steady-state conditions.

2. the thermal diffusivity for the determination of the transient conditions.

It is generally believed that conductivity of concrete depends on the conductivity of the aggregate, the mix proportions, the type of cement, the temperature of the concrete and the moisture content of the concrete. The ratio of thermal diffusivity to conductivity may be expressed as follows;

$$\frac{h}{k} = \frac{1}{C\rho} \quad (8.2.12)$$

where h is the thermal diffusivity

k is the thermal conductivity

ρ is the unit weight of concrete

C is the specific heat.

The main argument in favour of finite elements versus finite differences in heat conduction analysis concerns the combination of heat conduction and thermal stress analysis. The possibility of basing both analyses on the same element idealization, constitutes such a tremendous practical advantage that any slightly longer computation times for the actual thermal analysis are by far outweighed by the effort saved in data preparation and handling.

8.3 Thermal Creep of Concrete

Various analytical methods such as effective modulus, rate of creep, superposition etc. have been used to calculate creep strains in prestressed concrete members for PCRV. It is made clear, however, that these methods could not always give a realistic prediction for creep behaviours of concrete members especially in such stage as initial heat up of PCRV, when concrete undergoes fairly wide variation of temperature (131, 132)

The author proposed a new method of creep analysis based on the theory of strain hardening which assumes that the creep rate depends only on the current stress, the current temperature and the accumulated creep strain. The creep rate at any instant of time is defined by the following equation of state

$$\frac{d\varepsilon_c}{dt} = f(\sigma, T, \varepsilon_c) \quad (8.3.1)$$

$\frac{d\varepsilon_c}{dt}$ is the creep rate

σ is the effective stress

ε_c is the thermal creep

Equation 7.1.1 may be decomposed as below

$$\varepsilon_c = \frac{1}{10} \sigma t^{0.31} f(T) \quad (7.1.1)$$

where

$f(T) = \left[1 + \left(\frac{125 - T}{1000} t \right)^{0.6} \right]$, and assumed to be a function of temperature only.

The creep rate at any instant of time is then arrived approximately by the following expression

$$\frac{d\varepsilon_c}{dt} = 0.031 \sigma t^{-0.61} f(T) \quad (8.3.2)$$

Elimination of time t from equation (7.1.1) and (8.3.2) lead to the strain hardening law as follows

$$\frac{d\varepsilon_c}{dt} = 0.31 [\sigma f(T)]^{3.225} \varepsilon_c^{-2.225} \quad (8.3.3)$$

Equation (8.3.2) is the time-hardening law which states that the creep rate depends only on the current stress, the current temperature and the time from the beginning of the creep process. Fig. (8.1) illustrates the difference of both hardening rules for a varying stress and temperature. To implement Eq. (8.3.3) in large creep finite element program, it should be in incremental form in which the total creep at n th step will be

$$\varepsilon_{c(n)} = \varepsilon_{c(i)} + \sum_{i+1}^n 0.31 [\sigma_i f_i(T)]^{3.225} \varepsilon_{c(i)}^{-2.225} \cdot \Delta t_i \quad (8.3.4)$$

In this method the stress value determined by the end of each step will remain constant in the next step duration. Therefore, the time increment should be exceptionally small when stress changes are relatively large. From qualitative point of view this procedure may be considered to reflect realistic characteristics of creep in concrete at high temperatures.

8.4 Solution Procedure

At time t the applied load R_0 , the displacement distribution r_0 , and the stress distribution σ_0 are known from linear elastic analysis. After then the following steps are performed:

- a- Thermal load increment is applied

$$R_\Delta = - B^T E \Delta \epsilon^T \quad (8.4.1)$$

E is the elastic material stiffness matrix.

$\Delta \epsilon^T$ is the thermal strain increment

- b- Displacement increment

$$r_\Delta = - K^{-1} R_\Delta \quad (8.4.2)$$

Naturally, this relation only represents a forward and back substitution on the right-hand side R_Δ , since the stiffness matrix is assumed to remain unchanged throughout the solution procedure.

- c- Element displacement increment

$$p_\Delta = - a r_\Delta \quad (8.4.3)$$

a is the connectivity matrix

- d- Fictitious stress increment corresponding to

total strain increment $\gamma_A = B \rho_A$

$$\sigma_{tA} = - E B \rho_A \quad (8.4.4)$$

f. Correction to be made on Eq. (8.4.4) due to creep

$$\sigma_A = \sigma_{At} - E \Delta \epsilon^c \quad (8.4.5)$$

where $\Delta \epsilon^c$ is the creep increment during time increment Δt .

g. Finally, the incremental stresses obtained from Eq. (8.4.5) are added to the previous stresses to yield the new stress distribution $\sigma = \sigma_o + \sigma_A$. As long as the increment stresses $\Delta \sigma$ are small compared to the previous stresses, σ_o , no basic assumption is violated and the solution proceeds to the next time increment using the same procedure. If, however, the incremental stress due to temperature is not small compared with the previous stresses, the time step can be repeated with a smaller time increment.

During the initial heat up, the stresses change very rapidly so that it is necessary to choose small time increment in this region. However, as the solution approaches a steady state condition, it is theoretically possible to increase the time increments without violating any assumption.

This method is implemented in program VESSEL which has been developed by the author from ASKA package (see Appendix 3).

8.5 Prestressed Concrete Reactor Pressure Vessel

This academic example is chosen to illustrate the application of strain hardening method and to examine the influence

of 'Hot-Spots' close to the penetrations region.

The geometry of the axisymmetric vessel is considered symmetric with respect to radial and axial axes. Therefore, it is sufficient to analyse the upper quarter of the vessel. Fig. (8.2) shows details of geometry and loading due to internal gas pressure, dead weight and prestressing.

The idealisation of the vessel is illustrated in figs. (8.3, 8.4) in which 191 triangular ring elements were used. For temperature distribution analysis TRIAX 3 is considered to be adequate, whereas for stress analysis TRIAX 6 is used.

The temperature analysis is carried out by program THERMO (see Appendix A3) which has been developed by the author from refs. (129, 133, 134). This program is particularly useful in determining temperature distribution in structure where steep temperature changes occur without laborious mesh refinement. The following boundary conditions are imposed. The inside surface of the vessel is subjected to a prescribed temperature range $60 - 175^{\circ}\text{C}$ while $25 - 40^{\circ}\text{C}$ temperature range on the outside. The surfaces along axis of symmetry are insulated against heat flow. The thermal conductivity, $k = 1.4 \text{ k cal / m.h.C}$, and the rate heat generation is determined from the reactor thermal output, $P = 10^6$ kilowatts.

The resulting steady state temperature distribution is illustrated in fig. (8.5) in prospective form to show the steep temperature gradient in the 'Hot-Spot' region, whereas fig. (8.6)

shows the same results in form of contour lines.

The results of elastic thermal stress analysis are shown in figs. (8.7), (8.10). It appears that the shear stresses at the 'Hot-Spot' region (node 136, fig. 8.4) reach 1750 psi. Figs. (8.11) - (8.14) show the stress distribution after 21% reduction in temperature.

In comparison with the previous results, it seems that there are general reduction in stresses but the shear at the same point (node 136) is 1400 psi and still relatively high may be due to steep temperature gradient. Shear stresses are entirely resisted by prestressed concrete, unless shear reinforcement is included. Normally, this is adequate, but if a 'Hot-Spot' unfortunately occurs in an area of shear due to internal gas-pressure while cracks relieve temperature stress, stresses due to internal pressure may be relieved. It is important, therefore, to be able to base calculations of shear stresses in such areas on well-established structural behaviour of representative samples of the particular grade of concrete used. It may be found that temperature stresses at 'Hot-Spots' are not particularly significant. On the other hand, values of creep under triaxial stress are smaller than values for uniaxial stress. The relaxation of stresses due to creep and the margin of safety in relation to brittle shear failure may, therefore, be less than is realised, and some improvement in design may be desirable.

8.6 Recommendation for Future Research

- 1) Using the testing apparatus proposed, test series should

be carried out on thermal creep of concrete under transient temperature regimes to study the influence that the preheating period of one day had on the numerical values of the creep data obtained in this work.

2) Extending the temperature range up to 400°C under a standard rate of heating and definite moisture conditions, in order to render the results comparable.

3) Following the procedure described, further test on steady state thermal creep should be conducted, using different types of concrete mix, and under various ages.

4) Test series should be carried out to study the temperature and moisture dependence of the coefficient of thermal expansion, the specific heat, the thermal conductivity and the thermal diffusivity of concrete.

5) Test series should be conducted for thermal creep under state of compression-compression-tension.

6) Test series should be performed to investigate the effect of the size of test specimens on the development of thermal creep strains in concrete.

7) The results should be fitted into mathematical formulae.

8) It appears that the strain hardening approach described in 8.3 offers great advantage in numerical implementation for

transient temperature, relevant theories not based on the previous method may be difficult to implement.

9) Both programmes THERMO and VESSEL should be coupled together with plotting routines used by the author " SURFACE, COUNTR".

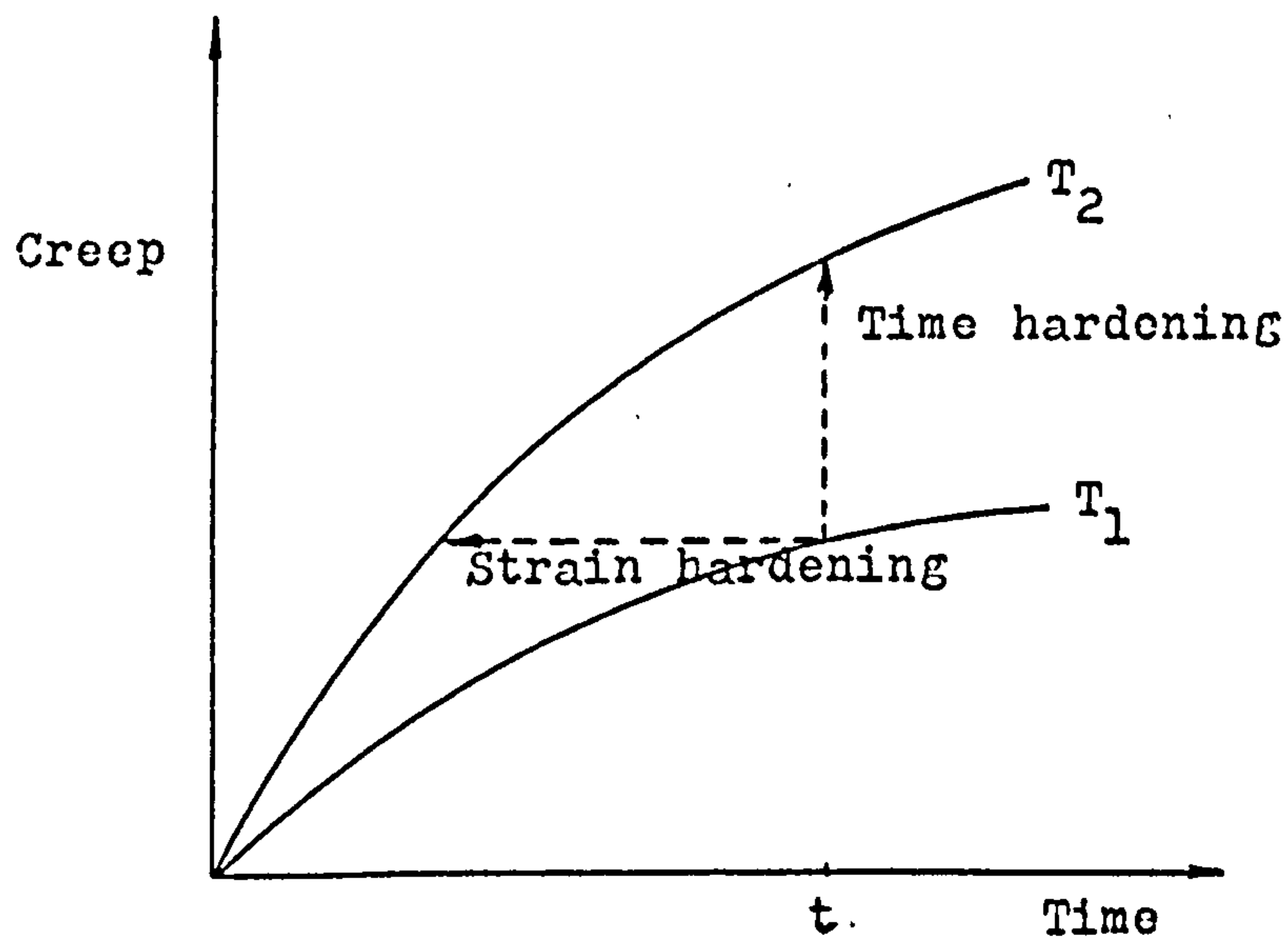


Figure 8.1 Variable temperature conditions

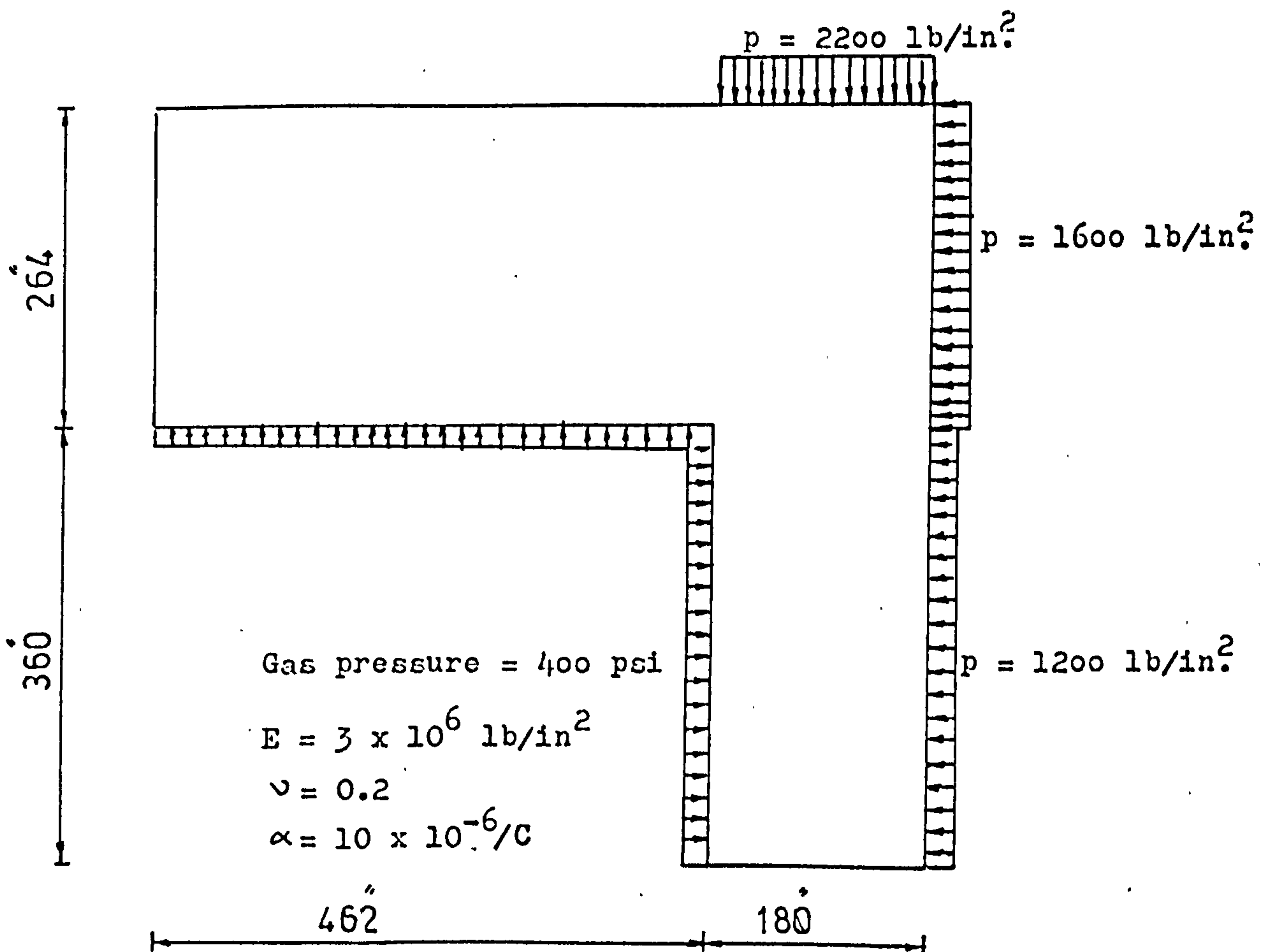


Figure 8.2 Prestressing and Gas pressure idealization

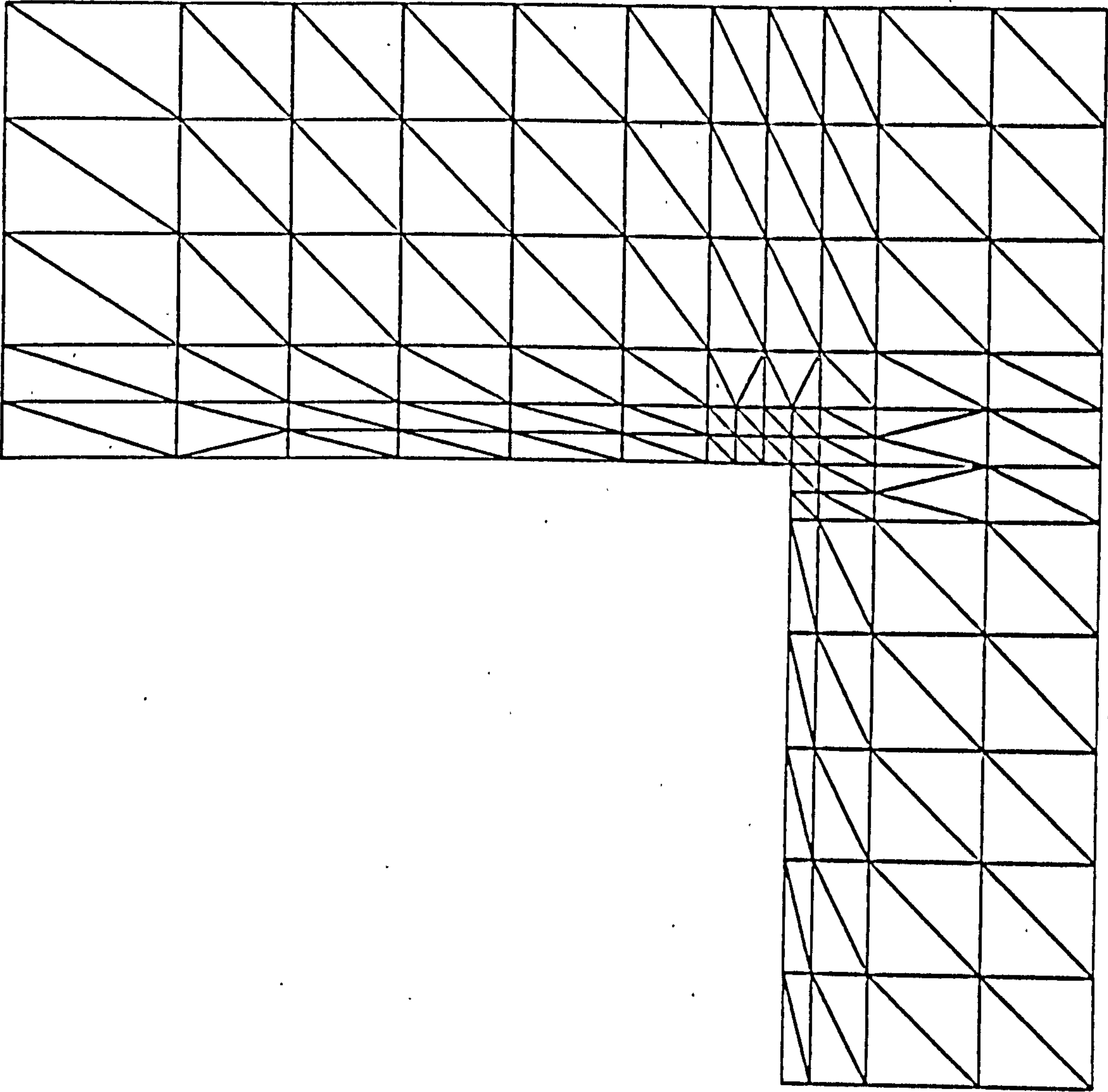


Figure 8.3 Finite Element Mesh for Axisymmetric Analysis

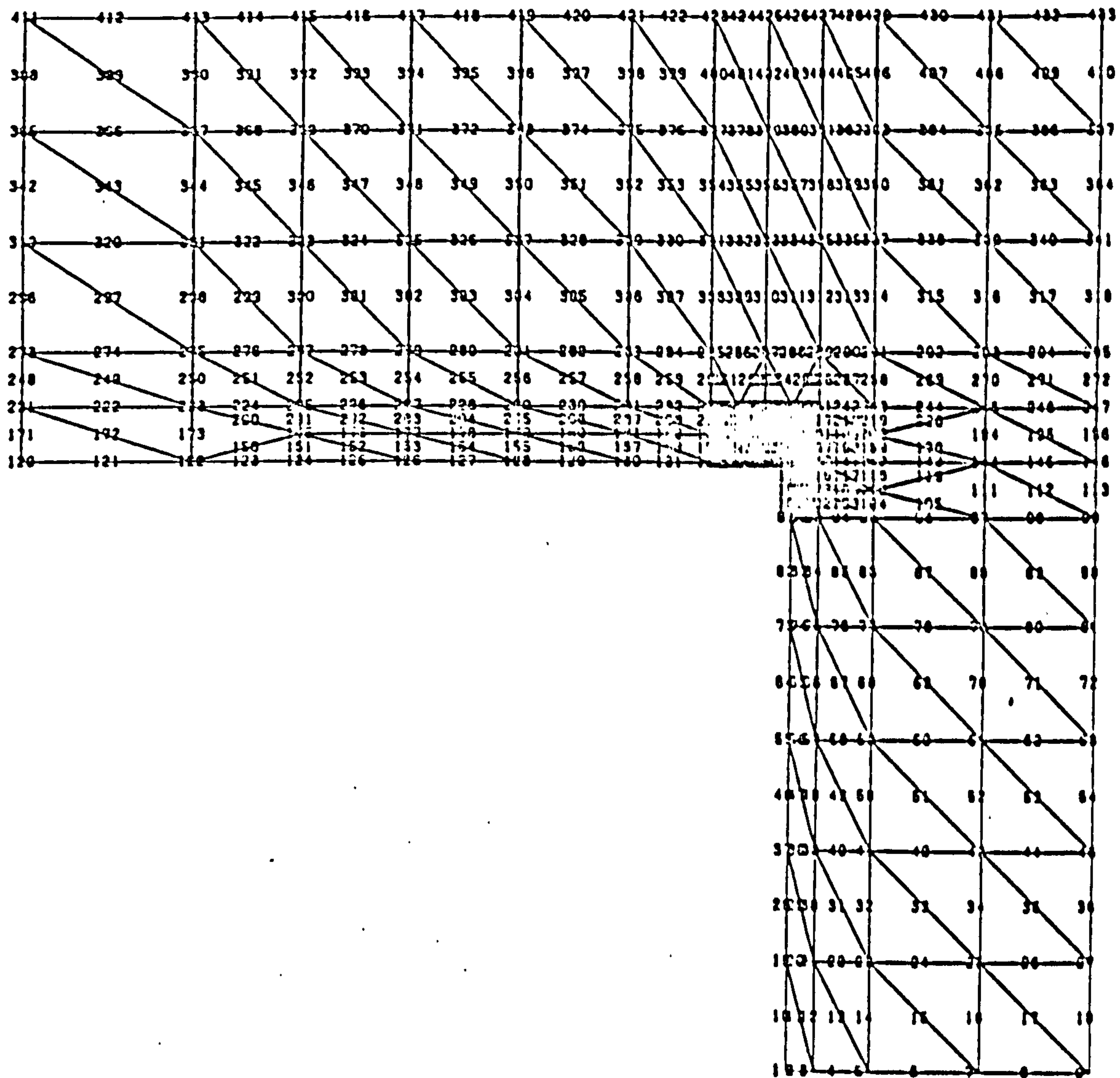


Figure 8.4 Topological Mesh for Axisymmetric Analysis
Using TRIAX 6 Elements

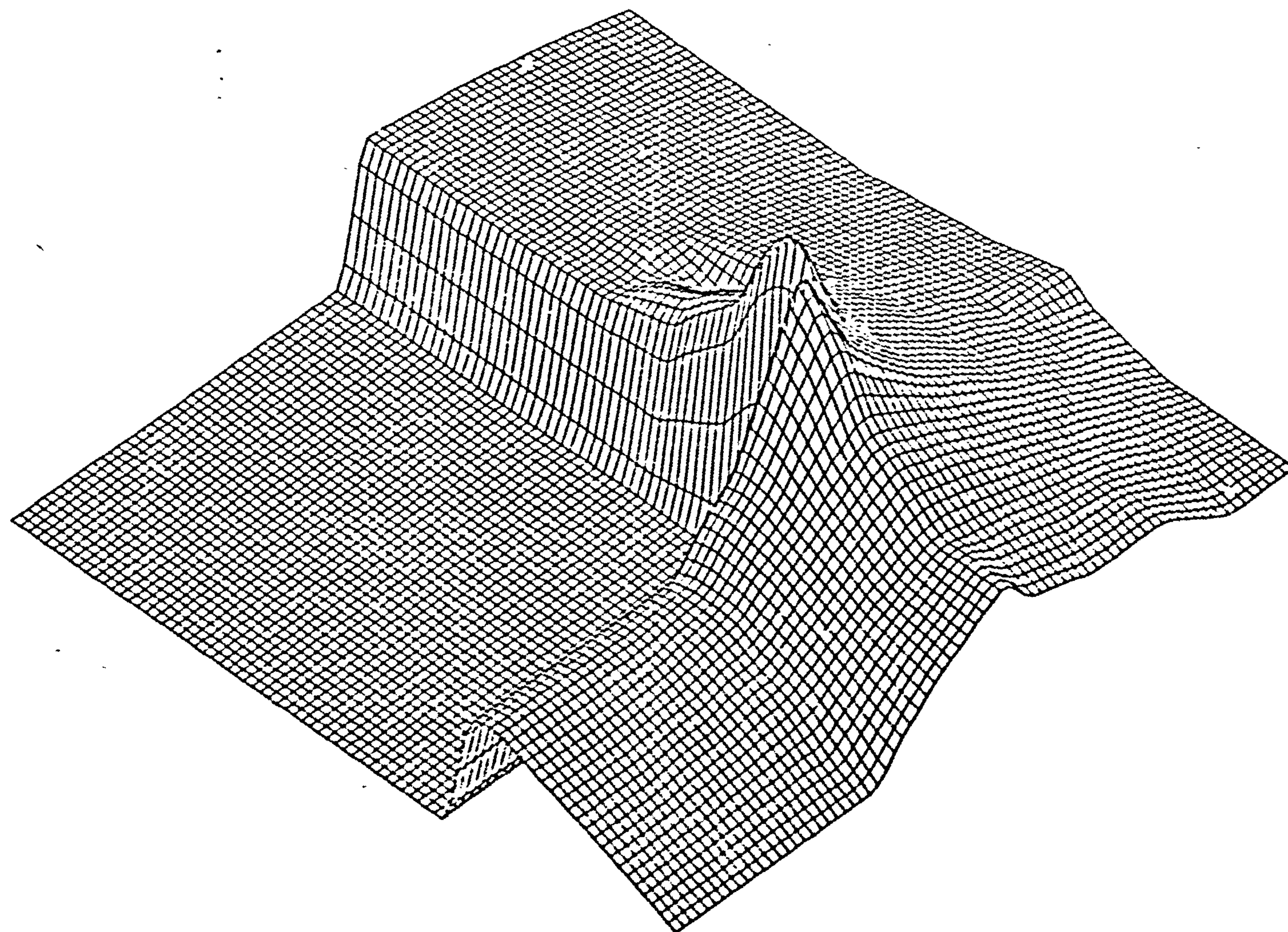


FIG. 8.5 TEMPERATURE DISTRIBUTION IN HOT SPOT OF PCPV

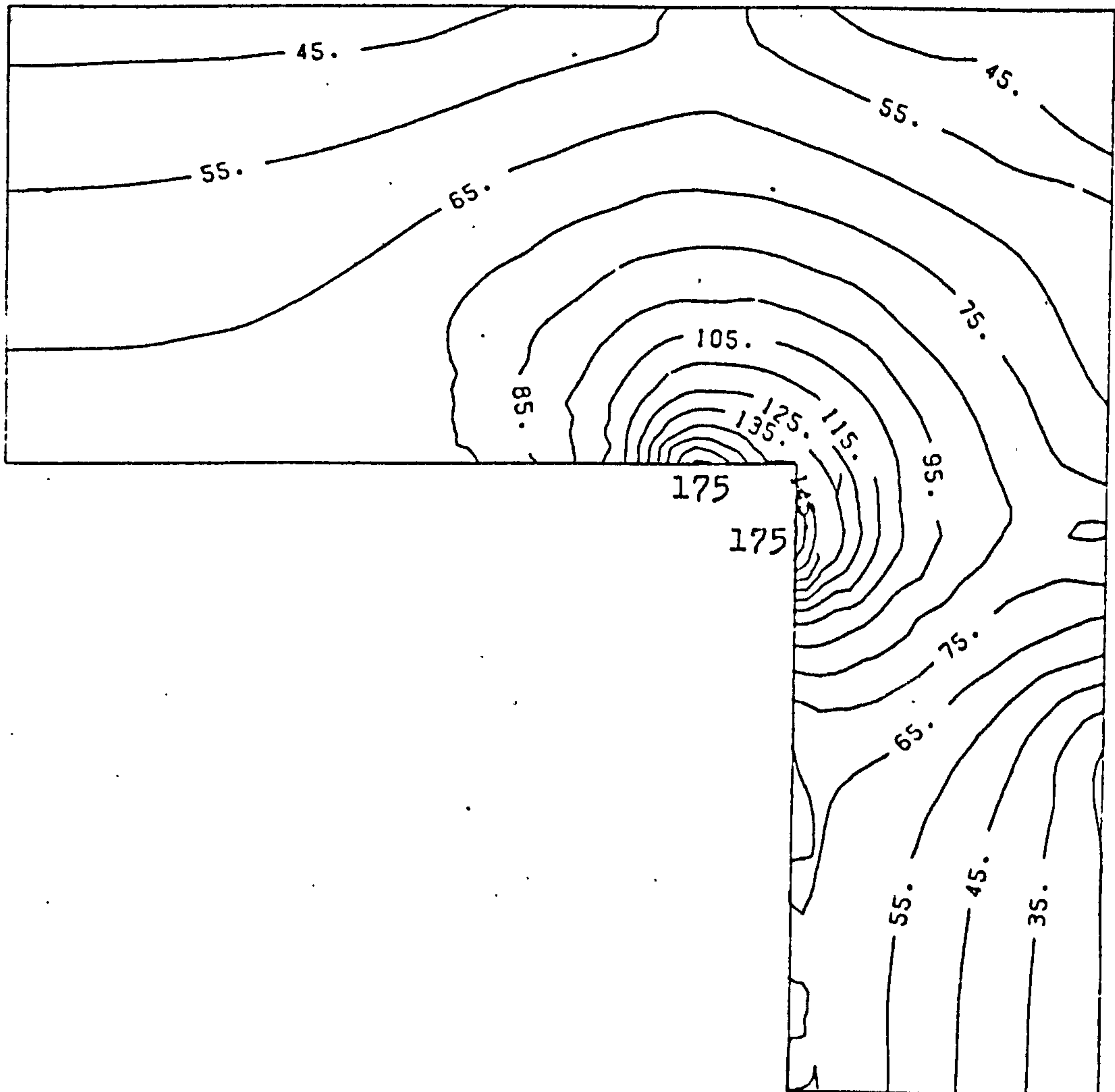


Figure 8.6 Temperature Distribution

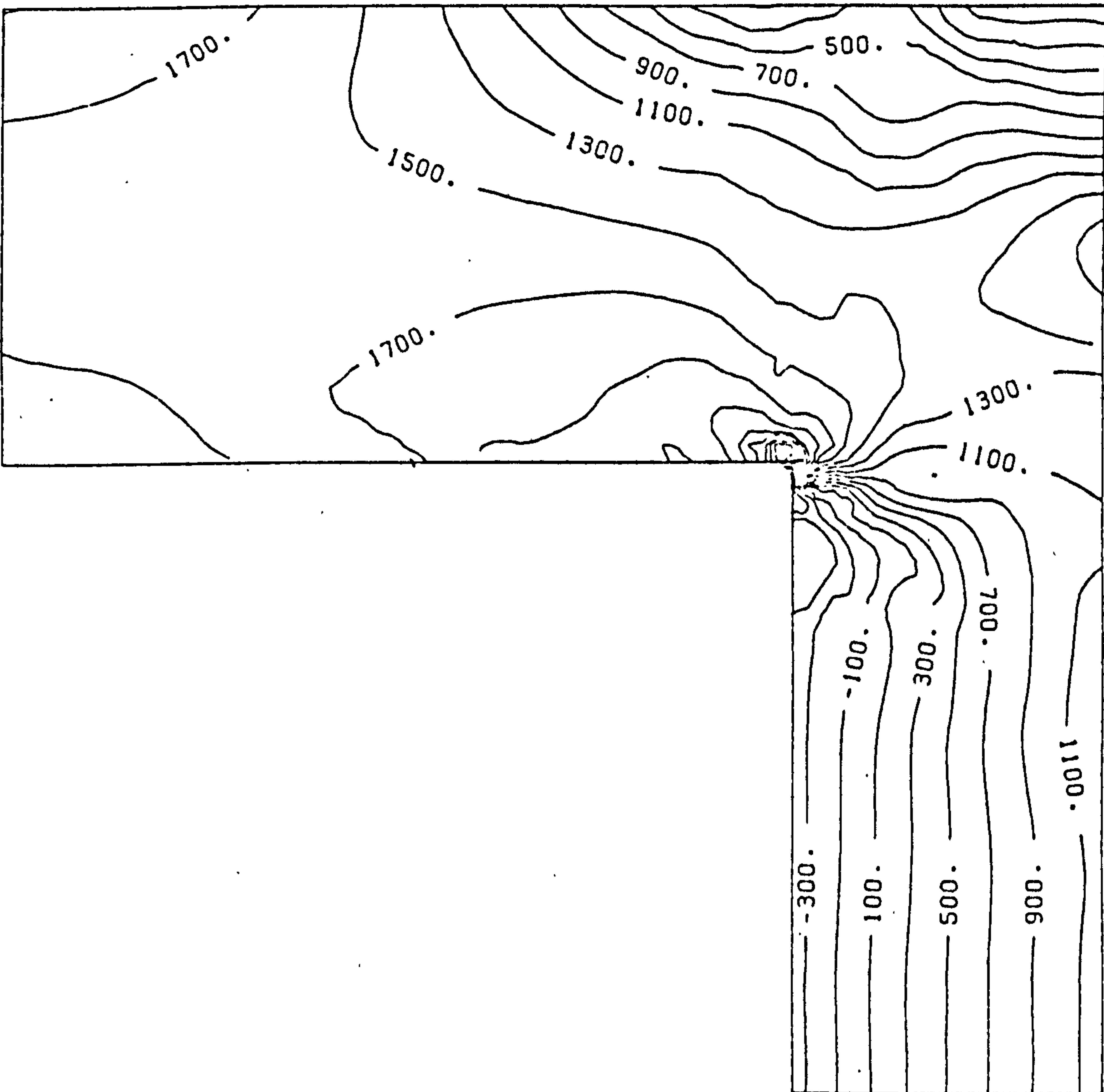


Figure 8.7 Contours of Radial Stresses at High Temperature

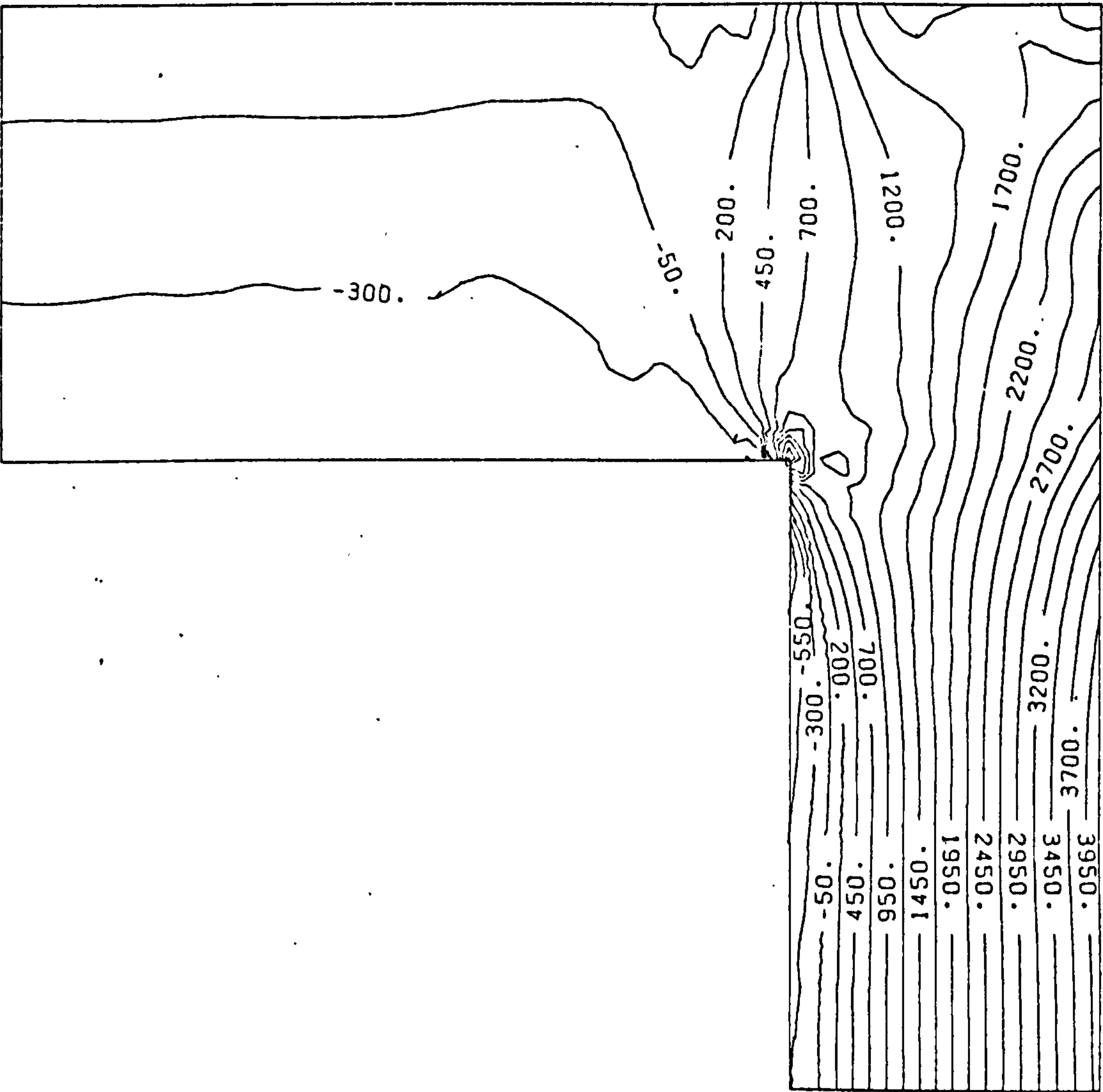


Figure 8.8 Contours of Vertical Stresses at High Temperature

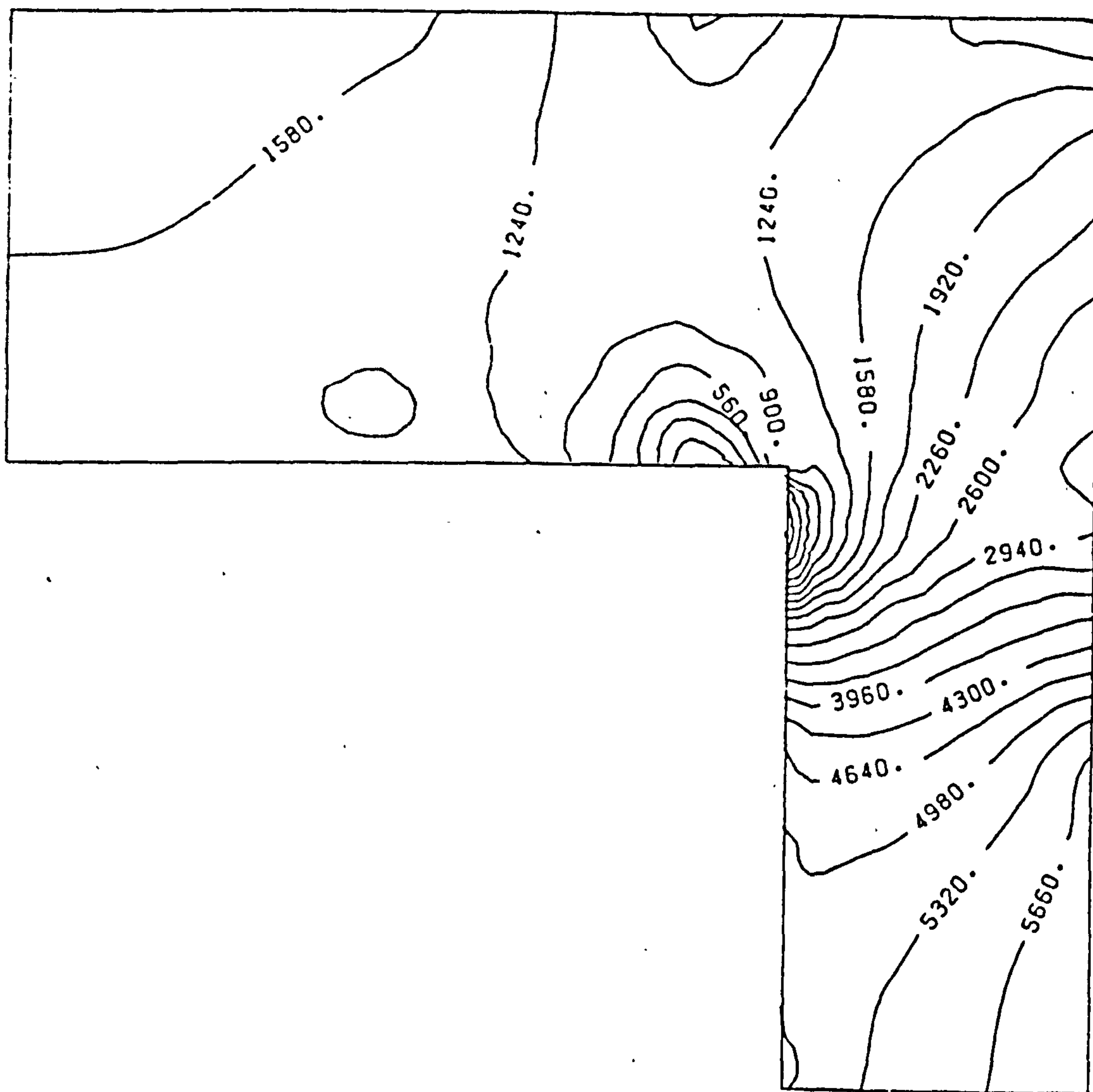


Figure 8.9 Contours of Tangential Stresses at High Temperature

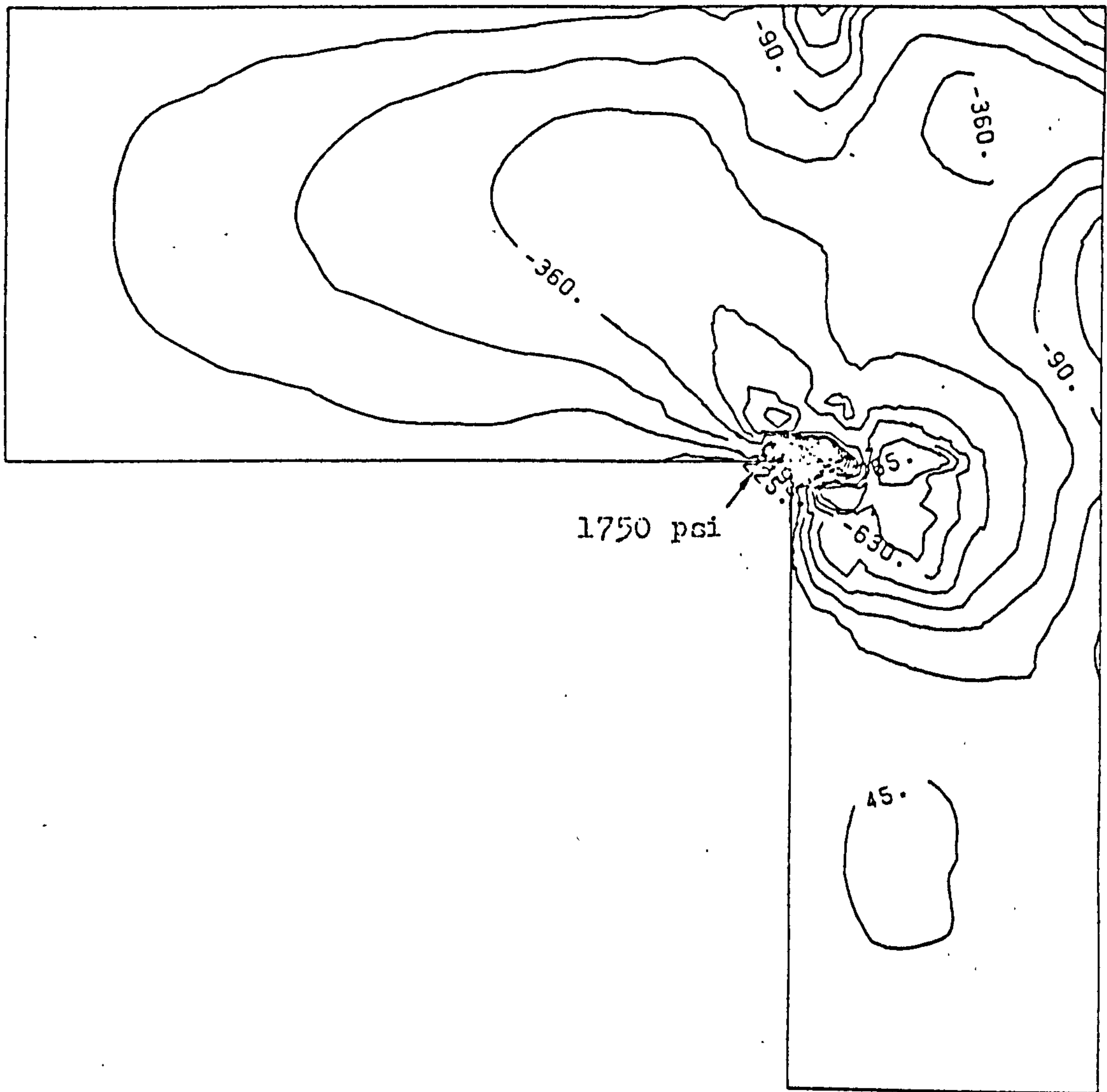


Figure 8.10 Contours of Shear Stresses at High Temperature

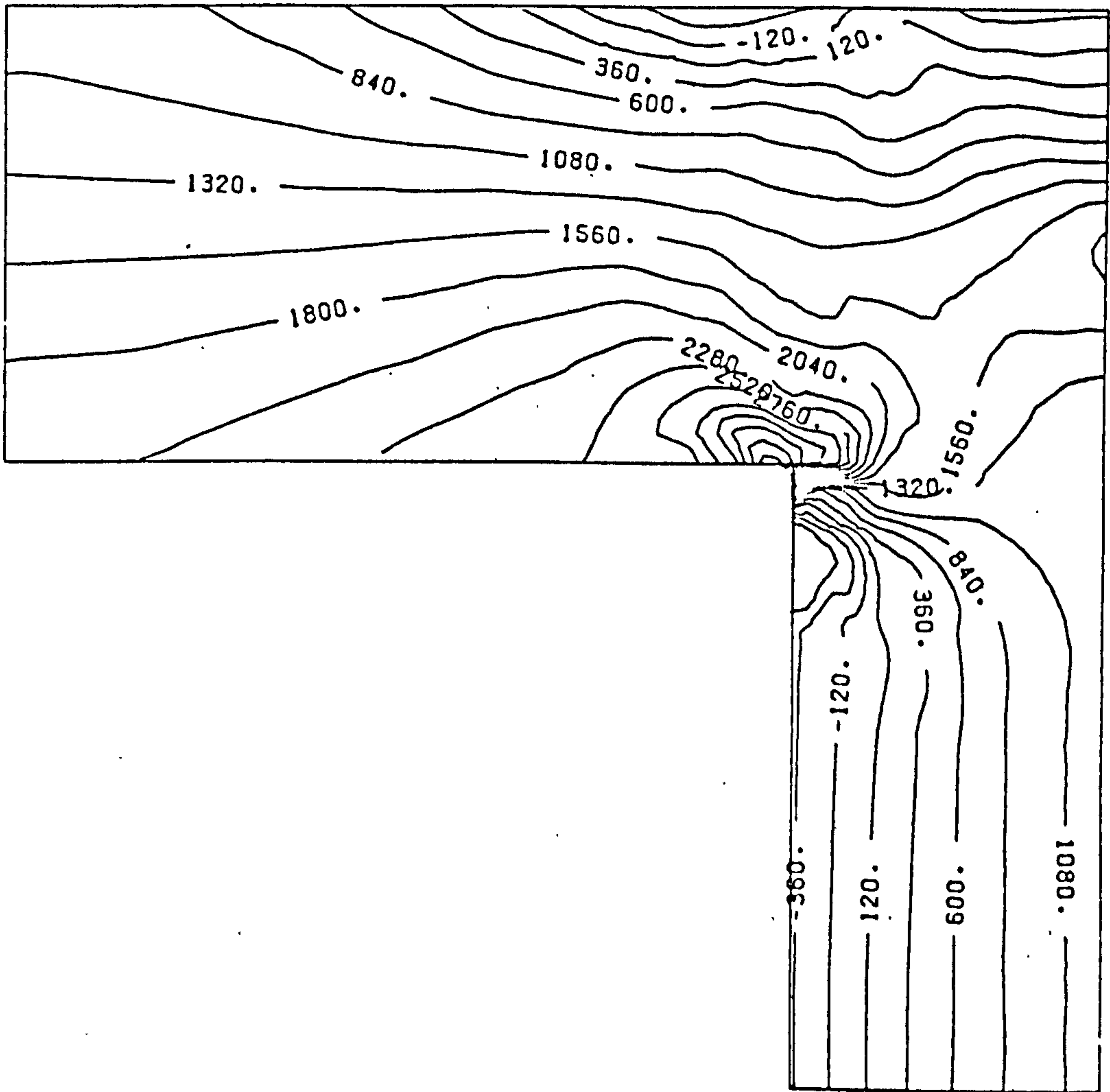


Figure 8.11 Contours of Radial Stresses at Low Temperature

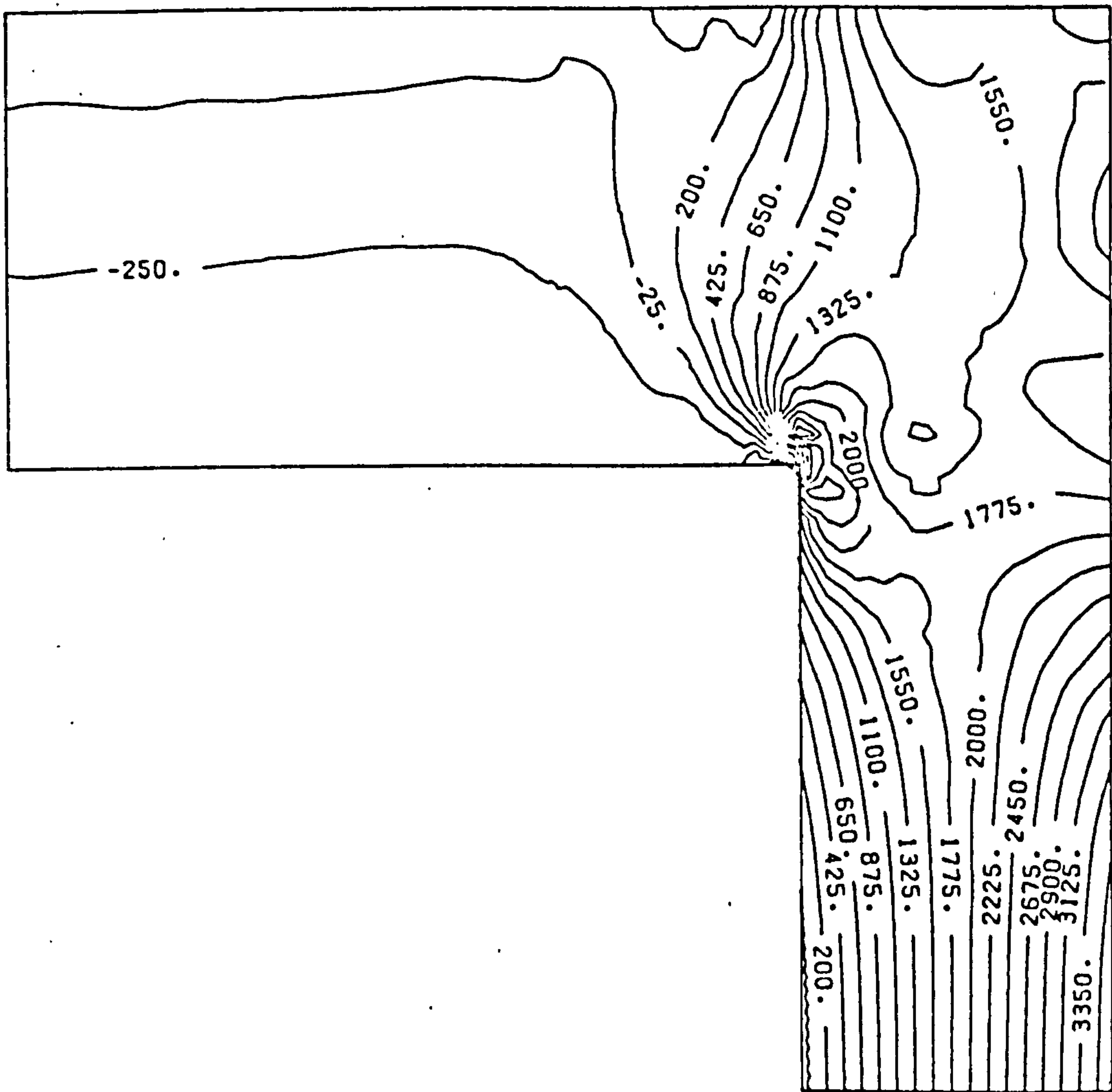


Figure 8.12 Contours of Vertical Stresses at Low Temperature

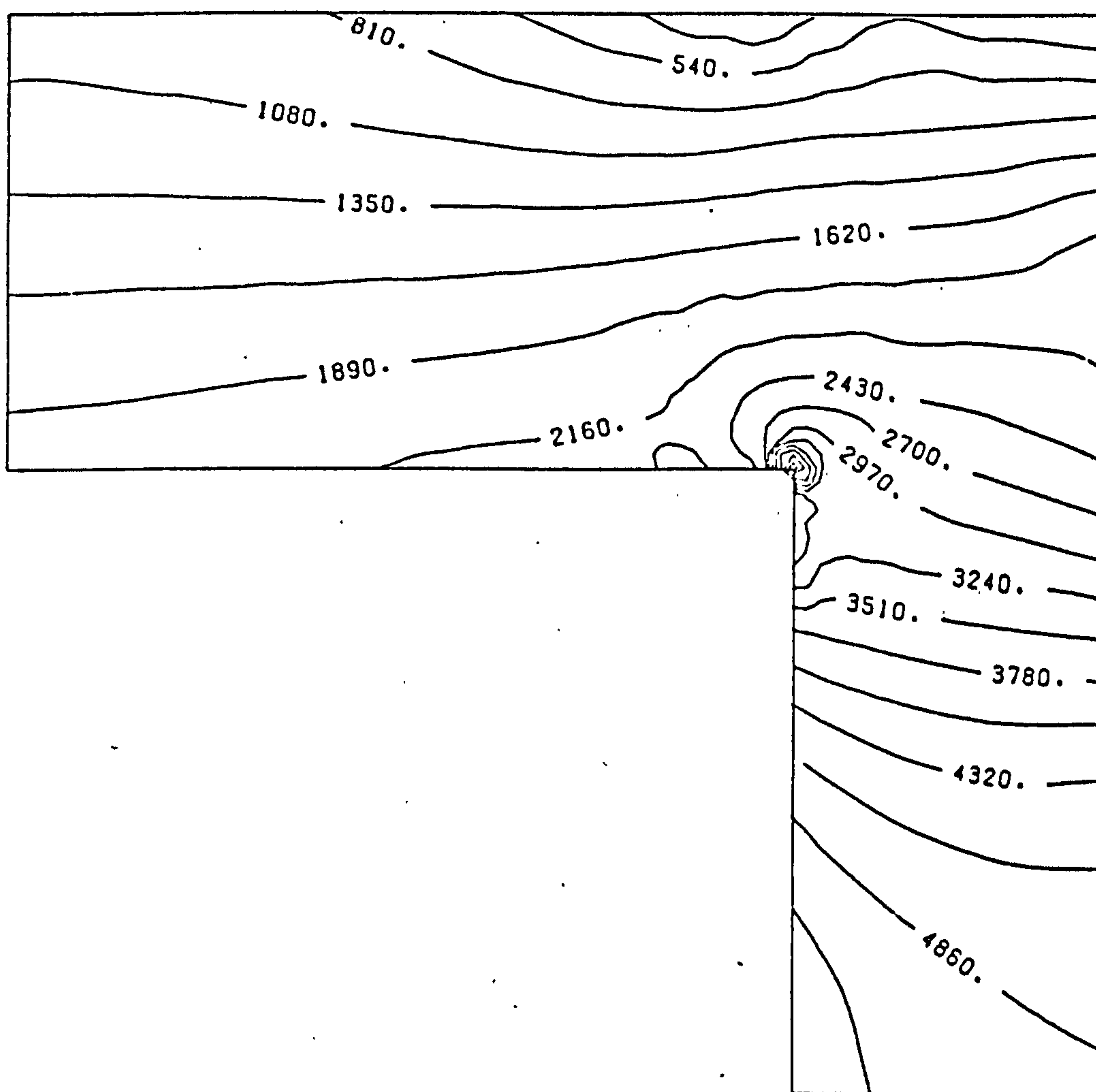


Figure 8.13 Contours of Tangential Stresses at Low Temperature

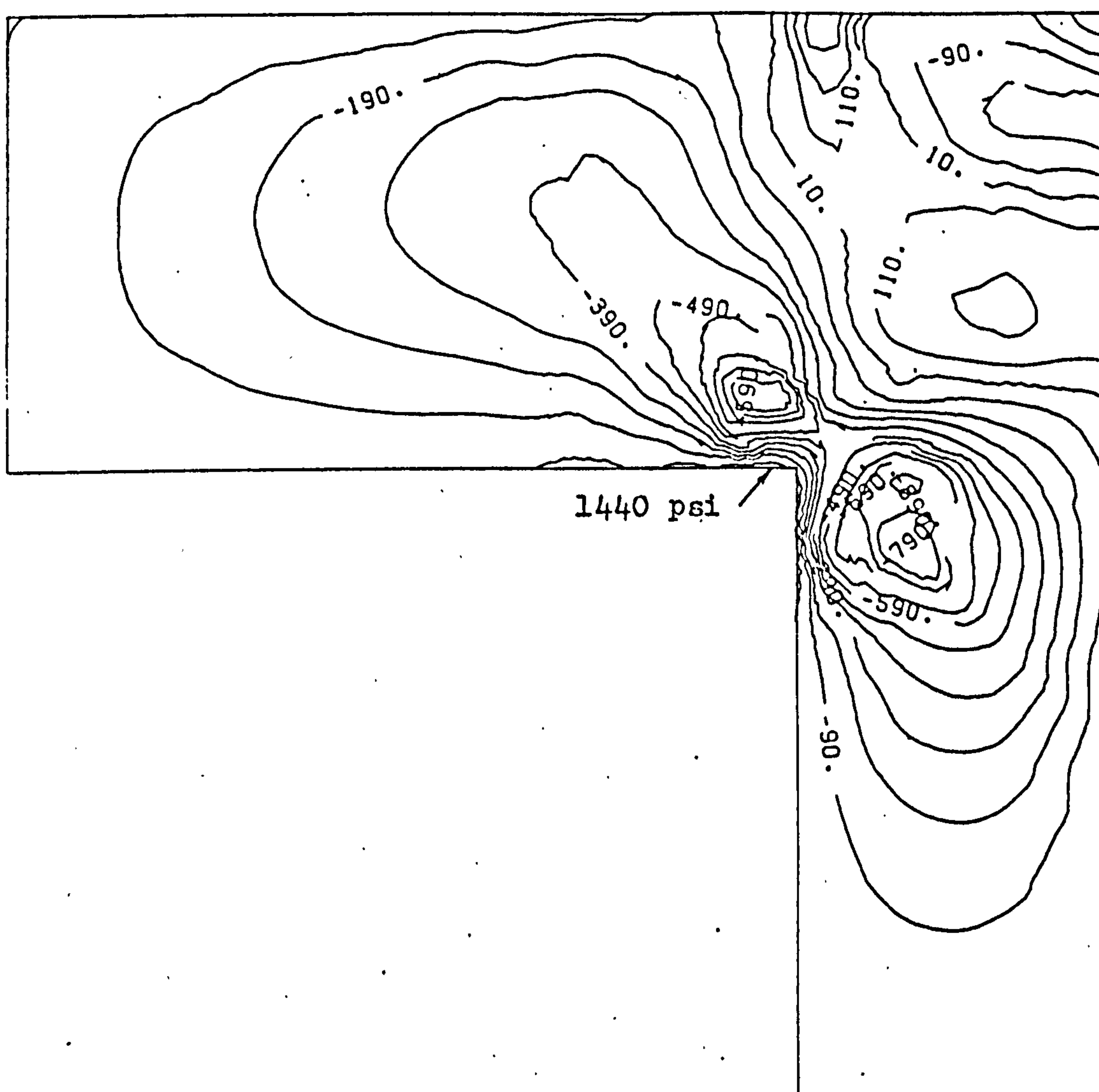


Figure 8.14 Contours of Shear Stresses at Low Temperature

Appendix 1

A1.1 Analysis of the Cylindrical Jacket as a Thin Shell

The Cylindrical "jacket" shell is under combined axial compression and external pressure that result, respectively, from axial friction and radial pressure. However, the preformation of circumferential convolutions close to the joint greatly reduces the axial stresses and prevents buckling in this direction. Therefore, the problem may be reduced to that of a two-dimensional ring. The ring is assumed to be bent elastically similarly to a straight strip restrained at two ends by moments and under a compression load P . With these simplifying assumptions, the bending equation for the problem becomes;

$$EI \frac{d\phi}{ds} = -Py + \frac{EI}{R} \quad (A1.1)$$

where

ϕ is the slope of the curve relative to chord joining B and C (see figure A1.1).

S is the distance along the curve.

R is the radius of the ring in the unstressed state.

EI is the flextural stiffness.

With the coordinate system as shown in figure A1.1 and the assumption that the buckled shape is represented by a sine curve, the deflection of the buckled part can be represented by (135).

$$y = y_c \sin \frac{\pi x}{L} \quad (A1.2)$$

The value of y_c is found to be

$$y_c = \sqrt{\left(\frac{L}{\pi}\right)^2 \tan^2 \alpha + \left(\frac{M}{P}\right)^2} \quad (A1.3)$$

The following expression relates axial load to the difference between the arc length and the chord length of a deformed structure in region beyond the Euler load.

$$\frac{\Delta L}{L} = 2\left(\frac{P}{PE} - 1\right) \quad (A1.4)$$

where

$$\Delta L = \text{length difference} = \frac{1}{4}L \tan^2 \alpha$$

$$L = \text{length of the structure} = 2R \sin \alpha$$

$$P = \text{compression load}$$

$$PE = \text{Euler load} = \frac{\pi^2 EI}{L^2} \quad (A1.5)$$

From the above the following value for P is obtained

$$P = \left(1 + \frac{1}{8} \tan^2 \alpha\right) \frac{\pi^2 EI}{4R^2 \sin^2 \alpha}$$

For approximation,

$$P = \frac{\pi^2 EI}{4R^2 \sin^2 \alpha} \quad (A1.6)$$

From the experiment carried out on the jacket, five vertical convolutions were formed by radial pressure. Perhaps the existing of welding line along the jacket is the main cause in reducing the number of convolutions from six to five. Therefore, to preserve symmetry, we consider them as six and α becomes 30° at the onset of buckling. If we replace P with σ_s (the steel stress) and σ_2 (the radial pressure) in eq. (A1.6), we arrive to the following;

$$\sigma_s = \frac{P}{A} = \frac{P}{lxt} = \frac{\pi^2 EI}{400k^2 \sin^2 \alpha} \quad (A1.7)$$

$$\sigma_2 = \frac{P}{R} = \frac{\pi^2 EI}{4R^3 \sin^2 \alpha} \quad (A1.8)$$

where t is the jacket thickness in in.

A is the area of the ring section

From eqs. (A1.6) (A1.7) and (A1.8) we can follow the process of buckling from the start up to the formation of vertical convolution. It appears from eq. (A1.8) that the increase in the radial pressure will reduce α . The results of this process are listed in table (A1.1). The steel reaches the yielding stress at the radial pressure of 80 P.S.I., and further increase in the radial pressure has a little significance on α .

If we release the radial pressure, the steel stress-strain curve would follow the pattern shown in figure A1.3 which indicates that the increase of radial pressure beyond 80 p.s.i. will make the convolutions permanent.

After the formation of the vertical convolutions, the calculation of the bending stress changes at the crowns of the vertical convolutions as the radial strain in the concrete specimen changes will be based on the following assumptions;

1. Restraint to bending of the convolutions by the

membrane resisting bending between them is negligible (ref. Fig. A1.3).

2. Ring thrust or tension per inch length resisted by the membrane is small even if friction with the concrete is high and from the zero $\bar{\epsilon}_2$ position = $0.01 \bar{\epsilon}_2 E_s$ up to steel yield at $\pm 24,000$ p.s.i. If friction is zero the ring stress is negligible.

3. Convolutions in "opening" or "closing" give a small bending resistance up to yield and beyond, so that very approximately full fluid pressure σ_2 is transmitted to the concrete.

$$4. \frac{E_c}{(1-\nu)} = 3 \times 10^6 \quad \text{then} \quad \bar{\epsilon}_2 = \frac{\sigma_2}{3 \times 10^6}$$

$$\begin{aligned} \text{Deflection of each half convolution} &= \frac{\text{change in circumference}}{12} \\ &= \frac{2 \pi R \bar{\epsilon}_2}{12} \dots (1) \end{aligned}$$

$$\text{and also} = \frac{M l^2}{3 E_s I} \dots \dots \dots (2)$$

Equating (2) to (1) and substituting for $\bar{\epsilon}_2$

$$\frac{1 \times 0.01^2 \times 24,000}{6} \times \frac{0.1^2 \times 12}{3 \times 3 \times 10^7 \times 0.01^3} = \frac{2 \pi R \sigma_2}{12 \times 3 \times 10^6}$$

gives $\sigma_2 = \underline{1,019}$ p.s.i. to change steel stress 24,000 p.s.i.

$$\bar{\epsilon}_2 = \underline{340 \times 10^{-6}}$$

Change of steel bending stress for σ_2 changing 1 = 23.6 p.s.i.

for $\bar{\epsilon}_2$ changing 1 = 72.0 p.s.i.

After reducing σ_2 to zero (ref. points (5) to (6)), σ_1 is reduced to zero (ref. points (6) to (7)), leaving a clearance between the concrete and the steel membrane, which facilitates extraction of the cylinder and insertion of a second specimen for which a similar cycle can be followed.

If the load σ_1 is sustained at (5), creep of the concrete increases $\bar{\epsilon}_2$ to point (5'). When σ_2 is released, $\bar{\epsilon}_2$ increases to point (6') and when σ_1 is reduced to zero $\bar{\epsilon}_2$ remains at (7') and the next specimen may be inserted.

Table A1.1

$\pi^2 EI$	α	$P(1b)$	$\sigma_s(\text{psi})$	$\sigma_2(\text{psi})$
24.67	30	2.74	274	0.91
"	3	240	24000	80
"	1.22	1500	2400	500

Table A1.2

Points of Diagram (Figure A1.1)	Changes			σ_2 has closed 0.02 in gap between steel and concrete
	$\bar{\epsilon}_2 \times 10^6$	σ_2 psi	σ_s psi	
(1)	0	0	Yield 24,000	
(1) to (2)	100	300	Yield 24,000	
(2) to (3)	- 100	- 300	- 7,080	
(3) to (4)	- 500	0	- 36,000	
(4) to (5)	+ 167	+ 500	+ 11,800	
(5) to (6)	- 167	- 500	- 11,800	
(6) to (7)	+ 500	0	+ 36,000	σ_1 released
(5) to (5')	- 100	0	- 7,200	Concrete creeps
(5') to (6')	- 167	- 500	- 11,800	
(6') to (7')	+ 500	0	+ 36,000	σ_1 released

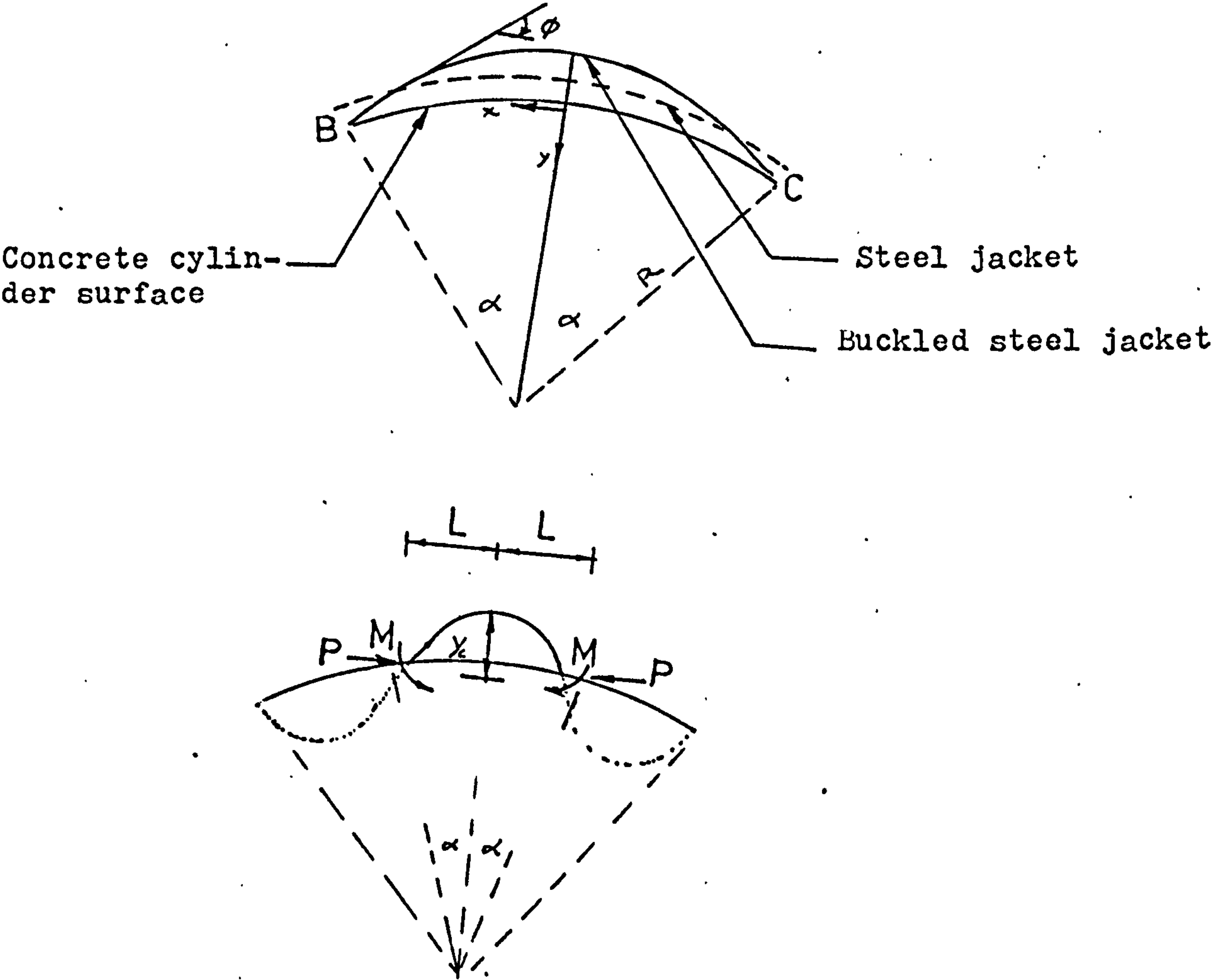


Figure A1.1 Initial Buckling Process

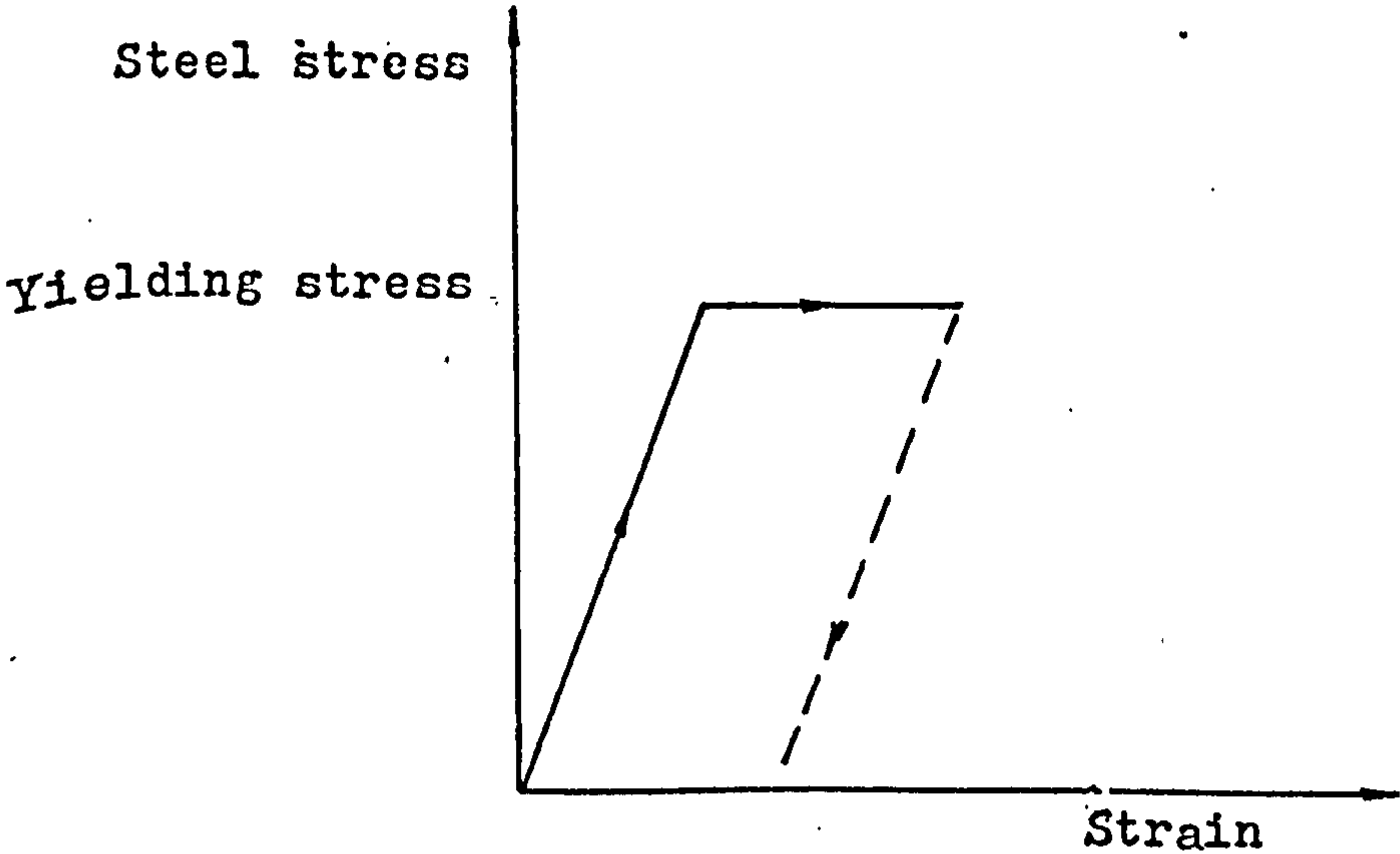


Figure A1.2 Stress-Strain Diagram

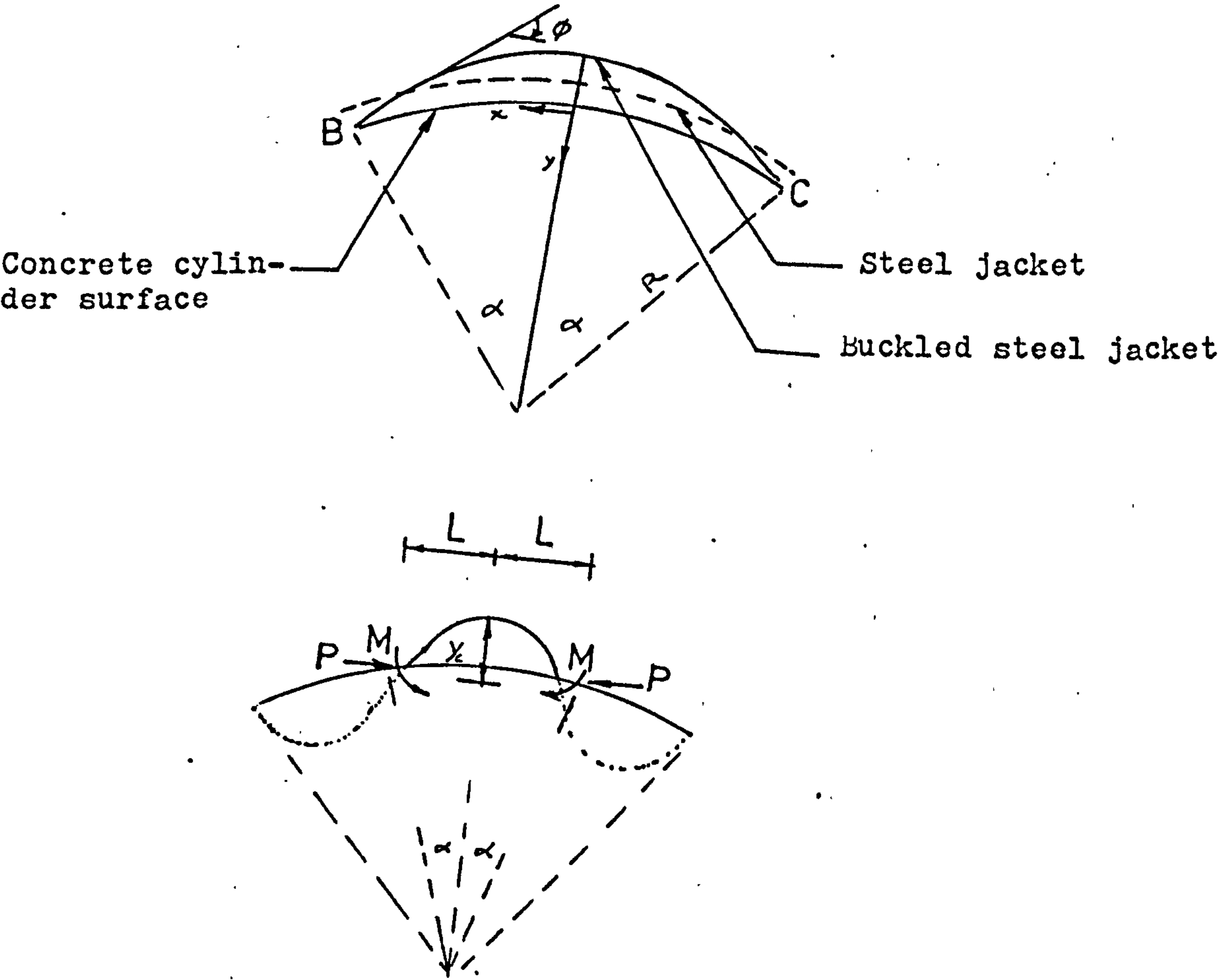


Figure A1.1 Initial Buckling Process

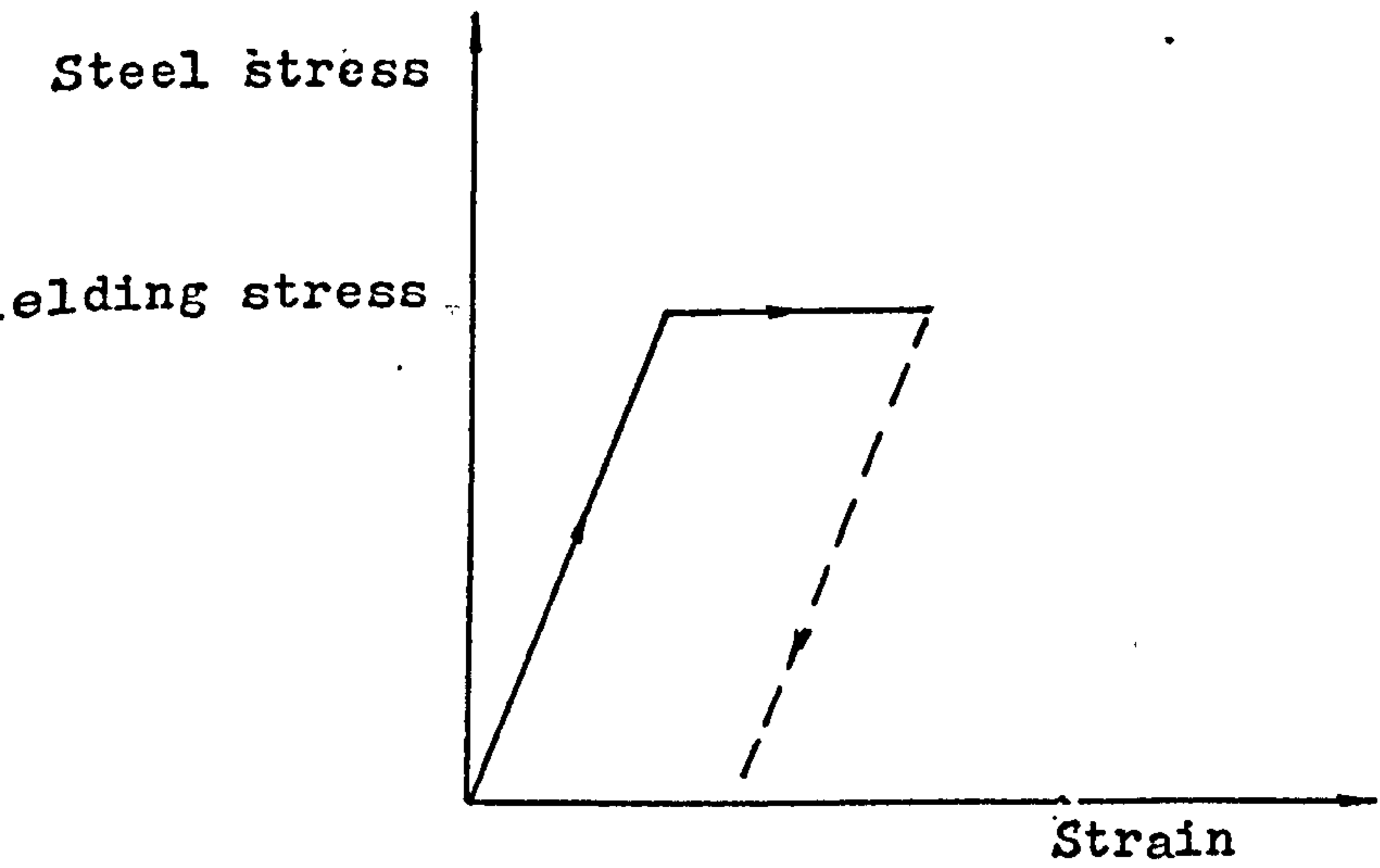


Figure A1.2 Stress-Strain Diagram

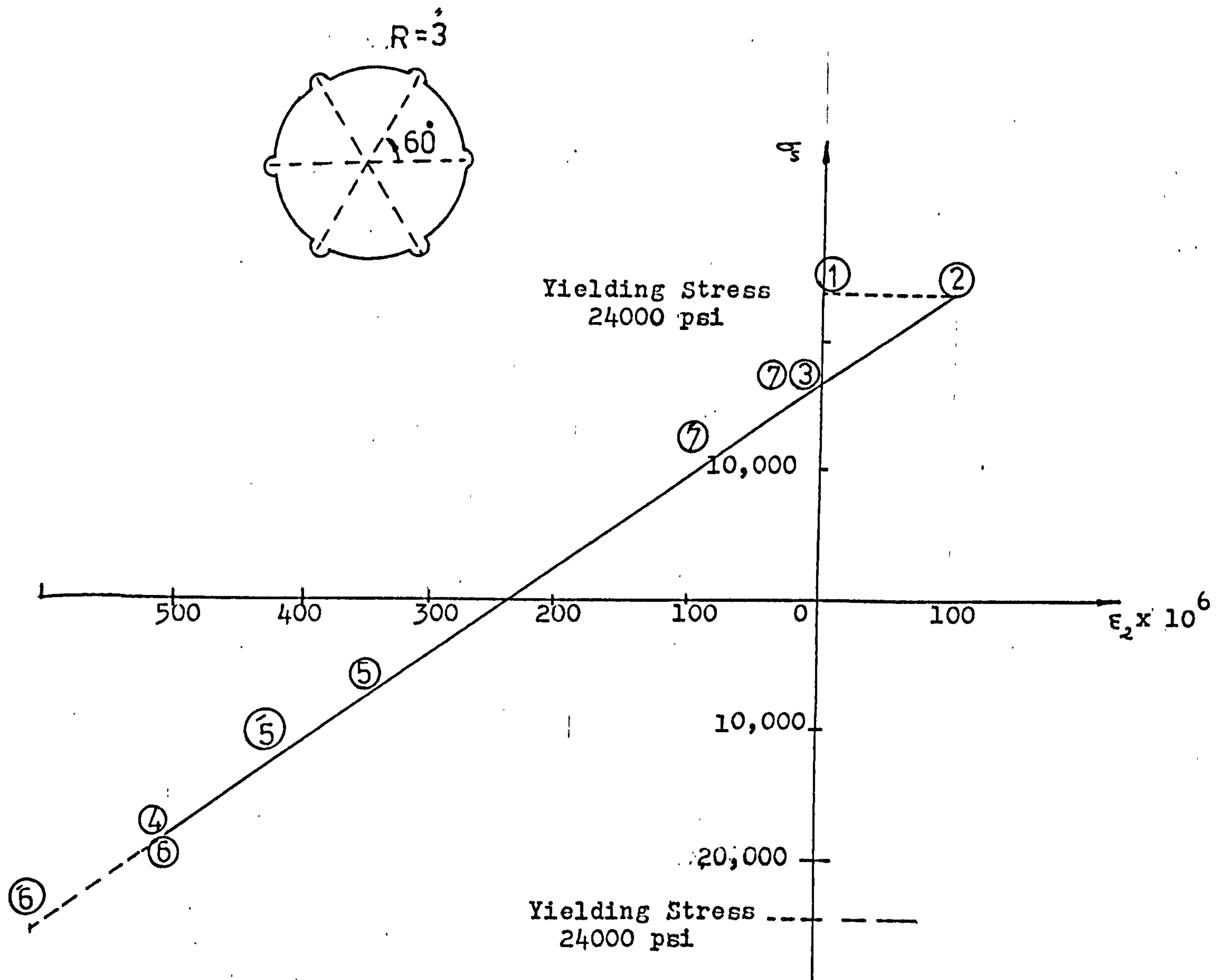
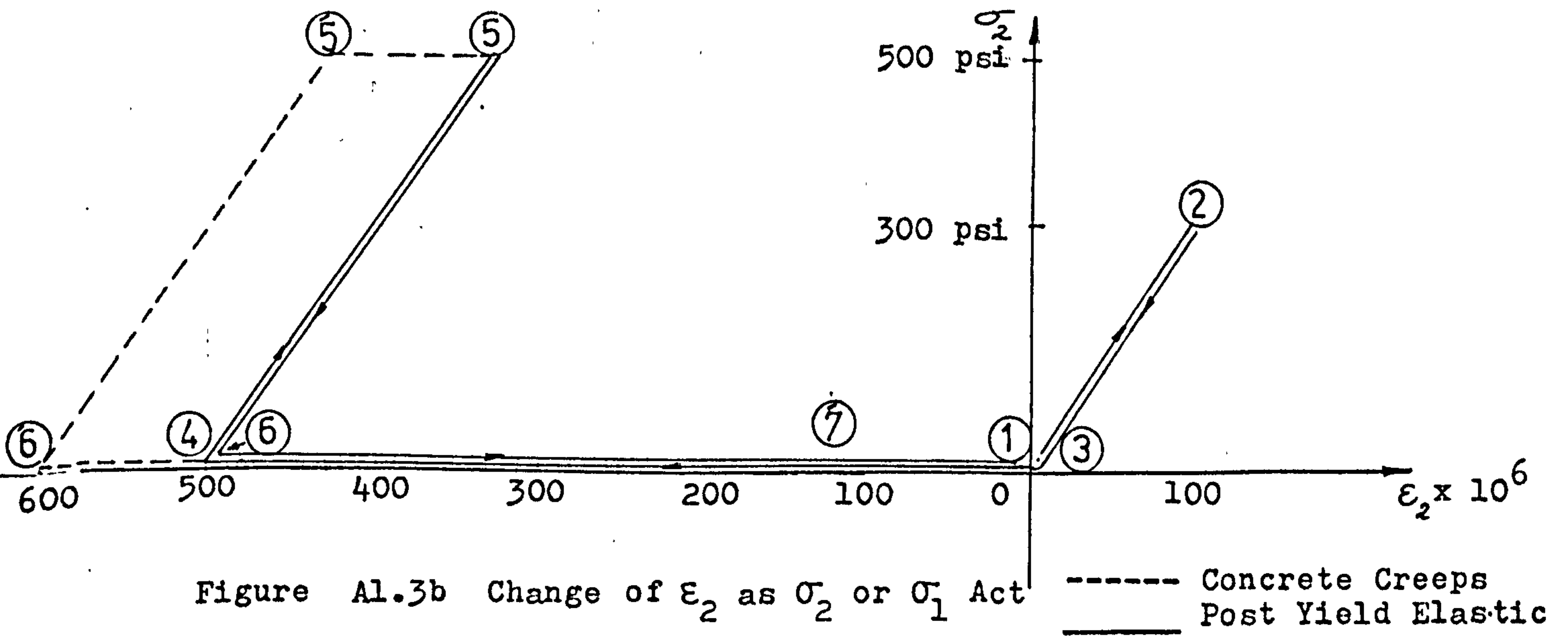


Figure A1.3a Change of Steel Stress at Point of Convolution



Appendix 2

A2.1 The Finite-Element Procedure

A continuous solid is idealized as an assemblage of elements in order to find an alternative form of the equilibrium equations, which are easier to solve than the governing differential equations of the continuous solid. One thereby reduces the problem from the solution of a system of differential equations to solution of an equivalent set of algebraic equations. This is achieved by the following procedure;

1. After an appropriate finite-element grid for the body to be analysed has been selected, admissible displacement fields for the elements are assumed in the form of polynomials.
2. By appropriate differentiation of the assumed element displacement functions, the element strains can be obtained.
3. The unknown coefficients of the displacements polynomials are related to the element nodal displacements by evaluating the displacement functions at the nodal points.
4. In steps 1 through 3 the element strains are expressed as linear combinations of the element nodal displacements.
5. The element stress-strain relations are represented by the generalized Hooke's law for isotropic, orthotropic, or anisotropic material.

6. The strain energy of the element is formulated by taking the inner product of the strains with the stresses. The external energy is the negative of the inner product of the nodal forces and the nodal displacements. The sum of the two quantities is the total potential energy of the element.

7. By differentiating the energy expression with respect to the displacement components in succession and setting the results equal to zero, a relation between the nodal forces and the nodal displacements is obtained. The coefficients relating the nodal forces to the nodal displacements are the stiffness coefficients of the element. The element nodal forces, which are statically equivalent to body forces and temperature loads, are derived by energy considerations such as those described above.

8. By superposition of the individual element stiffnesses contributing to each nodal point force, the stiffness matrix $[K]$ of the total assemblage of elements is obtained. This involves only simple matrix addition when all element stiffnesses have been expressed in the same coordinate system.

9. The equilibrium equation $[K] \{V\} = \{F\}$, where $\{V\}$ and $\{F\}$ are the nodal point displacement and force vectors, respectively, is solved. Since in most cases the number of equations to be solved runs into several hundreds or even thousands, this step is significant and may require special solution techniques.

10. The element strains are evaluated from the computed

nodal displacements by kinematic relations, and, finally, the element stresses are determined from the element strains by means of the stress-strain relations.

The dependability of the method is strongly controlled by the assumed deformation patterns of the individual elements. Therefore, if the finite element analysis procedure is to be meaningful, the assumed displacement field must satisfy the following requirements:

1. The deformation between adjacent elements must be compatible. No gaps or overlaps are permitted in the deformed finite-element system.
2. The displacement field must include all possible rigid body displacements of the element.
3. The displacement field must be continuously differentiable up to and including the highest derivative required in the formulation of the element strain energy function.

A2.2 Development of the Governing Equations

Consider the triangular element as shown in fig. A2.1. The coordinate system for this element is taken as a cylindrical system with the z-axis coincident with the axis of symmetry. The displacements of the element are assumed to be of the following form (ref. 136):

$$\begin{Bmatrix} u(r,z) \\ w(r,z) \end{Bmatrix} = \begin{bmatrix} 1 & r & z & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & r & z \end{bmatrix} \begin{Bmatrix} A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \end{Bmatrix} \quad (\text{A2.1})$$

or symbolically

$$\{V\} = [\phi] \{A\} \quad (\text{A2.2})$$

By evaluating eq. (A2.1) at each of the three vertices of the triangle, the following relation is obtained

$$\{V_i\} = [M] \{A\} \quad (\text{A2.3})$$

The generalized displacement vector A , is now expressed in terms of nodal point displacement by inverting eq. (A2.3), i.e.,

$$\{A\} = [M]^{-1} \{V_i\} = [B] \{V_i\} \quad (\text{A2.4})$$

Where

$$\begin{aligned} a_i &= \{r_j z_k - r_k z_j\}, \quad b_i = \{z_j - z_k\}, \quad c_i = \{r_k - r_j\} \\ B &= [B_i, B_j, B_k] \\ B_i^T &= \frac{1}{\beta} \begin{bmatrix} a_i & b_i & c_i & 0 & 0 & 0 \\ 0 & 0 & 0 & a_i & b_i & c_i \end{bmatrix} \end{aligned} \quad (\text{A2.5})$$

and

$$\beta = r_i b_i + r_j b_j + r_k b_k \quad (\text{A2.6})$$

The strain displacement relation in cylindrical coordinates are given by

$$\epsilon_r = \frac{\partial u}{\partial r}, \quad \epsilon_\theta = \frac{u}{r}, \quad \epsilon_z = \frac{\partial w}{\partial z}, \quad \gamma_{rz} = \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial r} \right). \quad (\text{A2.7})$$

Substituting eqs. (A2.1) into (A2.7) yields

$$\begin{Bmatrix} \epsilon_r \\ \epsilon_z \\ \epsilon_\theta \\ \gamma_{rz} \end{Bmatrix} = \begin{bmatrix} 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \\ \frac{1}{r} & 1 & \frac{z}{r} & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 1 & 0 \end{bmatrix} \begin{Bmatrix} A_1 \\ A_2 \\ A_3 \\ A_4 \\ A_5 \\ A_6 \end{Bmatrix} \quad (\text{A2.8})$$

or

$$\{\epsilon(r,z)\} = [N(r,z)] \{A\}. \quad (\text{A2.9})$$

For the solution of the creep problem the strain $\{\epsilon(r,z)\}$ is assumed to be composed of the elastic strain, thermal strain, and creep strain

$$\{\epsilon(r,z)\} = \{\epsilon^E(r,z)\} + \{\epsilon^C(r,z)\} + \{\alpha\Delta T\}, \quad (\text{A2.10})$$

where

$$\{\epsilon^E(r,z)\} = \text{Elastic strain}$$

$$\{\epsilon^C(r,z)\} = \text{Creep strain}$$

$$\{\alpha\Delta T\} = \text{Thermal strain}$$

Solving eq. (A2.10) for the elastic strain gives

$$\{\epsilon^E(r,z)\} = \{\epsilon(r,z)\} - \{\epsilon^C(r,z)\} - \{\alpha\Delta T\} \quad (\text{A2.11})$$

The stress strain relations for an elastic isotropic material are

$$\begin{Bmatrix} \sigma_r \\ \sigma_z \\ \sigma_\theta \\ \sigma_{rz} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 \\ \nu & 1-\nu & \nu & 0 \\ \nu & \nu & 1-\nu & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix} \begin{Bmatrix} \epsilon_r^E \\ \epsilon_z^E \\ \epsilon_\theta^E \\ \gamma_{rz}^E \end{Bmatrix} \quad (\text{A2.12})$$

Substituting eq. (A2.11) into (A2.12) and using the relation that the creep strains are incompressible yields

$$\begin{Bmatrix} \sigma_r \\ \sigma_z \\ \sigma_\theta \\ \sigma_{rz} \end{Bmatrix} = [E] \begin{Bmatrix} \epsilon_r \\ \epsilon_z \\ \epsilon_\theta \\ \gamma_{rz} \end{Bmatrix} - 2G \begin{Bmatrix} \epsilon_r^C \\ \epsilon_z^C \\ \epsilon_\theta^C \\ \gamma_{rz}^C \end{Bmatrix} - \frac{E\alpha T}{1-2\nu} \begin{Bmatrix} 1 \\ 1 \\ 1 \\ 0 \end{Bmatrix} \quad (\text{A2.13})$$

where

$$[E] = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 \\ \nu & 1-\nu & \nu & 0 \\ \nu & \nu & 1-\nu & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix}$$

or symbolically

$$\{\sigma\} = [E] \{\epsilon(r,z)\} - 2G \{\epsilon^C(r,z)\} - \{\epsilon^T\} \quad (\text{A2.14})$$

where

$$G = \frac{E}{2(1+\nu)}.$$

The stiffness matrix for the element may now be determined from the theorem of Minimum Potential Energy

$$\delta U = 0. \quad (\text{A2.15})$$

The potential energy for the element is

$$\begin{aligned} U = & \frac{1}{2} \int_V \left\{ \epsilon(r,z) \right\}^T [E] \left\{ \epsilon(r,z) \right\} dV - 2G \int_V \left\{ \epsilon(r,z) \right\}^T \left\{ \epsilon^C(r,z) \right\} dV \\ & - \int_V \left\{ \epsilon(r,z) \right\}^T \left\{ \epsilon^T \right\} dV - \int_{A_p} \left\{ v(r,z) \right\}^T \left\{ P \right\} dA_p, \end{aligned} \quad (\text{A2.16})$$

where

V = volume of the element

A_p = area over which the surface forces P are specified

Substituting eqs. (A2.2), (A2.9) and (A2.16) into (A2.15) gives

$$\delta \left\{ \frac{1}{2} \int_V \{A\}^T [N(r,z)]^T [E] [N(r,z)] \{A\} dV - 2G \int_V \{A\}^T [N(r,z)]^T \{\epsilon^C(r,z)\} dV \right. \\ \left. - \int_V \{A\}^T [N(r,z)]^T \{\epsilon^T\} dV - \int_{A_P} \{A\}^T [\phi(r,z)]^T \{P\} dA_P \right\} = 0. \quad (A2.17)$$

Taking variations of (A2.17) with respect to the generalized displacements $\{A\}$, yields the following system of equations:

$$[K] \{A\} = \{F\} \quad (A2.18)$$

where

$$\{K\} = \int_V [N(r,z)]^T [E] [N(r,z)] dV \\ \{F\} = 2G \int_V [N(r,z)]^T \{\epsilon^C(r,z)\} dV + \int_V [N(r,z)]^T \{\epsilon^T\} dV + \\ \int_{A_P} [\phi(r,z)]^T \{P\} dA_P \\ = \{F^C\} + \{F^T\} + \{F^P\}.$$

Thus far the stiffness matrix and load vector which have been presented are with respect to the generalized displacements $\{A\}$, equation (A2.18). However, it is much more convenient for combining stiffness matrices of adjacent element to work with the nodal displacements. Since the nodal displacements are related to the generalized displacements through eq. (A2.5), substituting into (A2.18) and premultiplying by $[B]^T$ yields the desired relationship

$$[B]^T [K] [B] \{v_i\} = [B]^T \{F\} \quad (A2.19)$$

or

$$\{K^*\} \{v_i\} = \{F_i^*\} \quad (A2.20)$$

where

$$[K^*] = [B]^T [K] [B], \quad (A2.21)$$

$$\{F_i^*\} = [B]^T \{F\}. \quad (A2.22)$$

Once the element stiffness matrix and load vector have been evaluated, they are assembled into a set of $2N$ equations of equilibrium for the body (N = number of nodal points). Equilibrium requires that all element forces at node i be in equilibrium with the external forces at that node. This is accomplished by adding the equations of all elements associated with the forces at that point, i.e.,

$$\{P\} = \sum_{m=1}^M \{F_i^*\}_m \quad (A2.23)$$

where M = number of elements.

Substituting eq. (A2.20) into (A2.23), and specifying that the displacement at any given node be the same for all elements attached to that node (i.e., compatibility), gives

$$\{P\} = \sum_{m=1}^M \{F_i^*\}_m = \sum_{m=1}^M [K^*]_m \{v_i\} \quad (A2.24)$$

or

$$[K] \{v_i\} = \{P\} \quad (A2.25)$$

where $[K]$ is a $2N \times 2N$ symmetric matrix. After the stiffness matrix for the body is assembled, displacement boundary conditions are specified and the matrix is constrained. The constrained equations of equilibrium are then solved by the method discussed in section (8.4)

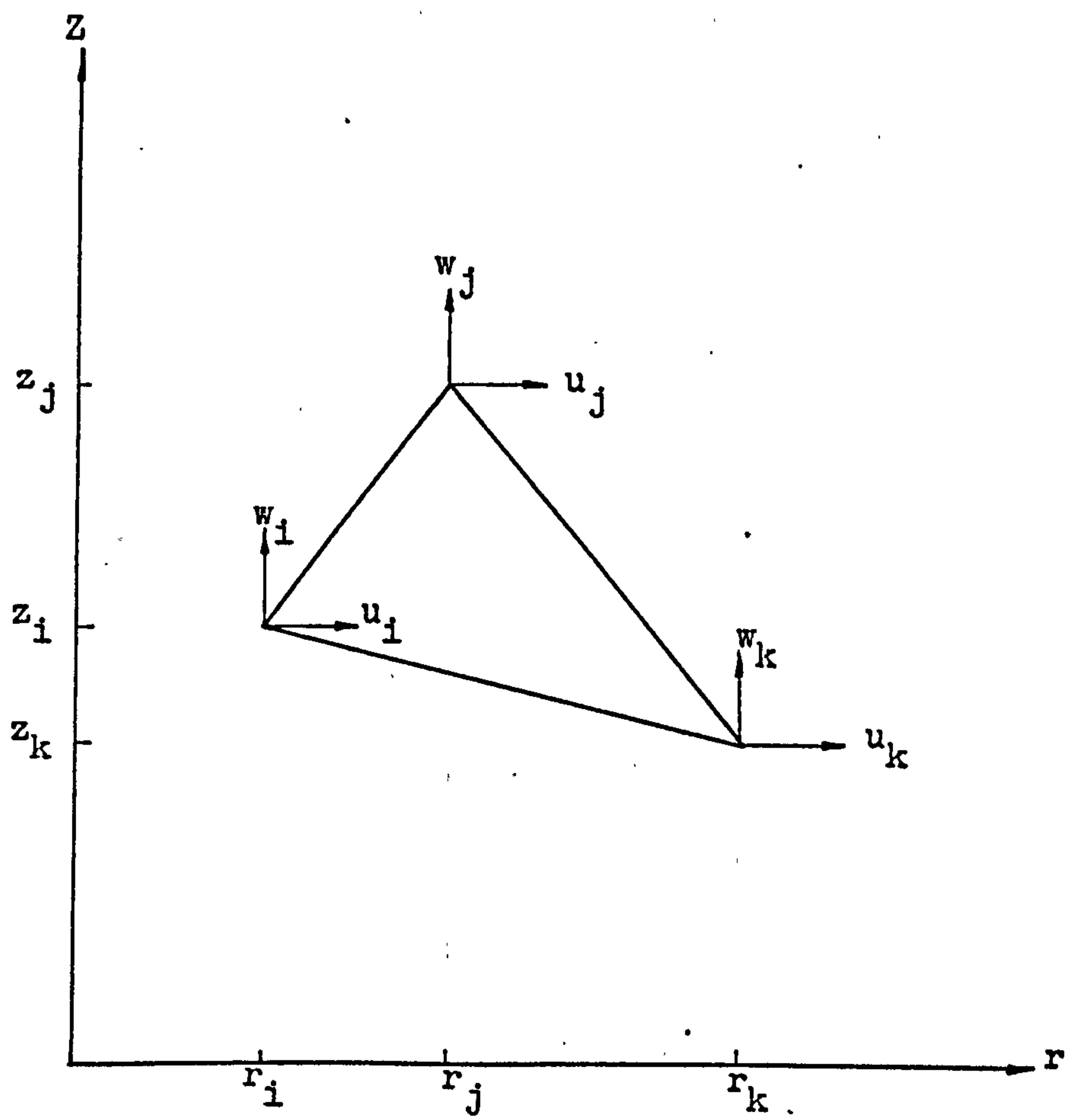


Figure A2.1 Axisymmetric Triangular Element

Appendix 3

A3.1 PROGRAM THERMO

The Variable Name List for the Main-program

<u>VARIABLE</u>	<u>DEFINITION</u>
AREA	area of a triangular element.
ANP	store for the number of adjacent nodes. associated with each node and for the number of the node itself.
BJ, BK CJ, CK	local element coordinates of the nodes.
CONST	constants used in function subprogram COND.
D	element geometry (dimension) matrix.
DT	transpose of D.
Q	the heat generated at the centroid of each element in the concrete due to radiation.
H	thermal load matrix.
IDENT	identifier for state of heat conduction: IDENT = 3 plane, IDENT = 4 axi-symmetric.
IN	specified variable for input unit.
IO	control parameter for mesh data output; IO = 0 none, IO = 1 coordinates of nodes, IO = 2 indices of nodes, IO = 3 both.

ITERA	cycle counter for conductivity iteration.
ITMAX	maximum number of iterations for conductivity.
ITPO	cycle interval for writing the Gauss-Seidel iteration progress.
JOUT	specified variable for output unit.
KEL	element conductivity matrix.
KT	overall conductivity matrix (non-zero terms only).
L	element counter.
LN	temporary store for element node numbers.
MAXIT	maximum number of Gauss-Seidel iterations.
N	nodal point counter.
NAP	counter for the total number of adjacent nodes associated with each nodal point.
NBP	total number of prescribed boundary nodes.
NEL	total number of elements.
NI,NJ,NK	numbers of element nodes.
NNP	total number of nodal points.
OMEGA	overrelaxation factor.
Q	thermal power generated per unit volume.
SDELK	summed magnitudes of changes in conductivity between successive cycles.
TEXT	output text.
THETA	required nodal temperatures.
THETAI	initial temperatures.
TITLE	input title being problem heading.
TOLER	Gauss-Seidel convergence criteria.

X	abscissae of the nodes.
Y	ordinates of the nodes.

Variable Name List for subroutine FORMKH

<u>VARIABLE</u>	<u>DEFINITION</u>
COND	thermal conductivity
CONDOC	"
CONFAC	thermal conductivity X FACTOR
CONOLD	previous thermal conductivity
FACTOR	$0.25/\text{AREA}$
HEL	element thermal load vector
I,J,K	indices of element nodes
MAT	material identifier
NA	adjacent nodal point counter
PI2	$2 \times \pi$
TEMP	temperature at the element centroid

Variable Name List for subroutine Bound

BDTEMP	prescribed boundary nodal temperatures.
FACT	$1.0\text{E } 20$
IBN	number of nodes along a particular heat transfer boundary.
NB	numbers of the prescribed boundary layers.
NPB	counter for boundary nodes.
NP	number of a particular node.

Variable Name List for subroutine SOLVE

<u>VARIABLE</u>	<u>DEFINITION</u>
DELT	change in THETA between successive cycles.
IT	iteration cycle counter.
NA	counter for the total number of adjacent nodes associated with each nodal point.
NAN	adjacent nodal point number.
NP	number of adjacent nodal points associated with a particular node.
RELDIF	SMDELT/SIGMAT
SMDELT	sum of the magnitudes of changes in THETA between successive cycles of iteration.
SIGMAT	sum (Σ) of the magnitudes of THETA.
THETAN	new value of THETA before overrelaxation.

The Variable Name List for subroutine SOURCE

<u>VARIABLE</u>	<u>DEFINITION</u>
ZI	internal half height of cylindrical vessel.
RI	internal radius of cylindrical vessel.

The Variable Name List for FUNCTION COND

<u>VARIABLE</u>	<u>DEFINITION</u>
COND	thermal conductivity of CONCRETE.

```

PROGRAM THERMO (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT,PUNCH)
C
C TWO DIMENSIONAL HEAT CONDUCTION ANALYSIS CODE USING THE FINITE ELEMENT METHOD
C
      INTEGER CASE,TITLE,TEXT, ANP
      REAL KT, KEL
      COMMON X(121),Y(121),NI(192),NJ(192),NK(192),NEL,NNP,NBP,GAP,
1          THETA(192),D(2,3),DT(3,2),KEL(3,3),LN(3),H(121),
2          JBOUT,IDENT,Q(192)
      COMMON /KT(121,20),ANP(121,20),NAP(121)
      DIMENSION TITLE(13),TEXT(3),N(192),L(192)
      DATA (TEXT(J),J = 1,3)/6HAXISYM,6HMETRIC,6H /
C      SPECIFY VARIABLES FOR I/O UNITS.
      IN = 5
      JBOUT = 6
C      GENERAL HEADING
      READ (IN,1000) TITLE
      WRITE (JBOUT,1010) TITLE
C      I/O FINITE ELEMENT MESH PARAMETERS.
      READ (IN,1020) NEL,NNP,NBP,IDENT,TEXT,I0
      WRITE (JBOUT,1030) NEL,NNP,NBP,IDENT,TEXT
1000 FORMAT (13A6)
1010 FORMAT (1H1/////////26X,13A6/26X,78(1H=) //)
1020 FORMAT (4I5,3A6,I5)
1030 FORMAT (15X,26H THE NUMBER OF ELEMENTS IS,I5,1H./
1          15X,29H THE NUMBER OF NODALPOINTS IS,I5,1H./
2          15X,43H THE NUMBER OF PRESCRIBED BOUNDARY NODES IS,I5,1H./
3          15X,26H THE PROBLEM IDENTIFIER IS,I5,5H FOR ,3A6,6H CASE./
4//)
      READ (IN,2000) (N(I),X(I),Y(I),I=1,NNP)
2000 FORMAT (5X,I5,6X,F8.4,2X,F8.4)
      READ (IN,2010) (L(J),NI(J),NJ(J),NK(J),J=1,NEL)
2010 FORMAT (4I10)
C      WRITE (JBOUT,1080)
1080 FORMAT (15X,51H THE BOUNDARY CONDITIONS ARE PRESCRIBED TEMPERATURE
1          30HS AT SPECIFIED BOUNDARY NODES./ )
      READ (IN,1090) ITP0,MAXIT,OMEGA,TOLER
      WRITE (JBOUT,1100) ITP0,MAXIT,OMEGA,TOLER
1090 FORMAT (2I5,F7.3,E13.5)
1100 FORMAT (36H0THE INTERVAL FOR PROGRESS OUTPUT IS, I5, 8H CYCLES. /
1          23H0THE ITERATION LIMIT IS, I5, 8H CYCLES. /
2          30H0THE OVER-RELAXATION FACTOR IS, F7.3, 1H. /
3          29H0THE CONVERGENCE TOLERANCE IS, E13.5, 1H. /)
C      DIMENSION ARE COVERED INTO METERS
      DO 500 I=1,NNP
      X(I)=X(I)/39.37
      Y(I)=Y(I)/39.37
500 CONTINUE
      IF (I0 .EQ. 0) GO TO 70
      IF (I0 .EQ. 1) GO TO 40
      IF (I0 .EQ. 2) GO TO 50
      IF (I0 .EQ. 3) GO TO 60
40 WRITE (JBOUT,1130) (N(I),X(I),Y(I),I=1,NNP)
      GO TO 80
50 WRITE (JBOUT,1140) (L(J),NI(J),NJ(J),NK(J),J=1,NEL)
      GO TO 80
60 WRITE (JBOUT,1130) (N(I),X(I),Y(I),I=1,NNP)
      WRITE (JBOUT,1140) (L(J),NI(J),NJ(J),NK(J),J=1,NEL)
      GO TO 80
70 WRITE (JBOUT,1110)
80 CONTINUE
1110 FORMAT (15X,30H NO MESH DATA OUTPUT REQUIRED./)

```

```

1130 FORMAT (1H1,130(1H=)/1X,5(26H NO-      COORDINATES      )/
1      1X,5(26H DE.      X      Y      )/1X,130(1H=)/
2      (5(I5,1X,F8.4,2X,F8.4,2X))
1140 FORMAT (1X,130(1H=)/1X,6(20H EL-      NODALPOINTS )/
1      1X,6(20H NO.      I      J      K )/1X,130(1H=)/(1X,24I5)/
      CALL SOURCE
      ITMAX = 100
      ITERA = 0
90 SDELK = 0.
      CALL FORMK
      CALL BOUND
      CALL SOLVE
      IF (ITERA . EQ . 1) GO TO 90
      IF (ITERA . EQ . 2) WRITE (JOUT,1150)
1150 FORMAT (1H1///
1      34X,46H THE PROGRESS OF THE ITERATION FOR CONDUCTIVITY/
2      34X,46H=====
3      34X,44H ITERA      CONDUCTIVITY DIFFERENTIAL /
4      34X,44H=====
      IF (SDELK . LT . 0.0001) GO TO 100
      WRITE (JOUT,1160) ITERA,SDELK
1160 FORMAT (34X,I3,14X,E15.5/)
      IF (ITERA . LT . ITMAX) GO TO 90
100 WRITE (JOUT,1170) (N(I),THETA(I),I=1,NNP)
1170 FORMAT (1H1////////
1      49X, 33H THE CALCULATED NODE TEMPERATURES /
2      49X, 33H=====
3      8(16H NODE      THETA)/128(1H=)/(8(I6,F10.1)) )
      PUNCH 6461, (N(I),THETA(I),I=1,NNP)
6461 FORMAT(8(I3,F7.1))
      STOP
      END
      SUBROUTINE FORMKH
      INTEGER CASE,      ANP
      REAL      KT,      KEL
      COMMON X(121),Y(121),NI(192),NJ(192),NK(192),NEL,NNP,NBP,GAP,
1      THETA(192),D(2,3),DT(3,2),KEL(3,3),LN(3),H(121),
2      JOUT,IDENT,Q(192)

      COMMON/ /KT(121,20),ANP(121,20),NAP(121)
      COMMON/INITIA/CONDUCT(192)
      DIMENSION THETA1(192)
      EQUIVALENCE (THETA1(1),CONDUCT(1))
      DATA PI2/6.2831853072/
      DO 20 N=1,NNP
        DO 10 NA=1,20
          KT(N,NA) = 0.
          ANP(N,NA) = 0
10      CONTINUE
          ANP(N,1) = N
          NAP(N) = 1
          H(N) = 0.
20      CONTINUE
          DO 110 L = 1,NEL
            I = NI(L)
            J = NJ(L)
            K = NK(L)
            BJ = Y(K)-Y(I)
            BK = Y(I)-Y(J)
            CJ = -X(K)+X(I)
            CK = -X(I)+X(J)
            AREA = (BJ*CK-BK*CJ)*0.5
            FACTOR=0.25/AREA
            IF (IDENT . EQ . 4) FACTOR = PI2*FACTOR*(X(I)+X(J)+X(K))/3.

```

```

D(1,1) = -BK-BJ
D(1,2) = BJ
D(1,3) = BK
D(2,1) = -CJ-CK
D(2,2) = CJ
D(2,3) = CK
S=Q(L)*AREA
IF (IDENT.EQ.4) HEL=PI2*S*(X(I)+X(J)+X(K))/3.
H(I) = H(I)+HEL
H(J) = H(J)+HEL
H(K) = H(K)+HEL
TEMP = THETA(L)
IF (ITERA . EQ . 0) THETA(L) = 0.
IF (ITERA . GE . 1) TEMP = (THETA(I)+THETA(J)+THETA(K))/3.
  DO 40 I = 1,3
    DO 30 J = 1,2
      DT(I,J) = D(J,I)
30    CONTINUE
40    CONTINUE
CASE = 2
IF (ITERA . GE . 1) GO TO 35
CONDC(L) = COND(TEMP,DOSE,CASE)
CONFAC = CONDC(L)*FACTOR
GO TO 45
35 CONOLD = CONDC(L)
CONDC(L) = COND(TEMP,DOSE,CASE)
CONFAC = CONDC(L)*FACTOR
SDELK = SDELK+ABS(CONOLD-CONDC(L))
45 DO 60 I = 1,3
  DO 50 J = 1,3
    KEL(I,J) = (DT(I,1)*D(1,J)+DT(I,2)*D(2,J))*CONFAC
50    CONTINUE
60    CONTINUE
LN(1) = NI(L)
LN(2) = NJ(L)
LN(3) = NK(L)
DO 100 I = 1,3
  N = LN(I)
  DO 90 J = 1,3
    NA = 0
65    NA = NA+1
    IF (NA . LE . 20) GO TO 75
    WRITE (JOUT,1000) NA
    STOP
75    IF (ANP(N,NA) . EQ . LN(J)) GO TO 85
    IF (ANP(N,NA) . NE . 0) GO TO 65
    ANP(N,NA) = LN(J)
    NAP(N) = NA
85    KT(N,NA) = KT(N,NA)+KEL(I,J)
90    CONTINUE
100   CONTINUE
110 CONTINUE
1000 FORMAT (1H0//// 5X,30HBAND WIDTH SHOULD BE INCREASED )
RETURN
END
SUBROUTINE BOUND

REAL KT(121,20)
DIMENSION H(121),NB(80),BDTEMP(121)
IN=5
READ(IN,140) (NB(J),BDTEMP(NB(J)),J=1,40)
140 FORMAT (I5,F6.1)
FACT=1.E20

```

```

      DO 25 N=1,NBP
      NP=NB(N)
      KT(NP,1)=FACT
      H(NP)=BDTEMP(NP)*FACT
25  CONTINUE
      RETURN
      END
      SUBROUTINE SOLVE
      INTEGER CASE, ANP
      REAL KT, KEL
      COMMON X(121),Y(121),NI(192),NJ(192),NK(192),NEL,NNP,NBP,GAP,
1  THETA(192),D(2,3),DT(3,2),KEL(3,3),LN(3),H(121),
2  JOUT,IDENT,Q(192),
      COMMON/ /KT(121,20),ANP(121,20),NAP(121)
C
      IF (ITP0 . GT . MAXIT) GO TO 5
      IF (ITERA . EQ . 0) WRITE (JOUT,1000)
1000 FORMAT (1H1 //)
1  34X, 35H THE PROGRESS OF THE GAUSS - SEIDEL ,
2  28H ITERATION TOWARD CONVERGENCE /
3  34X, 35H _____,
4  28H _____ //
5  33X, 39H ITERNO CONVERGENCE CRITERION,
6  26H RELATIVE DIFFERENCE)
      DO 120 N = 1,NNP
      THETA(N) = 0.
120 CONTINUE
      5 IT = 0
      15 SMDLT = 0.
      SIGMAT = 0.
      DO 30 N = 1,NNP
      NP = NAP(N)
      THETAN = H(N)
      DO 20 NA = 2,NP
      NAN = ANP(N,NA)
      THETAN = THETAN - KT(N,NA) * THETA(NAN)
20 CONTINUE
      THETAN=THETAN/KT(N,1)
C  LIMITING THE SOLUTION BETWEEN THE MAX. AND MIN. VALUES
      IF (THETAN . GT . 180. ) THETAN=180.
C  FIXINGKEEPING SOME VALUES AT SOME NODE CONST. DUE T
      IF (N.GE. 60 .AND. N .LE. 63 ) THETAN=68.
      IF ( N .GE. 48 .AND. N .LE. 51 ) THETAN=69.
      IF (THETAN .LE.0.) THETAN=25.
      DELT = THETAN - THETA(N)
      THETA(N) = THETA(N) + OMEGA * DELT
      SMDLT = SMDLT + ABS(DELT)
      SIGMAT = SIGMAT + ABS(THETA(N))
30 CONTINUE
      RELDIF = SMDLT/SIGMAT
C  CONVERGENCE TEST
      IT = IT + 1
      IF (ITP0 . GT . MAXIT . OR . MOD (IT,ITP0) . NE . 0) GO TO 35
      IF (ITERA . EQ . 0) WRITE (JOUT,1010) IT,TOLER,RELDIF
1010 FORMAT (1X,I36,3X,2E27.5)
      35 IF (RELDIF . GT . TOLER) GO TO 45
      IF (ITERA . EQ . 0) WRITE (JOUT,1020) IT
1020 FORMAT (1H0,34X,42H THE GAUSS-SEIDEL ITERATION HAS CONVERGED,
1  6H AFTER, I5, 8H CYCLES.)
      RETURN
      45 IF (IT . LT . MAXIT) GO TO 15
      WRITE (JOUT,1030) IT,RELDIF,TOLER
1030 FORMAT (1H0,28H THE ITERATION HAS COMPLETED, I5, 5H THE ,
1  52H MAXIMUM NUMBER OF CYCLES WITHOUT CONVERGING, BUT HAS /

```

```

2      34H ACHIEVED A RELATIVE DIFFERENCE OF, E13.5,
3      33H AGAINST A SPECIFIED TOLERANCE OF, E13.5, 1H. /
4      43H THE UNCONVERGED RESULTS ARE WRITTEN BELOW.)
      WRITE (JOUT,1040) (N,THETA(N),N = 1,NNP)
1040  FORMAT (1H0,49X,33H THE CALCULATED NODE TEMPERATURES /
1      49X,33H-----// /
2      8(16H NODE THETA)/128(1H=) /
3      (8(I6,F10.1))
      STOP
      END
      SUBROUTINE SOURCE
      COMMON X(121),Y(121),NI(192),NJ(192),NK(192),NEL,NNP,NBP,GAP,
1      THETA(192),D(2,3),DT(3,2),KEL(3,3),LN(3),H(121),
2      JOUT,IDENT,Q(192)
      COMMON /KT(121,20),ANP(121,20),NAP(121)
C      ALL DIMENSIONS ARE IN INCHES
      ZI=360./39.37
      RI=462./39.37
C      CONST IS A FACTOR CONVERTING MEV/SEC TO WATTS
      CONST=1.6*(1.E-13)
C      POWER IS THE REACTOR THERMAL OUTPUT IN KILOWATTS
      POWER=10.E5
      DO 10 L = 1,NEL
      I = NI(L)
      J = NJ(L)
      K = NK(L)
      YBAR=(Y(I)+Y(J)+Y(K))/3.0
      XBAR=(X(I)+X(J)+X(K))/3.0
      SS=ATAN(ZI/RI)
      THETAN=ATAN(ABS(YBAR)/XBAR)
      IF (THETAN.LE.SS) R6=RI/COS(THETAN)
      IF (THETAN.GT.SS) R6=ZI/SIN(THETAN)
C      SG IS GAMMA SOURCE (MEV/(M2) (SEC)) ENTERING CONCRETE.
      SG=(5.2*(10.E8)*POWER)/(R6**2)
C      SN,SN1 ARE ENTERING FASTS SLOW NEUTRON INTENSITIES IN (N/(M2) (SEC))
      SN=SN1=(2.6*(10.E7)*POWER)/(R6**2)
      RR=SQRT(XBAR*XBAR+YBAR*YBAR)
C      THICK IS THE DISTANCE TRAVELLED BY RADIATION IN CONCRETE.
      THICK=(RR-R6)
      FASTE=(SN*8.0/0.25)*EXP(-THICK/0.13)
      SLOWE=(SN1*8.0/0.20)*EXP(-THICK/0.05)
      GAMAE=(SG/0.2)*EXP(-THICK/0.2)
C      Q IS HEAT GENERATED AT CENTROID OF ELEMENT IN WATTS/M3
      Q(L)=CONST*(FASTE+SLOWE+GAMAE)*4.0/(3.0)
10  CONTINUE
      RETURN
      END
      FUNCTION COND(TEMP,DOSE,CASE)
      INTEGER CASE
      COND=1.40
      RETURN
      END

```

A3.2 Program VESSEL (ASKA Processors)

CALL	START	Prepares all initial information required to start on ASKA job.
CALL	SA	Evaluates the topological description, and assemble all information required for data allocation in subsequent processors steps.
CALL	DATIN	Reads the input data.
CALL	SK	Calculates the element stiffness matrix k
CALL	BK	Assembles via the kinematic connection matrix a the element stiffness matrix into the global structural stiffness matrices K.
$K = a^t k a$		
CALL	BQ	Builds up the load matrix due to uniformly distributed load.
CALL	BR	Build up the complete load matrix BRL
CALL	SR	The structural stiffness matrix is triangulized by the Cholesky method. After then calculate the displacements caused by the nodal point forces (BRL).
CALL	SP	The displacements are rearranged into element-wise representation.
CALL	ST	Calculates the elemental stresses referred to the global cartesian co-ordinate system.
CALL	NPST	Calculates nodal point stresses and the average stress.
CALL	DATEX	Writes the required output.

CALL	REFBUK	Release from storage the specified matrix.
CALL	INIPL	Initialize the non-linear analysis.
CALL	EQUST	Find the equivalent stress from NPST.
CALL	ETAEL	The matrix ETAP of initial strains at nodal point is reassembled in element nodal point.
CALL	BJ	Calculates the elemental initial load matrices.
CALL	BRJ	The elemental nodal point forces due to initial load are added to BRL.
CALL	ALTLAB	Changes the matrix name.
CALL	INIIT	Initialization of the iteration loop.
CALL	RHS	Calculates total load increment matrix R .
CALL	STR	Calculates the stress increment and add it to the stress at the beginning of the iteration loop.
CALL	ELDACC	Calculates the thermal creep increment according to the procedure described in 8.3, in which, ETAC is the thermal creep at time T and ETACD is the thermal creep increment due to time increment IDELTA.

Note: For more details see Refs. (138, 139).

```

OVERLAY(MAIN,0,0)
PROGRAM VESSEL (INPUT,OUTPUT,TAPE5=INPUT,TAPE6)
COMMON/CREEP/T,TDELTA,RATIO
CXXXXX ELASTIC CALCULATION
CALL START(1,-1)
CALL SA
CALL DATIN(0,4HEOF )
CALL ELCO
CXXXXX CALCULATE ELEMENT STIFFNESS MATRIX
CALL SK
CALL BK
CXXXXX CALCULATE LOAD MATRIX
CALL BQ
CALL BR
CALL SR
CALL USR
CALL DATEX(0,4HUSR )
CALL SP
CALL ST
CALL NPST
CALL DATEX(0,4HNPST)
CALL REFDUK(4HEPDR)
CALL REFBUK(4HBRD )
T=1.0
TDELTA=.1
CALL INIPL
CALL DATIN(0,4HEOF )
C EQUIVALENT STRESSES
CALL EQUST
C XXXX THERMAL LOAD INCREMENT
J=0
DO 100 I=1,100
WRITE(6,900) I
900 FORMAT(//,5X,17HTHERMAL LOAD STEP,I3//)
IF (I .NE. 1) CALL REFBUK(4HBRD)
IF (I .EQ. 1) CALL ETAEL
IF (I .EQ. 1) CALL BJ
CALL BRJ
CALL ALTLAB(4HBRD ,4HBRD)
CXXXXX INITIALIZE CREEP CALCULATION
C INITIALIZE TIME, ESTIMATE FIRST TIME INCREMENT
75 J=J+1
C CALCULATE EQUIVALENT CREEP STRAIN INCREMENT
CALL INIIT(0,0,0)
C RIGHT-HAND SIDE
CALL RHS
C SOLUTION
CALL SOLT1
C STRESSES
CALL STR
CALL REFDUK(4HSIG )
CALL ALTLAB(4HSIG ,4HSIG )
C UPDATE EQUIVALENT STRESSES
CALL EQUST
RATIO=0.
CALL ELNACC
C UPDATE TIME
T=T+TDELTA
WRITE(6,150) J,T,TDELTA,RATIO
150 FORMAT(11H1CREEP STEP,I4/GH TIME=E11.4,17H, TIME INCREMENT=
1 E11.4,20H, MAX. STRESS RATIO=2PF5.2,4H 0/0//)
C UPDATE TIME INCREMENT
TDELTA=TDELTA/RATIO*.05
CALL ADDH(4HSRL ,4HSRLD,4HSRL ,1)

```

```

CALL REFBUK(4HNPST)
CALL NPST
IF ( J .EQ. 10) CALL DATEX(0,4HNPST)
IF ( J .EQ. 100) CALL DATEX(0,4HNPST)
CALL SET(4HPRNT,-1)
100 CONTINUE
IF ( T .LT. 10.) GO TO 75
STOP
END
SUBROUTINE ELDAC1(IG,IE,NOPSTR,SIGB,SIG,SIGMX,ETAP,ETAC,
1 ETAPD,ETACD)
C CALCULATE MAX. STRESS RATIO
C NOPSTR-NUMBER OF STRESS POINTS
C SIGB-EQUIVALENT STRESS AT BEGIN OF LOADING STEP
C SIG-CURRENT EQUIVALENT STRESS
REAL SIG(1),SIGB(1),ETAP(1),ETACD(1),ETAC(1)
COMMON/CREEP/T,TDELTA,RATIO
DO 20 I=1,NOPSTR
PHI=(ABS(.001-ETAP(I))*T/0.01)**(0.6)
IF ( ETAP(I) .LT. .001 ) ETAC(I)=0.1*(T**0.31)*SIGB(I)*(1.-PHI)*(
1 1.E-6)
IF ( ETAP(I) .LT. .001 ) ETACD(I)=.31*(SIGB(I)*(1.-PHI))**3.225*(
1 ETAC(I)**(-2.22))*TDELTA*.1
IF ( ETAP(I) .GE. .001 ) ETAC(I)=0.1*(T**0.31)*SIGB(I)*(PHI+1.)*(
1 1.E-6)
IF ( ETAP(I) .GT. .001 ) ETACD(I)=.31*(SIGB(I)*(1.+PHI))**3.225*(
1 ETAC(I)**(-2.22))*TDELTA*.1
20 CONTINUE
DO 10 I=1,NOPSTR
X=ABS(SIG(I)-SIGB(I))/SIGB(I)
IF (X.GT.RATIO) RATIO=X
10 CONTINUE
RETURN
END
SUBROUTINE FLECS(L)
L=0
RETURN
END
SUBROUTINE SPACE
C INCREASE CORE SPACE FOR HEXE27 SK
COMMON/ /TABLES(148),IPAGES(8000)
COMMON/DRSDSC/IWPGSZ,IWCOSZ,XX2(4)
COMMON/DRSBIX/KEGSIZ,XX3(131)
COMMON/DRSDES/LENDES,KLUNIT(10),XX4(20),KPAGES(10),XX5(20)
COMMON/DRSECR/XX6(2),NPAGEC,XX7(302)
C
IWPGSZ=2000
IWCOSZ=8000
C***** EXTENDED CORE
CALL FLECS(LECS)
NPAGEC=LECS/IWPGSZ
IF (NPAGEC.GT.100) NPAGEC=100
C***** BACKING STORE
KEGSIZ=3
LENDES=10
DO 22 I=1,LENDES
KPAGES(I)=500
222 CONTINUE
C
KPAGES(LENDES)=2100
RETURN
END

```

REFERENCES

- 1 Hannant, D.J. --- The Strain Behaviour of Concrete under Compressive Stress at Elevated Temperatures. C.E.R.L. Note No. RD/LIN 67/66.
- 2 York et al, G.P.--- Effect of Temperature and Moisture on Triaxial Creep. Concrete for Nuclear Reactors vol 11, ACI SP-34, Detroit. 1972.
- 3 Linse, D., Aschl, H. and Stökl, S. --- Concrete for PCRVS: Strength of Concrete under Triaxial Loading and Creep at Elevated Temperatures. 3rd International Conference on Structural Mechanics in Reactor Technology, London. Sept., 1975.
- 4 Arthanari, S. --- Influence of Temperature and Biaxial Stresses on Creep in Concrete. PH.D. thesis. University of London. 1966.
- 5 Ross, A.D. --- Concrete Creep Data. Institution of Structural Engineers. 1937. Vol.15, No.8; pp: 314-325.
- 6 Ross, A.D. --- Creep and Shrinkage in Plain, Reinforced and Prestressed Concrete. 1943. Proc. Institution of Civil Engineers, vol. 21, No. 1.
- 7 Lea, F.M. and Lee C.R. --- Shrinkage and Creep in Concrete. Symposium on shrinkage and cracking of Cementitious Materials.

1947. Proc. SOC. of Chemical Industry.
- 8 Sherbourne, A.N. --- Shrinkage and Creep in Reinforced Concrete Construction. November, 1961. Civil Engineering and Public Works. Review No. 56, 664.
 - 9 Neville, A.M. --- Creep and Shrinkage of Concrete. R.C.A. November, 1961.
 - 10 Ross, A.D. --- Creep of Concrete under Variable Stress. A.C.I. Proc. v. 54. 1958. pp 739-758.
 - 11 Neville, A.M. --- Properties of Concrete. Sir Isaac Pitman, London 1963.
 - 12 Neville, A.M. --- The Relationship between Creep of Concrete and the Stress-Strength Ratio. Applied Scientific Research. vol 9A. 1960. pp 285-292.
 - 13 Freyssinet, E. --- Relations Entre Les Déformations et la Constitution des Ciments et des Matériaux de Structure Colloïdale (1926-1929). Travaux Tome 11, May 1966, p. 884. Idées et voies nouvelles. Science et Industrie. No. 1, January 1933; see also Travaux, Tome 11, May 1966, p. 921.
 - 14 Lynam C.G. --- Growth and Movement in Portland Cement Concrete. London, Oxford University Press, 1934.
 - 15 Feldman, R.F. --- Mechanism of Creep of Hydrated Portland

- Cement Paste. Cem. Conc. Res., vol.2, No. 5, September 1972, p. 521.
- 16 Powers, T. C. ---Some Observations on the Interpretation of Creep Data. Mat. Const., Ess. Rech., Bull. RILEM, No. 33, Decmber 1966, p. 381.
- 17 Feldman, R.F. and Sereda, P.J. ---A Model for Hydrated Portland Cement Paste as Deduced from Sorption Length Change and Mechanical Properties. Mat. Const., Ess. Rech., Bull. RILEM, vol. 1, No. 6, November-December 1968, p. 509; vol. 2, No. 8, March-April 1969, p. 155.
- 18 Hansen, T.C. ---Creep and Stress Relaxation of Concrete. Swedish Cement and Concrete Institute, Stockholm, Proc. 31, 1960.
- 19 Neville, A.M. ---Creep of Concrete: Plain, Reinforced and Prestressed. North-Holland Publishing Company, Amsterdam, 1970. p. 261.
- 20 Coutinho, S.A. ---A Contribution to Mechanism of Concrete Creep. RILEM. vol. 10, No. 55, 1977.
- 21 Theuer, A.E. ---Effect of Temperature on the Stress-Deformation of Concrete. Journal of Research, National Bureau of Standards, vol. 18, No. 2, February 1937, pp. 195-204.

- 22 Ruetz, W. --- On the Deformation of Hardened Cement Pastes. RILEM Colloquium on the Influence of Time upon Strength and Deformation of Concrete, Munich, Nov 1958.
- 23 Serafim, J.L. and Guerreiro, M.Q. --- Influence of Temperature on the Creep of Mass Concrete. RILEM Colloquium on the Influence of Time upon Strength and Deformation of Concrete, Munich, Nov 1958; also RILEM Bulletin No. 6 (Réunion Internationale des Laboratoires d'essais et de Recherches sur les Matériaux et les Constructions), Mar 1960, pp 23-32.
- 24 Hansen, T.C. --- Creep and Stress Relaxation of Concrete, A Theoretical and Experimental Investigation. Proceedings NR 31, Swedish Cement and Concrete Research Institute, Royal Institute of Technology, Stockholm, 1960.
- 25 England, G.L. and Ross, A.D. --- Reinforced Concrete under Thermal Gradients, Magazine of Concrete Research, Cement and Concrete Association, vol 14, No. 40, Mar 1962, pp 5-12.
- 26 Nasser, K.W. and Neville, A.M. --- Creep of Concrete at Elevated Temperatures. Proceedings, American Concrete Institute, vol 62, 1965, pp 1567-1579.
- 27 Nasser, K.W. and Neville, A.M. --- Creep of Old Concrete at Normal and Elevated Temperatures. Proceedings, American Concrete Institute, vol 64, 1967, pp 97-103.

- 28 Peschel, G. --- The Viscosity of Thin Water Films Between Two Quartz Glass Plates. RILEM Colloquium on the Physical and chemical causes of Creep and Shrinkage of Concrete, Munich, Apr 1968; also Materials and Structures (RILEM), vol 1, No. 6, Dec 1968, pp 529-534.
- 29 Marechal, J.C. --- Causes Physiques et Chimiques du Fluage et du Retrait du Béton, RILEM Colloquium on the Physical and Chemical Causes of Creep and Shrinkage of Concrete, Munich, Apr 1968; also Materials and Structures (RILEM), vol 2, No. 8, Mar 1969, pp 111-116.
- 30 Marechal, J. C. --- Évolution des Propriétés Thermiques Mécaniques des Béton et autres Matériaux en Fonction de de la Température. Annales Travaux Publics vol 21 No. 246, June 1968, pp 852-855.
- 31 Philleo, R. --- Some Physical Properties of Concrete at High Temperatures. Proceedings, American Concrete Institute, vol 54, 1958, pp 857-864.
- 32 Thorne, C.P. --- Concrete Properties Relevant to Reactor Shield Behavior. Proc. American Concrete Institute, vol 57, 1961, pp 1491-1508.
- 33 Hickey, K.B. --- Creep, Strength, and Elasticity of Concrete at Elevated Temperatures. Report No. C-1257, Dec 1967, Bureau of Reclamation, Washington, D. C.

- 34 Arthanari, S. and Yu, C.W. --- Creep of Concrete under Uniaxial and Biaxial Stresses at Elevated Temperatures. Magazine of Concrete Research, Cement and Concrete Association, vol 19, No. 60, sept 1967, pp 149-156.
- 35 Da Silveira, A.F. and Florentino, C.A. --- Influence of Temperature on the Creep of Mass Concrete. American Concrete Institute Symposium on the Effect of Temperature on Concrete, Memphis, Nov 1968.
- 36 McHenry, D. --- A New Aspect of Creep in Concrete and Its Application to Design. Proc., American Society for Testing Materials, vol 43, 1943, pp 1069-1084.
- 37 Browne, R.D. and Blundell, R. --- The Influence of Loading Age and Temperature on the Long Term Creep Behavior of Concrete in a Sealed, Moisture Stable, State, RILEM Colloquium of Physical and Chemical Causes of Creep and Shrinkage of Concrete, Munich, Apr 1968; also Materials and Structures (RILEM), vol 2, No. 8, Mar 1969, pp 133-144.
- 38 Sullivan, P.J.E. --- The Structural Behavior of Concrete at Elevated Temperature. PH.D. thesis, University of London 1970.
- 39 Gross, H. --- Computer Aided Thermal Creep Analysis of Concrete Continua. PH.D. thesis, University of London, 1973.

- 40 Schneider, U. and Kordina. --- On the Behaviour of Normal Concrete under Steady State and Transient Temperature Conditions. 3rd International Conference on Structural Mechanics and Reactor Technology, Vol.3, Sept 1975, London.
- 41 Wang, H.C. --- Creep of Concrete at Elevated Temperatures. ACI, Special Publication SP-27, Detroit, 1972.
- 42 Gill, S. --- A Study of Viscoelastic and Thermal Behaviour of the Perforated End Slabs for P.C.P.V. Cylinders. Ph.D. thesis, City university, London. 1975.
- 43 Geymayer, G.H. --- Effect of Temperature on Creep of Concrete. A Literature Review. ACI, Special Publication SP-34, Detroit, 1972.
- 44 Troxell, G.E., Raphael, J.M., and Davis, R.E. --- Long-Term Creep and Shrinkage Tests of Plain and Reinforced Concrete. A.S.T.M. 1958, p. 91.
- 45 Neville, A.M. --- Creep as a Function of Cement Paste Content. M.C.R. vol. 16; No. 46. March 1964, pp 21-30.
- 46 Weil, G. --- Influence of Dimensions and Stress on the Shrinkage and Creep of Concrete. R.I.L.E.M. Bulletin No. 3 July 1959. pp 4-14.
- 47 Davis, R.E., Davis, H.E. and Brown, F.H. --- Plastic Flow and Volume Changes of Concrete. Proc. A.S.T.M. vol 38

part 11. 1938. pp. 410-417.

- 48 Neville, A.M. --- Tests on the Influence of the Properties of Cement on the Creep of Mortars. R.I.L.E.M. Colloquium Munich 1948.
- 49 Davis, R.E., and Davis, H.E. --- Flow of Concrete under Sustained Compressive Stress. ASTM. Proc. vol 30. part 11. 1930. p. 707.
- 50 Ross, A.D. --- Shape, Size and Shrinkage. C. C.E. vol 39. No 8. August 1944. pp 193-199.
- 51 Hansen, T.C. and Mattock, A.H. --- Influence of Size and Shape of Members on the Shrinkage and Creep of Concrete. P.C.A. Research and Development Labs. Bulletin D. 103.
- 52 Neville, A.M. --- Role of Cement in the Creep of Mortars. A.C.I. Proc. vol 54. 1958-59.
- 53 Roll, F. --- Long-Term Creep Recovery of Highly-Stressed Concrete Cylinders. Symposium on Creep of Concrete. A.C.I. Special Publication SP9. 1964.
- 54 Crispino, E. --- Studies in the Technology of Concretes under Thermal Conditions. ACI, Special Publication SP-34, Detoit, 1972.
- 55 Marechal, J.C. --- Variations in the Modulus of Elasticity

- and Poisson's Ratio with Temperature. ACI, Special Publication SP-34, Detroit, 1972.
- 56 Powers, T.C. and Brownnyard, T.L. --- Studies of the Physical Properties of Hardened Portland Cement Paste. Proc. ACI vol 43, 1947. Also Bulletin no. 22 of Research and Development laboratories of the Portland Cement Association, Chicago, 1948.
 - 57 Luikov, A.V. --- Heat and Mass Transfer in Capillary Porous Bodies. Pergamon Press. London. pp 191-195.
 - 58 Nielsen, K.E.C. --- Measurement of Water Vapour Pressure in Hardened Concrete. Bulletin no. 35 of Swedish Cement and Concrete Research Institute. Stockholm. 1967. pp 9 -15.
 - 59 Ishai, O. --- Drying Shrinkage Mechanism in Hardened Cement Paste and Mortar. Applied Materials Research vol 5, No. 3 July 1966. pp 154 - 161.
 - 60 Vos, B.H. and Tammes, E. --- Moisture and Moisture Transfer in Porous Materiald. Report No. BI-69-96, Institute TNO for Building Materials and Building Structures, Netherlands, 1969.
 - 61 Gilkey, H.J. --- Engineering News Record, vol 119, 1937. pp 630.
 - 62 Lea, F.M. and Desch, C.H. --- Chemistry of Cement and Con-

rete. Revised Edition, Edward Arnold Limited. London.
1956. pp 637.

- 63 Mills, R.H. --- Strength - Maturity Relationships for Concrete which is Allowed to Dry. Proceedings, Symposium on Concrete and Reinforced Concrete in Hot Countries. RILEM, Paris, 1960. Part 1, pp 1 - 28.
- 64 Neville, A.M. --- Properties of Concrete. Sir Isaac Pitman and Sons Ltd. London, Reprinted. 1968.
- 65 Hannant, D.C. --- Effect of Heat on Concrete Strength. Engineering (London), vol 197, No. 21, February 1964. pp 302.
- 66 Lankard, D.R., Birkimer, D.L. and Fondriest, F.F., and Snyder, M.J. --- Effects of Moistures Content on the Structural Properties of Portland Cement Concrete Exposed to Temperature up to 500 F. ACI Special Publication No. SP-25, Temperature and Concrete. 1971. pp 59 - 102.
- 67 Carlson, R.W. --- Drying Shrinkage of Concrete as Affected by Many Factors. Proc. A.S.T.M. vol 38, part 11. 1938. pp 419 - 437.
- 68 Davis, R.E., Carlson, R.W., Troxell, G.E. and Kelley, J.W. --- Cement Investigation for the Hoover Dam. ACI. Proc. vol 29. 1933. pp 275 - 284.

- 69 Davis, R.E. --- Summary of Results of Investigations into Volumetric Changes in Cement Mortars and Concretes due to Causes other than Stress. ACI. Proc. vol 26. 1930. pp 407 - 443.
- 70 Mears, A.R. --- The Effects of Aggregate Size and Grading on Initial Air-drying Shrinkage. C & C.A. Technical Report TRA 449. September 1970.
- 71 Johnson, R.P. --- Structural Concrete. McGraw - Hill 1967.
- 72 Schimmelwitz, P. and Hundt, J. --- A Note on the Moisture Condition of Concrete. ACI, Special Publication, SP 34, vol.2, Detroit, 1972.
- 73 England, L.G. and Ross, D.A. --- Shrinkage, Moisture and Pore Pressures in Heated Concrete. ACI, Special Publication SP 34, vol 2, Detroit, 1972.
- 74 Ross, A.D., Illston, J.M. and England, G.L. --- Short and Long-Term Deformations of Concrete as Influenced by Its Physical Structure and State. Int. Conference on the Structure of Concrete, London, 1965.
- 75 Sharp, T.J. --- The Influence of Elevated Temperature on the Physical Behaviour of Water in Concrete. Ph.D. thesis, University of London, August, 1971.
- 76 Parkinson, J.D. --- The Variation with Time of Water Content

and Shrinkage of Concrete Subjected to Thermal Gradients.
Ph.D. thesis, University of London, May, 1966.

- 77 England, G.L., and Sharp, T.J. --- Migration of Moisture and Pore Pressures in Heated Concrete. Proc. the First International Conference on Structural Mechanics in Reactor Technology. Paper H2/4, Berlin, 1971.
- 78 Hornby, I.W. --- Moisture Measurements in Mass Concrete. Conference on Prestressed Concrete Reactor Vessels, and their Thermal Insulation, Brussels, 1969.
- 79 McDonald, J.E. --- An Experimental Study of Moisture Migration in Concrete. ACI. Conference on Concrete for Nuclear Reactors. Paper SP 34-45, Berlin, 1970.
- 80 Hundt, J. and Schimmelwitz, P. --- Heat and Moisture Transfer in Concrete under the Effect of a Temperature Gradient. Proc. of the Second International Conference on Structural Mechanics in Reactor Technology, Berlin, 1973.
- 81 Chapman, A.D. and England, L.G. --- Effects of Moisture Migration on Shrinkage, Pore Pressure and Other Concrete Properties. Transactions of 4th Int. Conf. on Structural Mechanics in Reactor Technology, San Francisco, USA, Aug. 1977.

- 82 Davis, Raymond E. --- A Summary of the Results of Investigations Having to Do With Volumetric Changes in Cements, Mortars and Concretes and Concretes Due to Causes Other Than Stress. ACI Journal, Proceedings vol. 26, No. 4, Feb. 1930, pp. 407 - 443.
- 83 Mitchell, L.J. --- Thermal Expansion Tests on Aggregates, Neat Cement, and Concretes. Proc. ASTM, vol. 53, 1953, pp. 963 - 977.
- 84 Meyers, S.L. --- Thermal Expansion Characteristics of Hardened Cement Paste and of Concrete. Proc. Highway Research Board, vol. 30, 1950, pp. 193 - 203.
- 85 Bonnel, D.G.R., and Harper, F.C. --- Thermal Expansion of Concrete. Journal, Institution of Civil Engineers (London), vol. 33, No. 4, Feb. 1950. pp. 320 - 330.
- 86 Hatt, W.K. --- The Effect of Moisture on Concrete. Transactions, ASCE, vol. 89, 1926, pp. 270 - 370.
- 87 Walker, Stanton, Bloem, D.L., and Mullen, W.G. --- Effects of Temperature Changes on Concrete as Influenced by Aggregates. ACI Journal, Proc. vol. 48, No. 8, Apr. 1952, pp. 661 - 680.
- 88 Berwanger, Carl., --- The Modulus of Concrete and the Coefficient of Expansion of Concrete and Reinforced Concrete at Below Normal Temperatures. Temperature and Concrete,

- SP-25, American Concrete Institute, Detroit, 1971, pp. 191 - 233.
- 89 Emanuel, H.J. and Hulsey, L.J. --- Prediction of the Thermal Expansion of Concrete. ACI Journal, April 1977, pp. 149 - 155.
- 90 Philleo, R. --- Some Physical Properties of Concretes at High Temperatures. ACI Journal, Proceedings vol. 54, No. 10, Apr. 1958, pp. 857-864.
- 91 Malhotra, H.L. --- The Effect of Temperature on the Compressive Strength of Concrete. Magazine of Concrete Research (London), vol. 8, No. 23, Aug. 1956, pp. 63-68.
- 92 Sullivan, P.J., and Poucher, M.P. --- The Influence of Temperature on the Physical Properties of Concrete and Mortar in the Range 20C to 400C. Temperature and Concrete, SP-25, American Concrete Institute, Detroit, 1971, pp. 103-135.
- 93 Ali, I. and Kesler. --- Rheology of Concrete. A Review of Research. The Engineering Experiment Station, University of Illinois Bulletin vol. 62, No. 68, 1965.
- 94 Newman, B.J. --- Deformational Behaviour, Failure Mechanisms and Design Criteria for Concrete under Combinations of Stress. Ph.D. Thesis, University of London, 1973.

- 95 Mitzel, A. and Klapoc, M.J. ---Tests on the Non-Linear Creep of Concrete. Building Science, vol. 3, 1968, pp. 1-6.
- 96 Concrete Society. ---Main Clauses for Inclusion in a Standard for Concrete Cube Compression Testing Machines. 3rd Report of a Concrete Society Working Party, 1972, pp. 10.
- 97 L'Hermite, R. --- What Do We Know about the Plastic Deformation and Creep of Concrete? RILEM Bulletin No.1, March 1959.
- 98 Bakoss, S.L., Burfitt, A.J., and Cridland, L. ---A Low Cost Rig for Compression Creep Tests on Concrete. Civil Engineering Monograph CE 74/I ME. The New South Wales Institute of Technology, School of Civil Engineering, Sydney, 1974, pp. 14.
- 99 Tyler, R.G. ---Developments on the Measurement of Strain and Stress in Concrete Bridge Structures. Ministry of Transport, RRL Report LR 189, 1968.
- 100 Distefano, J.N. and Creus, C. ---Equipment and Instruments for Creep and Creep Buckling Tests of Concrete. RILEM Bulletin No.22, March 1964, pp. 61-63.
- 101 Illston, J.M. ---Load Cells for Concrete Creep Testing. Engineering, London, Vol. 195, No. 5054, 1963, pp. 318-319.

- 102 Newman, K. and Sigvaldason, O.T. ---Testing Machine and Specimen Characteristics and Their Effect on the Mode of Deformation, Failure and Strength of Materials. Imperial College, Civil Engineering Department Report; 1965.
- 103 Ghosh, R.S. and Timusk, J. ---Creep of Portland Cement Paste Sub-Zero Temperature. *Materiaux Et Constructions*. Vol. 4, No. 21, 1971
- 104 Parrott, L.J. ---An Improved Apparatus for Biaxial Loading of Concrete Specimens. *Journal of Strain Analysis*, Vol. 5, No. 3, 1970.
- 105 Davis, R.E. and Davis, H.E. ---Flow of Concrete under the action of sustained loads. *Journal of the American Concrete Institute. Proceedings*, Vol. 27, March 1931.
- 106 Shank, J.R. ---The Plastic Flow of Concrete. Bulletin No. 91, Engineering Experiment Station, Ohio State University, 1935, pp. 14.
- 107 Washa, G.W. and Fluck, P.G. ---Effect of Sustained Loading on Compressive Strength and Modulus of Elasticity of Concrete. *Journal of American Concrete Institute, Proceedings*, Vol. 46, May 1950, pp. 693-700.
- 108 Freudenthal, A.M. and Roll, F. ---Creep and Creep Recovery of Concrete under High Compressive Stress. *Journal of the*

- American Concrete Institute. Proceedings, Vol. 54, June 1958, pp. 1111-1142.
- 109 Hansen, J.A. ---A 10-Year Study of Creep Properties of Concrete. Concrete Laboratory Report No. SP-38, Engineering Laboratories Branch, Design and Construction Division, Reclamation, Denver, Colorado, 1953, pp. 14.
- 110 Best, C.M., Pirtz, D. and Polivka, M. ---A Loading System for Creep Studies of Concrete. ASTM Bulletin No. 224, September 1957, pp. 44-47.
- 111 Meyers, B.L. and Pauw, A. ---Apparatus and Instrumentation for Creep and Shrinkage Studies. Highway Research Record No 34, 1963, pp. 1-18.
- 112 Buettner, D.R. and Hollrah, R.L. ---Creep Recovery of Plain Concrete. Journal of the American Concrete Institute. Proceedings, Vol. 64, June 1968, pp. 452-461.
- 113 Gamble, B.R. ---Regulated Creep Loading System . Civil Engineering Department Reports, 1970, The University of Calgary, Canada, 1970, pp. 12.
- 114 Sekisambu, S. ---Apparatus for Triaxial Testing of Concrete at Elevated Temperature. MSc. Thesis, University of Wales. Swansea, 1975.

- 115 Sigvaldason, O.T. ---The Influence of the Testing Machine on the Compressive Strength of Concrete. Proceedings, Symposium on Concrete Quality, 1964. Cement and Concrete Association, 1965.
- 116 British Standards Institution. B.S. 1881:1952. Methods of Testing Concrete, London. 66 pp.
- 117 American Society for Testing and Materials. C.39-62. Test for Compressive Strength of Molded Concrete Cylinders.
- 118 Sigvaldason, O.T.---Spherical Seating Behaviour in Testing Machine. The Engineer.
- 119 Wright, P.J.F. ---Compression Testing Machines for Concrete. The Engineer, Vol. 203, No. 5283, 26th April 1957, ... pp. 639-641.
- 120 Sigvaldason, O.T. ---The Influence of Testing Machine Characteristics on the Cube and Cylinder Strength of Concrete. The Engineer,
- 121 Chilver, A.H. ---The instability of Testing Machines. Proceedings of the Institution of Mechanical Engineers, Vol. 169, No.25, 1955. pp. 407-418.
- 122 Flint, A.R. ---Discussion of Paper "The Instability of Testing Machines" by A.H. Chilver. Proceedings of the

- Institution of Mechanical Engineers, Vol. 169, No. 25, 1955. pp. 418-419.
- 123 Newman, K. and Lachance, L. ---The Testing of Brittle Materials under Uniform Uniaxial Compressive Stress. Proceedings, American Society for Testing and Materials, Vol. 65, 1965.
- 124 British Standards Institution. ---B.S. 1610:1964. Verification of Testing Machines, London.
- 125 Freudenthal, A.M. ---The Inelastic Behaviour of Engineering Materials and Structures, 1950, New York, John Wiley & Sons Inc. pp. 587.
- 126 Bertero, V.V., Bresler, B. and Polivka, M. ---Instrumentation and Techniques for Study of Concrete Properties at Elevated Temperatures. Concrete for Nuclear Reactors, SP-34, ACI, Detroit, 1972.
- 127 Schittenhelm, C. and Angeli, R. ---Strain Measurements at High Temperatures in a Concrete Structure Representing a Cylindrical Section of a PCRV with Hot Liner. 3rd International Conference on Structural Mechanics In Reactor Technology. Paper H 1/10, London, 1975.
- 128 Mayer, N. ---Ein Fluidisches System Zur Langzeitmessung Verschiedener Größen in Kerntechnischen Anlagen Bei Hohen Temperaturen. 2nd SMIRT, Paper H 4/11, Berlin 1973.

- 129 Zienkiewicz, O. C. ---The Finite Element Method in Engineering Science, McGraw-Hill, London, 1971.
- 130 Berg, P. N. ---'Calculus of Variations' in Handbook of Eng. Mechanics, Ch. 16, ed. N. Flügge, McGraw-Hill, 1962.
- 131 Argyris, J. H., Buck, K. E., Scharpf, D. W., William, K.J. ---Non-linear Methods of Structural Analysis, Nuclear Engineering and Design 19 (1972) 403 - 404. North-Holland Publishing Company.
- 132 Aoyagi, H., Abe, Ohnuma, H. ---Experimental Verification of Creep Analysis for Prestressed Concrete Reactor Vessels. Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology San Francisco, California, USA 15 - 19 Aug. 1977.
- 133 Eysink, B. J. ---Two Dimensional Heat Conduction Analysis of Nuclear Reactor Components Using the Finite Element Method. M.Sc. Thesis, University of London 1973.
- 134 Khory, G. --- Private Communications.
- 135 Tan, Ch. P. ---Prestressed Concrete in Nuclear Pressure Vessels. A Critical Review of Current Literature. Oak Ridge National Laboratory, ORNL-4227, May 1968.
- 136 Greenbaum, G. A. and Rubinstein, M. F. ---Creep Analysis

of Axisymmetric Bodies Using Finite Elements, Nuclear Engineering and Design 7 (1968) 588-590. North-Holland Publishing Comp., Amsterdam.

137 Cruz, C.R. ---Apparatus for Measuring Creep of Concrete at High Temperatures. J. PCA, pp. 36 et seq, (1968).

138 ASKA User's Reference Manual, ISD-Report No. 73, Stuttgart 1971.

139 ASKA III 1 - Material Non-Linearities, User's Reference Manual, ASKA UM 207, Stuttgart 1972.