LATERAL PRESSURES OF CONCRETE
ON FORMWORK

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CHAPTER I

INTRODUCTION

1.1 Statement of the Problem

Concrete has been the subject of much investigation for many years by engineers and constructors. One of its properties which has been particularly perplexing is its physical behavior between the time of mixing and the time the cement paste has obtained its final set. During this time the shear strength of the concrete matrix goes from practically zero to a substantial amount. Because of this low shear strength, the concrete is in a plastic or flow condition and must be contained in a rigid form to develop the desired shape. This container is called concrete formwork and the cost associated with it is usually the major item in the total cost of the concrete work. Formwork costs vary from thirty-five per cent to sixty per cent of this total cost depending on the degree of complexity. It is therefore evident that to minimize the total cost, the center of attention should be placed on formwork costs. In order to do this, it is imperative to understand the nature of the formwork requirements. According to the American Concrete Institute (1) these requirements are threefold:

1) Quality -- to design and build accurately so that the desired size, shape and finish of the cast concrete can be obtained.

2) Safety -- to build substantially so that it is capable of supporting all dead and live loads without collapse or danger to workmen and to the concrete structure.

3) Economy -- to build efficiently, saving time and money for the owner and builder alike.

In all three of these requirements, the load on the formwork looms as the one predominant factor. The surface quality and amount of deformation of the formwork depends on the load. Similarly, formwork safety also demands an understanding of the loads involved. The first two requirements could easily be satisfied by a conservative design. The third requirement — economy — however, dictates that the amount of formwork be minimized. To be conservative and overestimate the loads leads to overdesign in formwork which in turn is uneconomical. Note that on a project where the concrete portion amounts to $1,000,000 and the formwork is complex, a cost reduction of five per cent in the forming operation can lead to a $30,000 saving. It is therefore expedient to investigate the subject of concrete loads on formwork and to determine the factors which influence it. In wall and column formwork, this load is defined as lateral concrete pressure.

Concrete pressures have been the subject of numerous studies, most of them empirical. In all of the investigations the major problems encountered were caused by the multitude of factors which influenced the pressure. In

Peurifoy's book, "Formwork for Concrete Structures," (2) there are listed ten such factors:

1) Rate of placing the concrete
2) Temperature of the concrete
3) Proportion of the concrete mix
4) Consistency of the concrete mix
5) Method of consolidating the concrete
6) Impact during depositing
7) Size and shape of the formwork
8) Amount and distribution of the reinforcing steel
9) Weight of the concrete
10) Height of placement.

In addition to the above list, ACI Committee 622 (3) also considered the following factors to be influential:

1) Ambient air temperature
2) Smoothness and permeability of the formwork
3) Pore water pressure
4) Type of cement.

From these listings it is obvious that for any research study to be meaningful, it should be done in a controlled environment with a limited number of factors thoroughly analyzed.

1.2 Literature Review

As mentioned in the introduction, a large number of lateral concrete pressure studies have been performed. In almost all cases these were conducted in an empirical manner on actual construction sites.

The first reported pressure study was in 1894 when McCullough (4) measured pressures on a column form. He did this by placing a board on the side of the form and then placed the concrete in the form until the board broke. From this he concluded that the pressure was equivalent to a hydrostatic pressure with a unit weight of eighty pounds per cubic foot. Although the method of testing was very crude and the results inadequate, it did point out that the concrete pressure was something less than that of an ideal fluid with an assumed unit weight for concrete of one hundred and fifty pounds per cubic foot.

After this initial report there were many field studies performed of which those of Shunk (5), McDaniel (6), Smith (7), Teller (8), Roby (9), Stanton (10), Hoffman (11) and Macklin (12) are representative.

In 1952 Rodin (13) made a comprehensive investigation of all work which had been done up until then and summarized the main studies. He concluded that the previous investigators had attempted to find the pressure for particular construction conditions rather than investigate the general effect of different variables. He then attempted to give a rational explanation of the physical phenomena causing the types of pressure distribution found in practice. He also gave a formula for the maximum pressure;

$$P_{max} = 110\ Hm\ \text{(psf) for hand spaded concrete} \quad (1\text{-}1)$$

$$P_{max} = 150 \text{ Hm (psf) for internally vibrated concrete} \quad (1\text{-}2)$$

where

$$Hm = 3.6 \, R^{1/3} \text{ (ft)} \quad (1\text{-}3)$$

and

$$R = \text{rate of pour (ft/hr)}.$$

In his related discussion he touched on an important fact. The fact is concerned with the rate of increase in the shearing resistance of the concrete and its significance in determining the maximum pressures.

Since 1952 there have been a number of other studies performed. Schjodt (14) presented a theoretical formula using the earth pressure theories of Terzaghi. However, this assumes that full Rankine shear develops in the concrete which is not the case in typical formwork because of the restricted deformations. He also concluded that pore water pressures are of major significance.

In 1955 a subcommittee was formed by ACI Committee 622 to review all the previous pressure studies and to recommend a safe design formula for the pressure. This recommendation had to be generalized so that it could be readily adapted to everyday use. The variables considered to be most significant were rate of concrete placement, temperature of the concrete, and effect of vibration. The subcommittee used data assembled from tests made by several U. S. form manufacturers as their methods of investigation and the factors included were similar. The recommended formulas were:

<u>For walls with R less than or equal to 7 ft/hr</u> (1-4)

$$P = 150 + \frac{9000\ R}{T} \text{ or } 150\ h\text{, whichever is less}$$

<u>For walls with R greater than 7 ft/hr and less than 10 ft/hr</u>

$$P = 750 + \frac{2800\ R}{T} \text{ or } 150\ h \text{ or } 2000 \text{ psf, whichever is least} \quad (1\text{-}5)$$

<u>For columns</u>

$$P = 150 + \frac{9000\ R}{T} \text{ or } 150\ h \text{ or } 3000 \text{ psf, whichever is least} \quad (1\text{-}6)$$

where

P_{max} = maximum lateral pressure, psf

R = rate of placement, ft/hr

T = temperature of concrete mix, $^{\circ}$F

h = height of wall, ft.

Included in the coefficients of R/T is the effect of normal vibration. These equations assume the unit weight of concrete to be one hundred and fifty pounds per cubic foot and the slump less than four inches. The American Concrete Institute denotes any vertical formwork having a maximum horizontal distance of six feet as a column and all others as a wall.

Later, in an article by two of the committee members, Fleming and Wolf (15), the wall pressure formula for R greater than 7 ft/hr was revised to the following:

$$P_{max} = 150 + \frac{43400}{T} + \frac{2800\ R}{T} \text{ or } 150\ h \text{ or } 2000 \text{ psf, whichever is least.} \quad (1\text{-}7)$$

The studies of both Rodin and ACI pointed to the lack of reliable experimental data and the need for a more detailed and planned study. In response to this request numerous studies have been done in the past five years.

Ritchie (16, 17) conducted a number of tests in a controlled environment and concluded that fresh concrete develops a pressure distribution very close to that predicted by Rodin. He noted that the workability of the mix and the size and shape of restricted sections were of major significance. In another of his reports (18) he measured the workability of concrete by use of a triaxial test device. He varied the mix proportions and water-cement ratios and determined the cohesion and internal friction angle. In this testing the concrete was mixed and then tested as soon as feasible. Here the method of testing was an undrained or quick triaxial test. His sample was four inches in diameter and eight inches in height. His results showed that both the angle of internal friction and cohesion increased as the ratio of aggregate to cement increased. Also as the water-cement ratio increased the cohesion increased but the internal friction angle decreased. However, he does not show how these strength parameters are affected by different amounts of set time. This study will show that the shear strength to set time relationship is of major significance.

A study was performed by Jackson (19) to verify the formulas established by ACI. These tests were performed in a laboratory environment and the range of the variables tested was as follows;

$$9 \text{ ft/hr} \geq R \geq 2 \text{ ft/hr}$$

$$80°F \geq T \geq 40°F \; .$$

In all cases, with the exception of temperatures between 40°F and 60°F, the results agreed closely with the values obtained from ACI formulas. In the tests with the temperature below 60°F the ACI values were shown to be as much as one hundred per cent conservative.

In 1965, Hurley (20) made an experimental study and concluded that for very high rates of pour (greater than 25 ft/hr) the pressures on formwork approached a hydrostatic pressure with a unit weight of 150 lb/ft^3. Also in 1965, a French team of Adam, Bennasr and Santos Delgado (21) did an experimental investigation and their results showed that the factors considered by ACI and Ritchie were the most significant. They did, however, add that when small lifts are involved arching has little effect because of the vibration.

The Civil Engineering Research Association of London (CERA) (22), recognizing the lack of uniformity of previous tests and the limitations of the ACI work, began a very extensive experimental study in 1963 to develop a more inclusive formula. To measure the pressure they used a pressure balance which had been cross checked against two other gauges to guarantee accuracy. Using this gauge they collected data in a uniform manner from the formwork of two hundred contractors. The results showed that the pressures are limited by two factors — stiffening of the concrete and arching in the wall. The formulas recommended by CERA are as follows:

Using the least pressure given by Equations 1, 2, 3;

1) Stiffening Criteria

$$P = \frac{Rt}{1 + C\left[\frac{t}{t_{max}}\right]^4} + 12\,(8-R) + 200 \qquad (1\text{-}8)$$

2) Arching Criteria

$$P = 300 + 50d + 20R + 200 \qquad (1\text{-}9)$$

3) Equivalent Fluid Criteria

$$P = 150h + 200 \qquad (1\text{-}10)$$

where

P = pressure (psf)

= unit weight of concrete (pcf)

R = rate of placement (fph)

d = width of wall (inches)

t = time from commencement of placing (hours)

(In Eq. (1) if $t > t_{max}$ set $t = t_{max}$)

t^*_{max} = stiffening time of the concrete (hours)

C* = factor depending on the workability of the concrete and the continuity of vibration

h = height of the wall or column.

*t_{max} and C are found from charts which take into account the concrete temperature, concrete slump and amount

of vibration. In all three equations the 200 psf is added to allow for impact forces.

In the stiffening criteria the factor 12 (8-R) is a correction for reduced vibration influence at higher rates of pour. To simplify calculations the arching and stiffening criteria are combined in a nomograph which gives a concrete pressure under a set temperature and mix. To correct for any variations in the temperature and mix another graph was used. This corrected pressure was then compared with the pressure given by Equation 1-10, and the greater of the two used.

Interestingly, the results from both the formulas of Rodin and ACI were seen to fall between the results from the arching and stiffening criteria. To be specific, the ACI and CERA results agreed very closely for concrete placed in columns at low temperatures; however, ACI was more conservative for both high temperatures and thin walls when arching became a factor. CERA noted some important factors overlooked by earlier investigators. These include:

1) The stiffening action of the concrete is dependent on the chemical changes in the cement matrix and the degree of mechanical interlocking. The former is dependent on time, temperature, type and fineness of cement while the latter depends on the pressure, workability and history of vibration.

2) The age of the concrete before being placed influences the maximum pressure.

3) The effect of vibration is normally very localized and this may largely retard the development of arching action.

4) The effects of the formwork rigidity and the degree of relaxation of the form face

relative to the partially stiffened concrete would probably permit some further reduction in concrete pressures.

1.3 Scope of the Investigation

The scope of this investigation is threefold:

1) Establish the shear strength variation of concrete as a function of time.

2) Develop a method to relate the shear strength variation to the lateral pressure.

3) Compare the pressures resulting from this new method with pressures obtained from recognized pressure formulae (CERA and ACI).

Virtually all previous studies have been of an empirical nature; that is, based on prototype tests on actual formwork. Because of this very little emphasis has been placed on the mechanism behind the pressure, the shear strength of the concrete. To establish the shear strength it would first be necessary to find out if the Mohr-Coulomb Rupture theory was applicable. If the validity of this theory could be ascertained, then a new lateral pressure theory based on the shear strength could be established.

To evaluate the shear strength a triaxial testing device similar to that of Ritchie was used. Since the concrete is continually setting and increasing in shear strength, the time at which the test was performed was considered extremely significant. To isolate the effect of time all of other factors were either controlled or considered insignificant. Under these conditions, triaxial tests were run for different set times and the data analyzed on the computer. The computer gave both the concrete shear strength and associated strain for each set of test data. Using these results, together with the work of Terzaghi and Peck (23), a concrete pressure theory was postulated which considered shear strength the most significant factor.

In the comparison with ACI and CERA pressure formulas four factors will be correlated:

1) Rate of Concrete Pour
2) Width of Wall
3) Height of Wall
4) Time of Set between mixing and placing operations.

CHAPTER II

EXPERIMENTAL PROCEDURES AND DATA

2.1 Principles of the Triaxial Test

The method of triaxial testing was used to determine the shear strength of the concrete. This is a three dimensional compression test on a cylindrical specimen. The specimen, surrounded on all sides by an impervious membrane, is subjected to a confining pressure through the medium of air, water or oil (see Figure 1). An axial force is applied by the piston rod at a constant strain rate until failure is produced. Failure occurs when the load applied becomes constant or reduces, or the total amount of axial strain in the specimen reaches twenty per cent (ASTM (24)).

The pressure of the confining medium is taken as the minor principal stress and denoted as σ_3. The axial force divided by the cross-sectional area is considered to be the major principal stress and denoted as σ_1. When failure is reached, a critical Mohr stress circle is defined (see Figure 2). The diameter of this failure circle, $\sigma_1 - \sigma_3$, is defined as the deviator stress. Figure 3 shows a typical stress-strain relationship and deviator stress for a given test. If σ_3 is zero then the deviator stress is called the

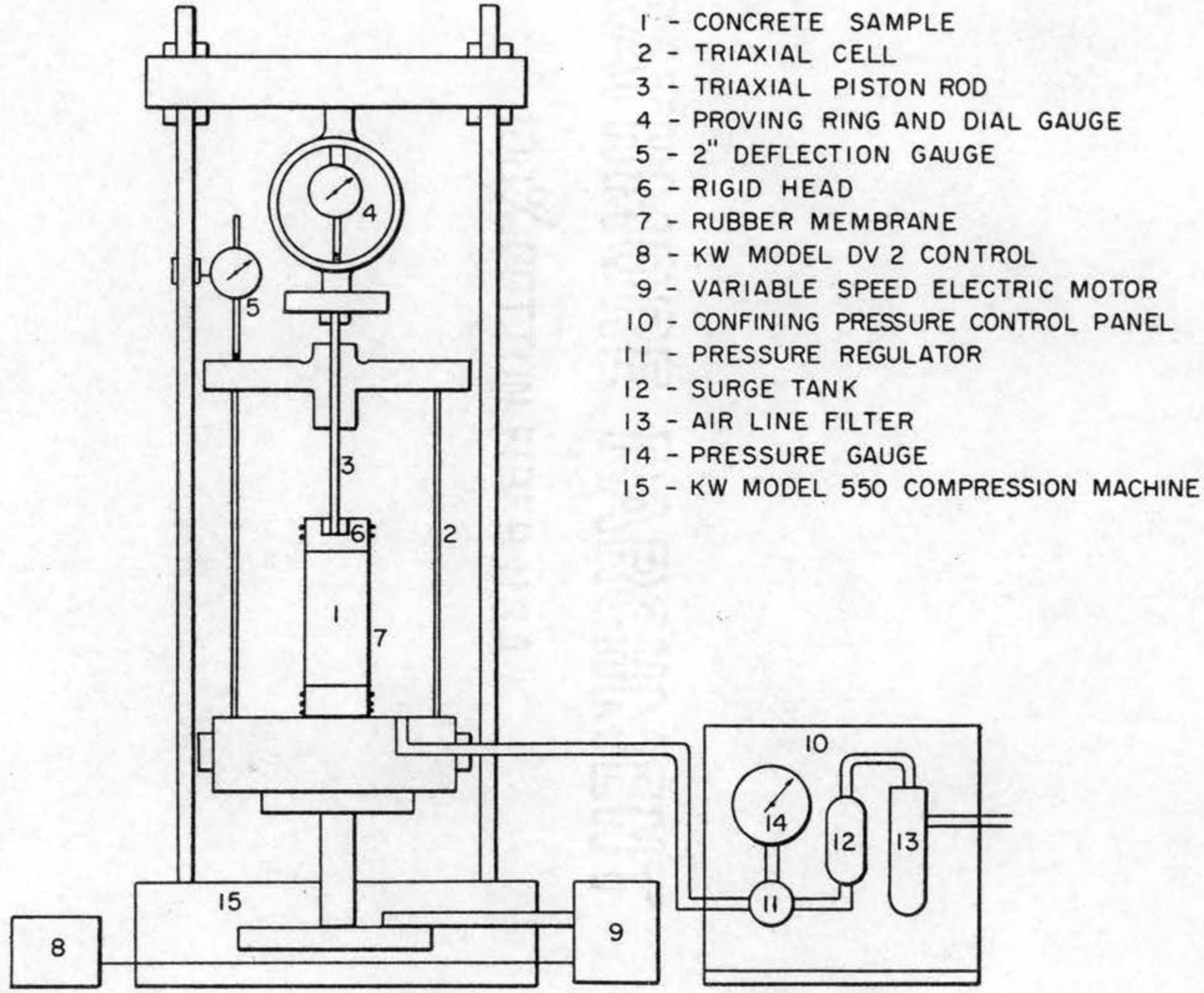

Figure 1. Schematic Diagram of the Triaxial Test

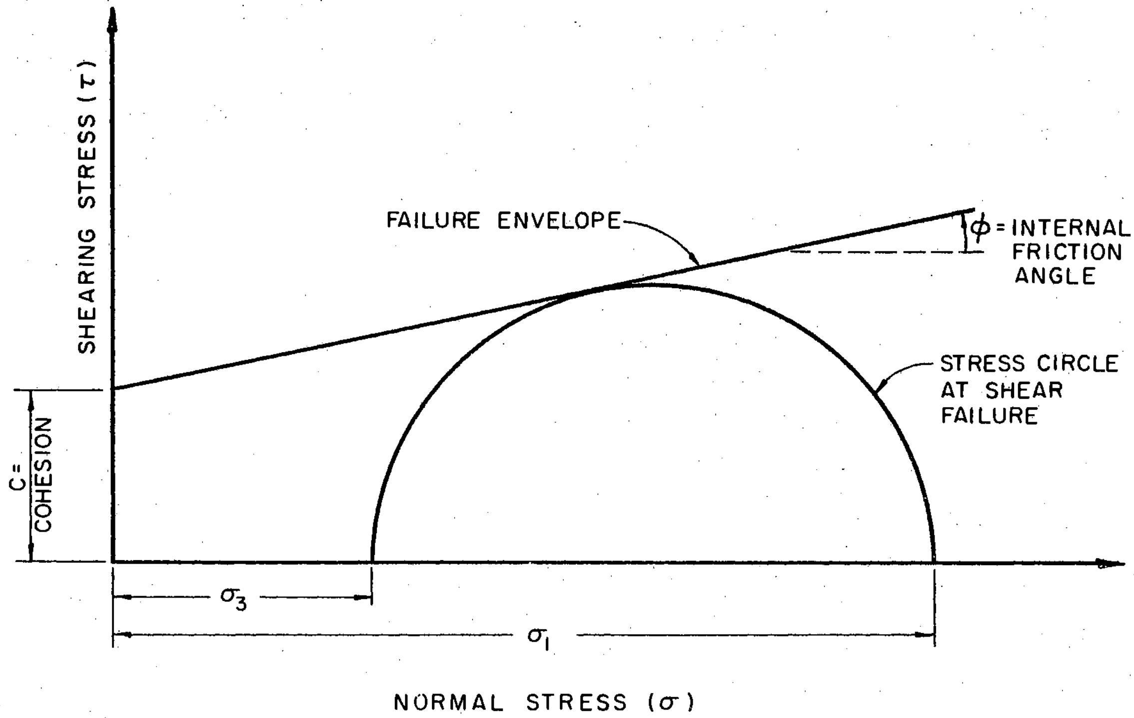

Figure 2. Typical Mohr Diagram with Failure Envelope

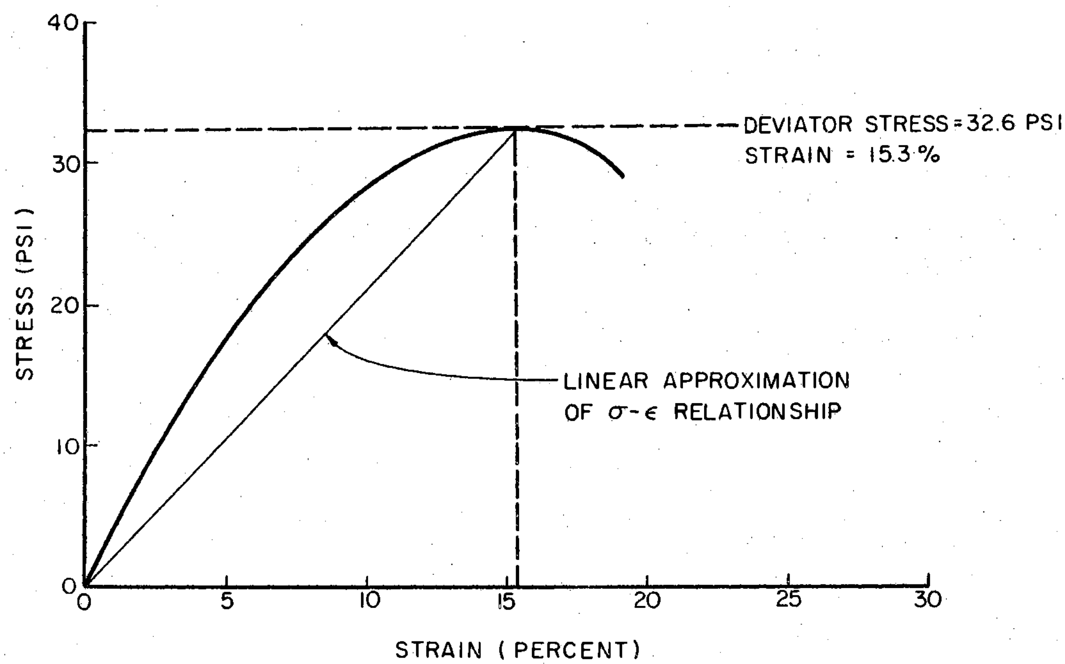

Figure 3. Typical Stress Versus Strain Relationship

unconfined compressive strength (q_u).

When a number of Mohr stress circles with different confining pressures are obtained, a tangent is drawn to all of them resulting in a Mohr failure envelope. The Mohr-Coulomb theory assumes the failure envelope to be a straight line.

There are three basic triaxial tests:

1) The Undrained Test permits no escape of moisture from the sample as it is compressed to failure. This test is also called the unconsolidated quick test.

2) The Consolidated-Undrained Test allows complete consolidation to occur in the sample before any axial load is applied through the piston. No drainage is permitted during the application of the axial force. This test is sometimes called the consolidated quick test.

3) The Drained Test is similar to the Consolidated-Undrained Test in that full consolidation first occurs under the chamber pressure. However, drainage is allowed during this stage as well as during testing. To do this the load is increased slowly so that no significant pore pressure is developed. This test is sometimes called the consolidated slow test.

A more detailed explanation of the triaxial test can be found in virtually all soil mechanics textbooks (Jumikis (25), Terzaghi (26)).

2.2 Triaxial Testing Procedure

Preparation of Sample.—One mix was used which had a ratio of cement to sand to coarse aggregate of 1.0:1.5:1.5 by weight with a water-cement ratio of 0.4. The cement used was Type I having an initial set time of 120 minutes using a Vicat test (27). The sand was saturated surface-dry river sand with a fineness modulus of 3.08. To insure uniformity in grading, the sand was separated by sieving and then combined to the gradation shown in Figure 4. The coarse aggregate was pea gravel with 100 per cent passing the 1/2 inch sieve and 50 per cent passing the 3/8 inch sieve. Distilled water was used. All materials were stored in a container at 77°F prior to mixing in order to guarantee temperature uniformity. The size of the concrete sample used was 2.8 inches in diameter and 6.0 ± 0.25 inches in height. Enough material for three samples was mixed dry in a porcelain pan and then water added and the time recorded. A rubber membrane of 0.025 inch thickness was placed in a compaction mold. The compaction molds used were the same as those normally used to prepare sand samples for triaxial testing. Then each of the three samples was placed in a mold in three layers with each layer receiving thirty strokes of a one-half inch tamping bar. A vacuum was applied to the rubber membrane while filling so that no air pockets occurred between the mold and the membrane. The head was then placed on the sample and the sample allowed to set until time of testing (see Figure 5). The time of set was taken to be the time from the addition of water to the time the testing began. It was felt that the time of testing (average seven minutes) was small compared with the amount of set time (average 75 minutes).

Description of Equipment and Procedure.—To facilitate the testing of the three samples three Lanzi triaxial cells

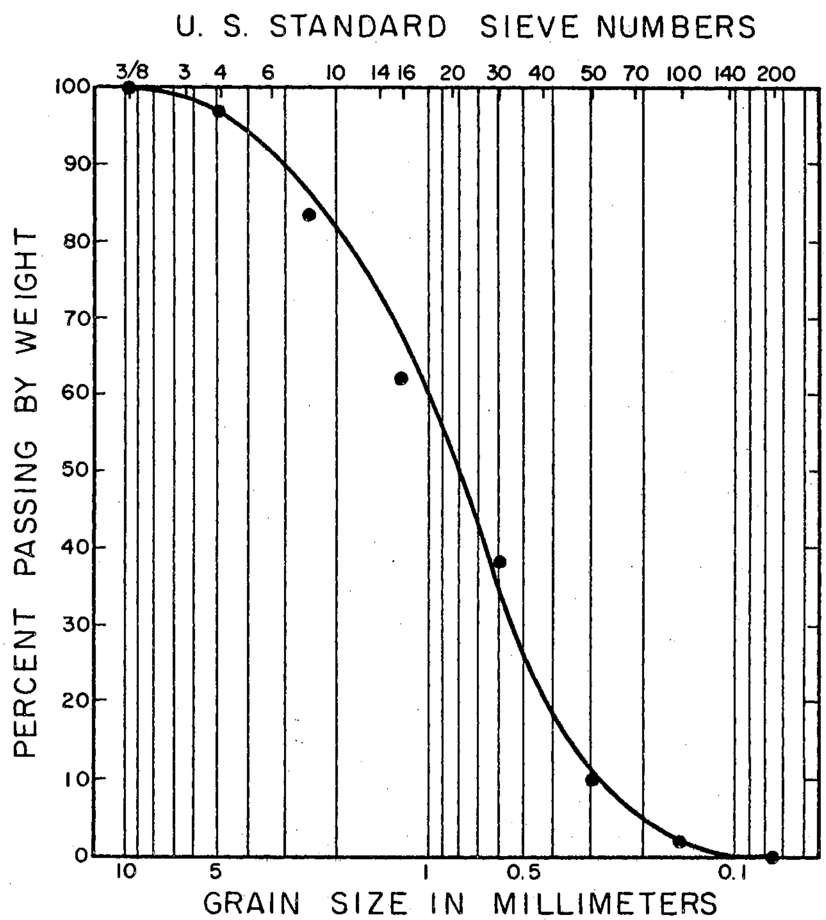

Figure 4. Sieve Analysis of the Sand

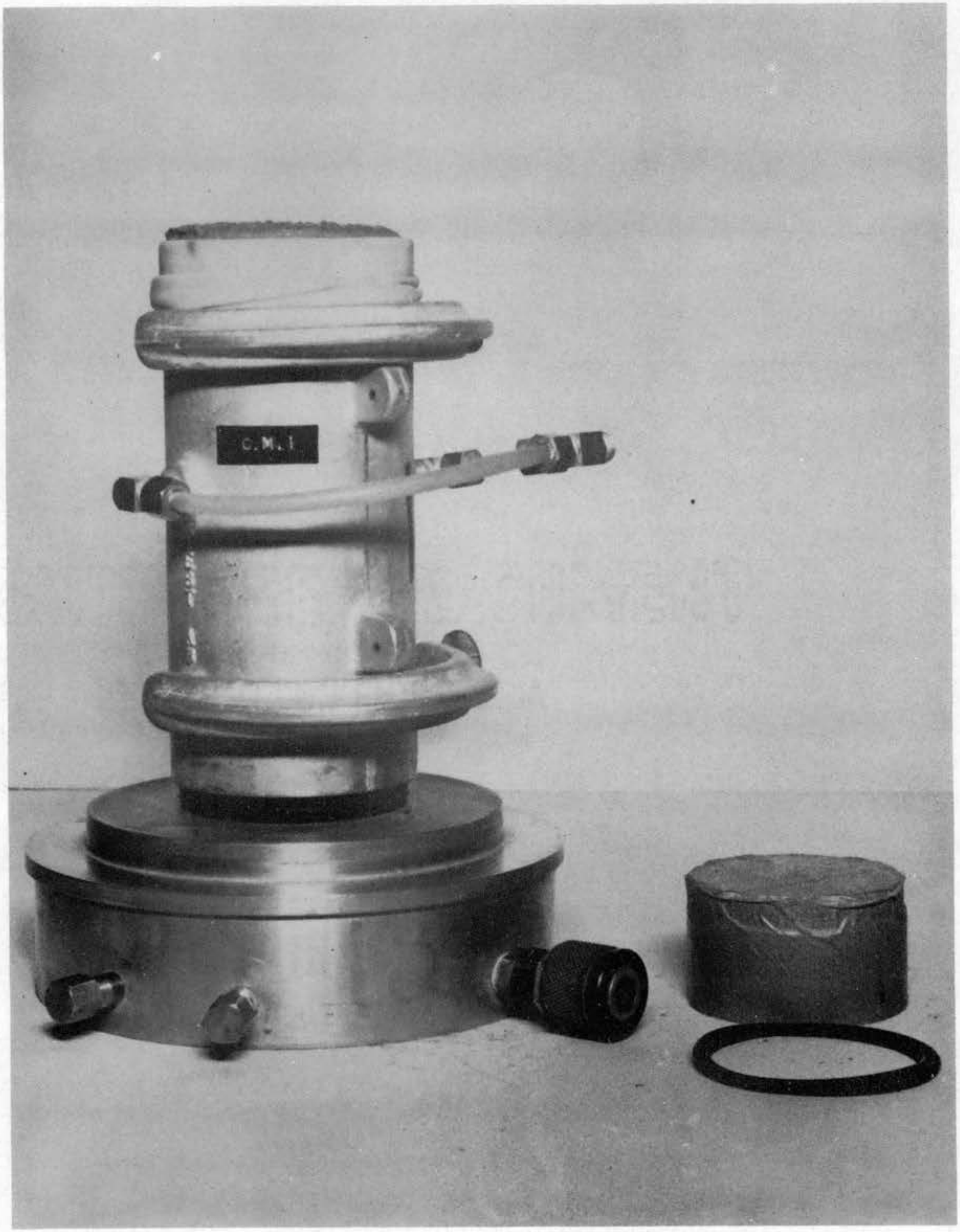

Figure 5. Concrete Sample While Setting

with bayonet connections were used (see Figure 6). Air was used as a confining medium and the exact pressure was regulated through a confining pressure control panel (see Figure 7). Compressed air at approximately one hundred and sixty psi was passed through a filter and into a surge tank to take care of input pressure irregularities. The pressure was then reduced using a commercial Norgen pressure regulator before connecting to the triaxial cell. The axial load was applied through a Karol Warner compression machine (model 550) using a KW model DV2 electric variable speed drive (1/6 hp A.C. motor). Ritchie (18) found that strain rates between two and four per cent per minute had very little effect on triaxial results. For this reason all tests were run at 2.5 per cent per minute. This control was possible through the use of calibration charts for the KW model DV2.

A proving ring with a capacity of five hundred pounds and sensitivity of one pound was used to measure the load. The amount of deformation was measured with a Soiltest LC-10 dial gauge that had a two inch total deflection with a sensitivity of 0.001 of an inch.

After the concrete had set the required length of time, the mold was taken off and the triaxial cell assembled. It was then placed in the compression machine and the proving ring was brought in contact with the piston rod of the cell. The confining pressure apparatus was then connected and adjusted to the desired pressure. Both the deformation and load gauges were set at zero. The test was then started and

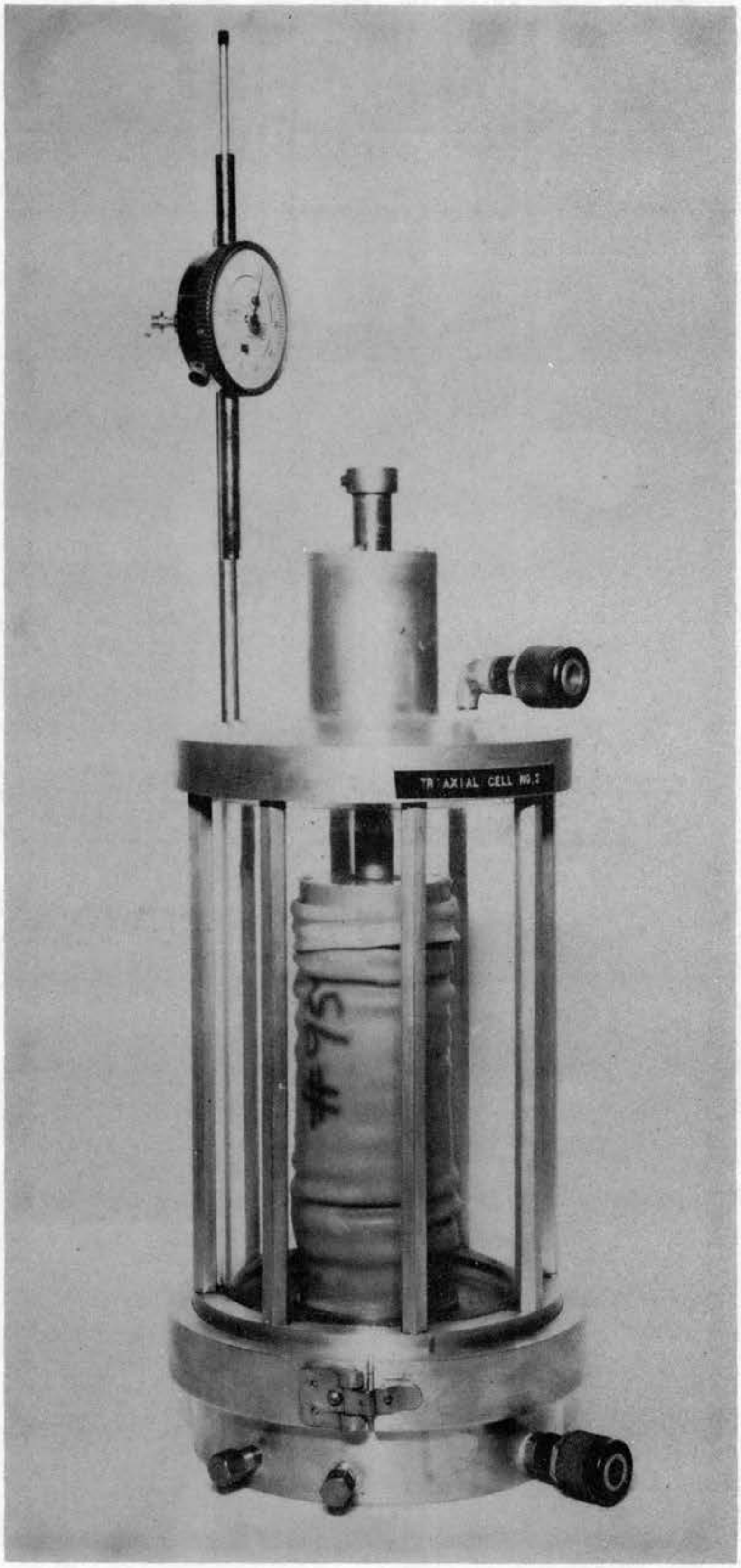

Figure 6. Triaxial Cell Before Testing

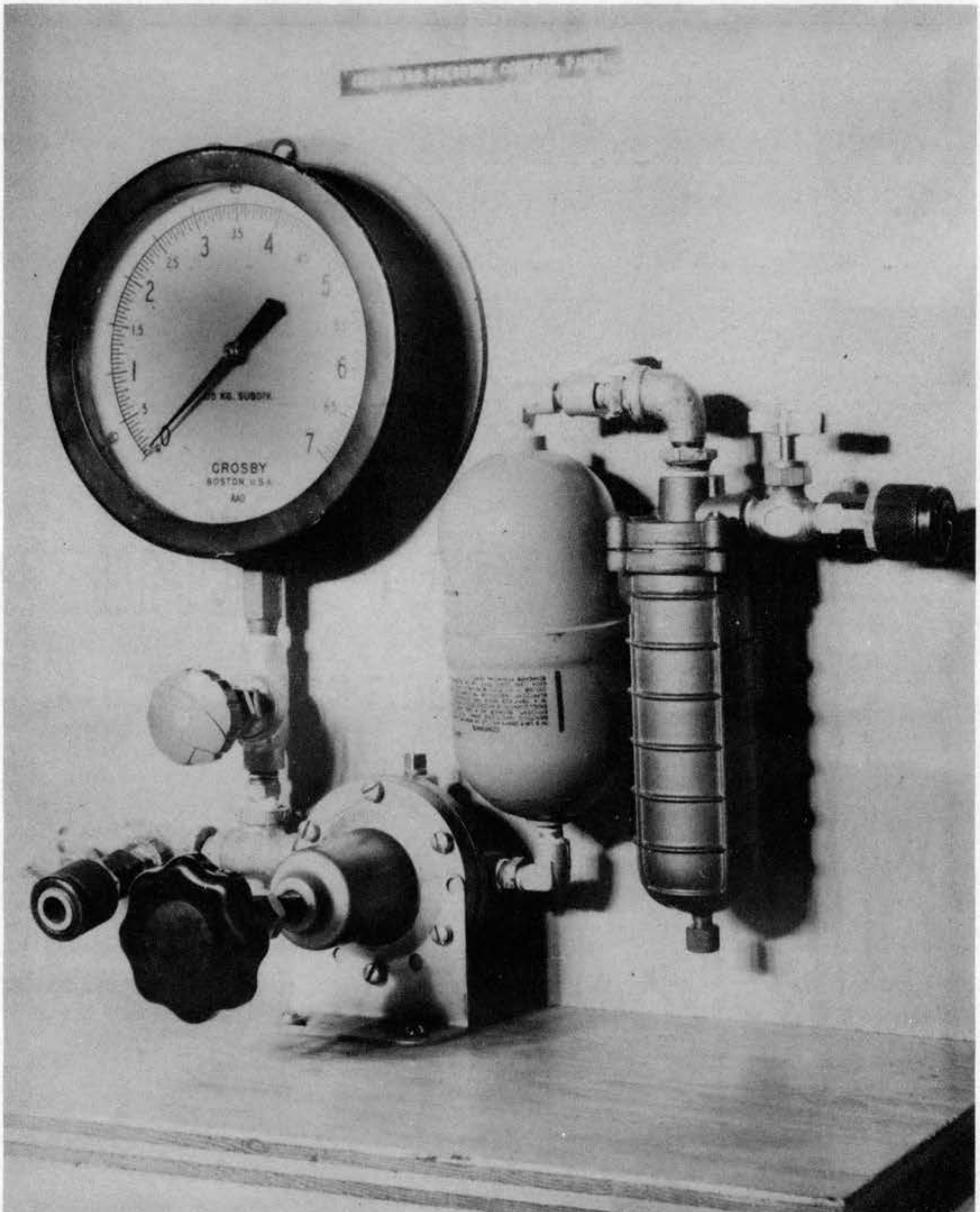

Figure 7. Confining Pressure Control Panel

was continued to failure with gauge readings taken every thirty seconds. Figure 8 shows this complete test setup.

Triaxial tests were performed for eight different set times, 20, 30, 45, 60, 75, 90, 120 and 180 minutes and for four different confining pressures, 20, 40, 60 and 80 psi. A minimum of five tests were performed for each confining pressure—set time (CP-ST) combination. In all, there were thirty-two CP-ST combinations. A typical data sheet is shown in Figure 9.

A normal distribution was assumed for the deviator stress results within each CP-ST combination. For each combination the average deviator stress, standard deviation and coefficient of variation were calculated as follows:

$$DS_{ave} = \frac{\sum_{i=1}^{n} DS_i}{n}$$

$$s_{DS} = \sqrt{\frac{\sum_{i=1}^{n} (DS_i - DS_{ave})^2}{n - 1}}$$

$$c_{DS} = \frac{s_{DS}}{DS_{ave}} \times 100$$

where

DS_{ave} = average deviator stress (psi)

DS_i = individual deviator stress for each test (psi)

n = number of deviator stress within the combination (-)

s_{DS} = standard deviation of the deviator stress within the combination (psi)

c_{DS} = coefficient of variation (per cent).

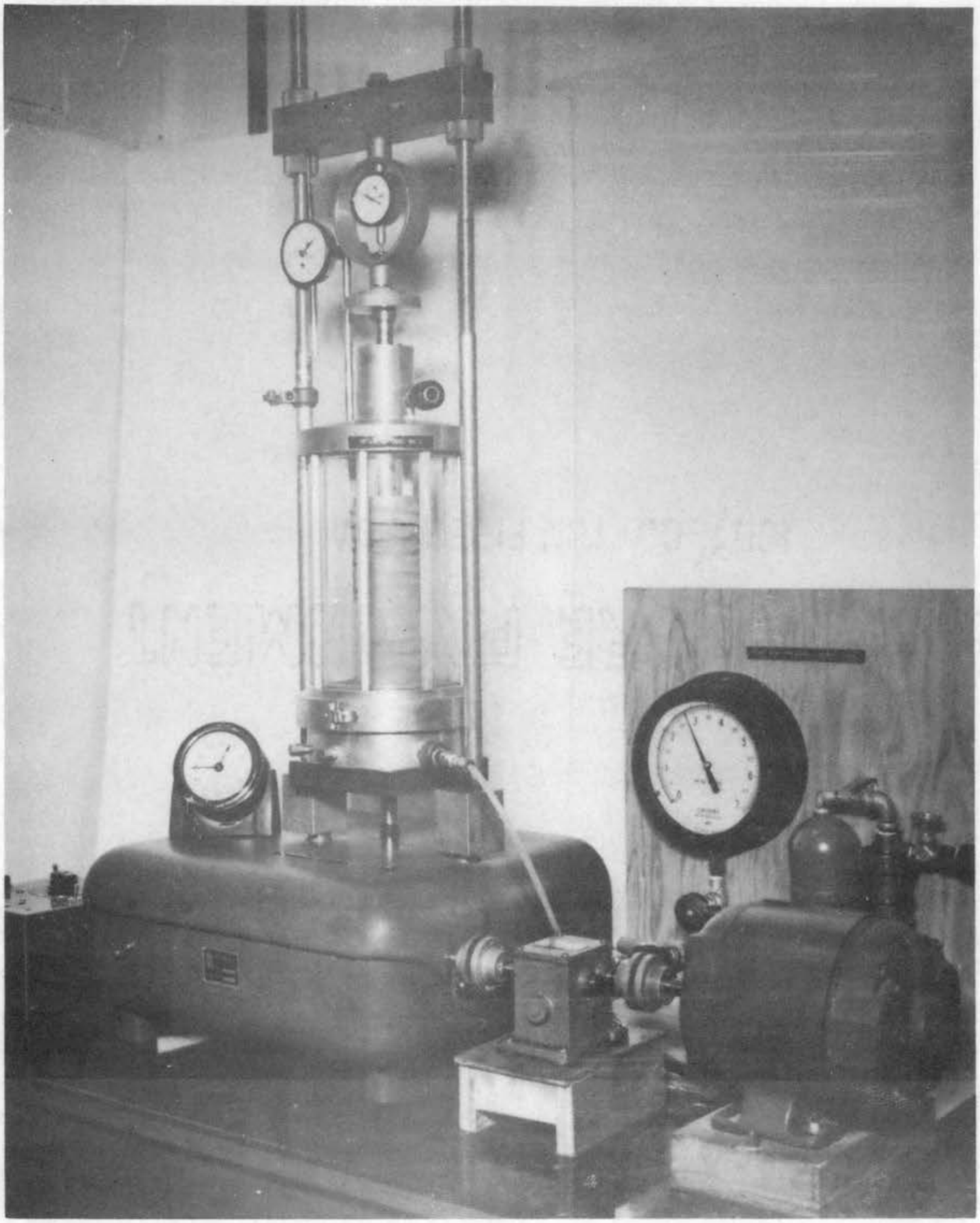

Figure 8. Triaxial Testing Apparatus

Day 2/7/67

Test 10

Mix 3

Sand Grad.# 4

Chamb. Press. 20 (1.41) psi

Wght. Empty 13.67 lbs.

Wght. Full 16.76 lbs.

Wght. Sample 3.09 lbs.

Height Sample 5.938 in.

Membrane # 52

Triaxial Cell # 1-1

Proving Ring # 1

Time of Mix 4 49

Time of Test 6 04

Time of Set 75 min.

Conc. Temp. 73 °F

AIR TEMP (AM) — 77°F
WATER TEMP (AM) — 71°F
OVEN TEMP (AM) — 76°F
AIR TEMP (AT) — 77°F

Concrete Lateral Pressure Study

Principal Investigator: Roy Olsen

Thesis Advisor: Prof. E. L. Bidwell

Faculty Advisor: Prof. R. L. Janes

Time	Deflect. Read.	Proving Ring Read.
Min.	0.001"	0.0001"
0.0	100	300
0.5	178	310
1.0	261	318
1.5	339	327
2.0	419	334
2.5	490	341
3.0	555	349
3.5	632	355
4.0	709	363
4.5	785	369
5.0	860	376
5.5	947	384
6.0	1011	390
6.5	1087	397
7.0	1157	403
7.5	1234	404
8.0	1309	395
8.5	1385	384
9.0		
9.5		
10.0		

Remarks APPEARED ON WET SIDE

Figure 9. Typical Data Sheet

The total number of tests performed within each combination was based upon the coefficient of variation. When the coefficient of variation was approximately one third or less then the results for that combination were considered satisfactory.

Occasionally individual tests within a particular CP-ST combination gave results which were significantly different. This condition, when present in a small sample size, adversely affected the average deviator stress and standard deviation. To alleviate this effect all tests which were more than two standard deviations from the average deviator stress were eliminated. In actuality this eliminates the two per cent fringes on the normal distribution curve (see Figure 10). The average deviator stress and standard deviation were then recalculated with the remaining tests of that particular combination.

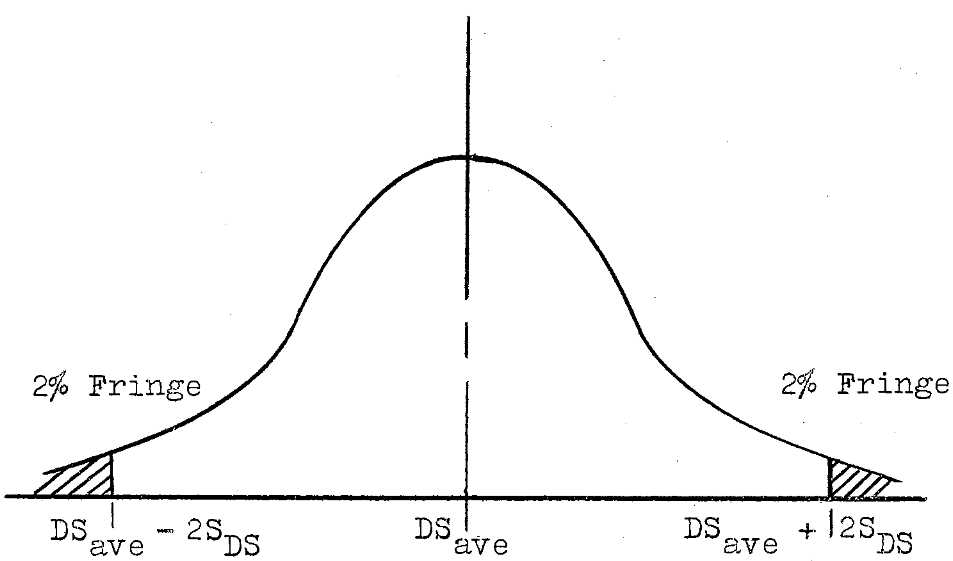

Figure 10. Normal Distribution of Deviator Stress

Presentation of Data.—Two hundred and ninety-seven triaxial tests were performed and each was given an individual number. The data sheets were arranged so that they could easily be transferred to computer cards and analyzed on the IBM 1620 computer. The analysis for each test was performed on two computer programs designated I and II. Flow charts for these programs are given in Appendix C.

Program I was used to find the maximum deviator stress and associated strain for each test. The deformation of the concrete during the test necessitated a correction for the cross-sectional area. This was done by assuming a uniform area and relating it to the height and axial strain. The resulting formula is:

$$A_{corr} = \frac{V_{orig}}{h(1 - \epsilon)}$$

where

A_{corr} = corrected area

V_{orig} = original sample volume

h = average height of the individual sample

ϵ = axial strain.

The deviator stress was calculated by dividing the axial load at failure by the corrected area. The average strain rate was also computed in this program to see if it was reasonably close to 2.5 per cent per minute. A summary of the results for each test is given in Appendix A together with additional information recorded during the test. A plot is shown of all the deviator stresses for each confining pressure on Figures 11, 12, 13 and 14. Also in Table I a summary of all the deviator stresses is given.

A plot was made of every stress-strain relationship on the IBM 565 Calcomp plotter by writing another program (Program II). All of the tests of a particular CP-ST combination were put on one graph sheet using the same data cards as in Program I. Eight representative plots are shown and discussed in Appendix B. There is one plot for each set time with different confining pressures.

The statistical parameters (DS_{ave}, s_{DS} and c_{DS}) were calculated for each combination in Program III. As noted in a footnote for Table I there were thirteen tests which were eliminated because of the two standard deviation criteria. This represented four per cent of the total number of tests. In addition to this, four tests were omitted because of improper technique representing one per cent of the total number of tests. In all five per cent of the total number of tests were rejected. After this had been determined, the average deviator stress, standard deviation and coefficient of variation were then recalculated with the remaining tests. These results are summarized together with the average associated strain in Table II. This table also gives the number of tests after the elimination that were used in each of the combinations. Notice that the coefficient of variation is slightly higher than one third for the CP(20 psi) - ST(60 min) and CP(40 psi) - ST(60 min) combinations. However, since they were both just

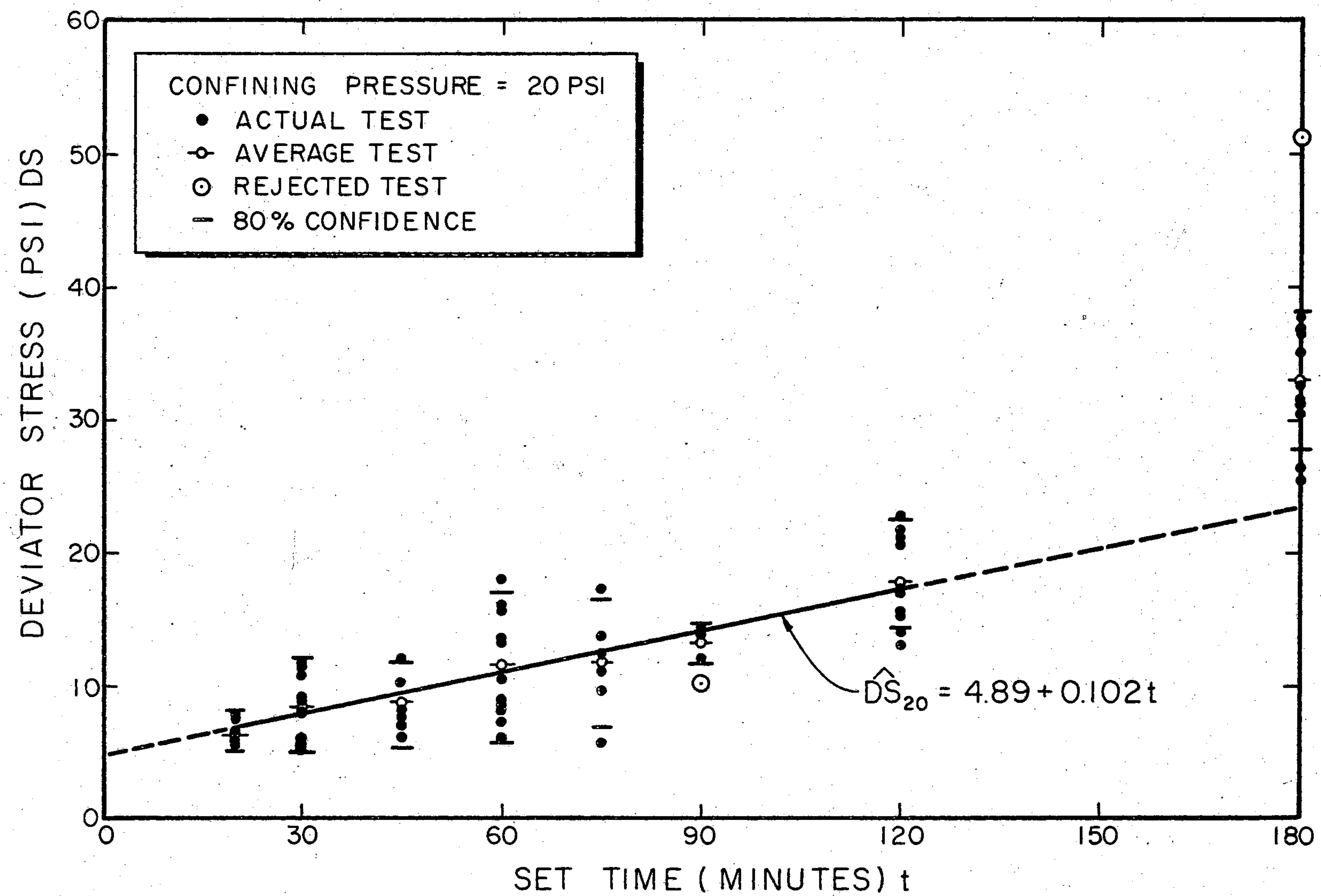

Figure 11. Deviator Stress Versus Set Time (CP = 20 PSI)

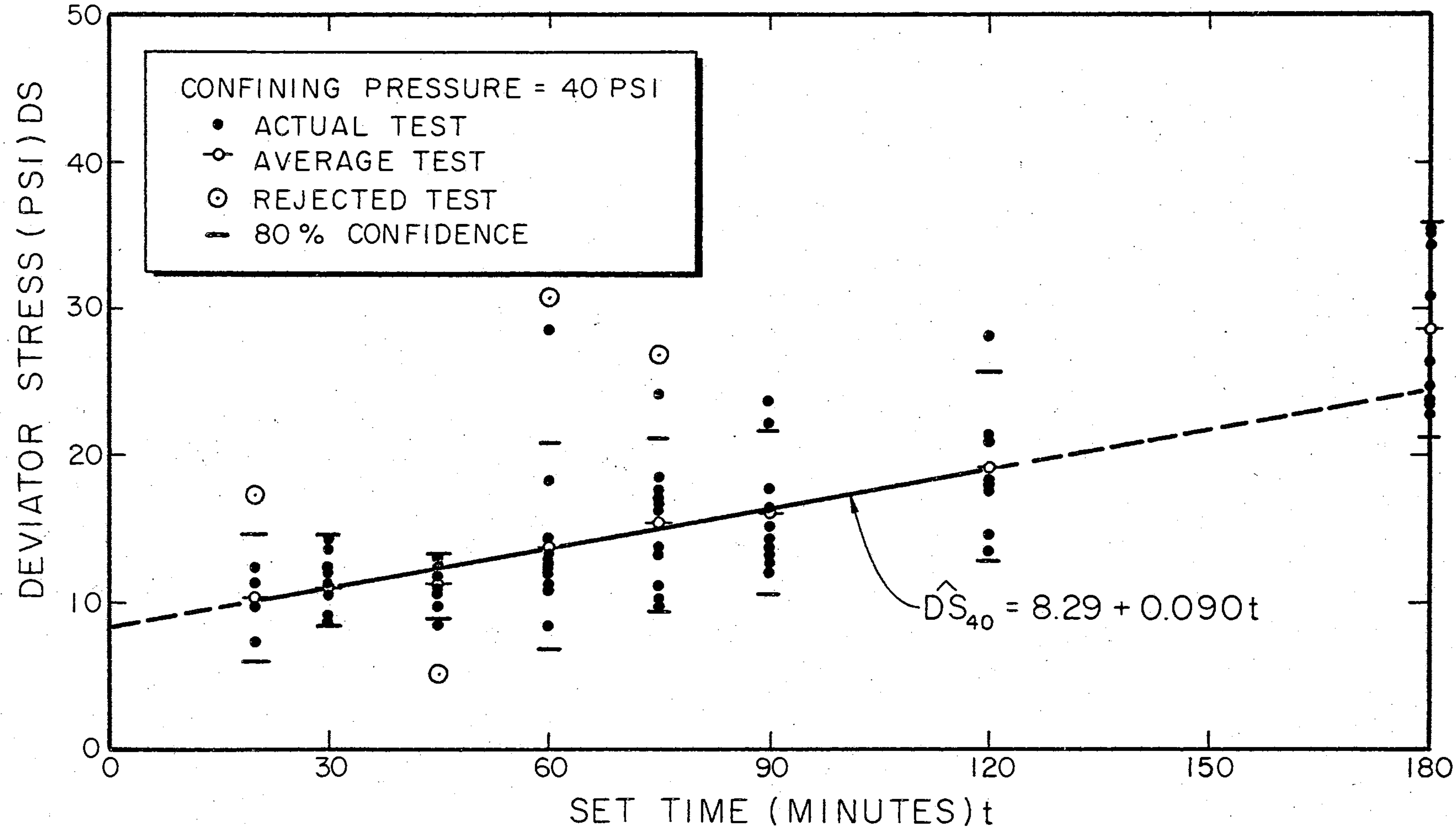

Figure 12. Deviator Stress Versus Set Time (CP = 40 PSI)

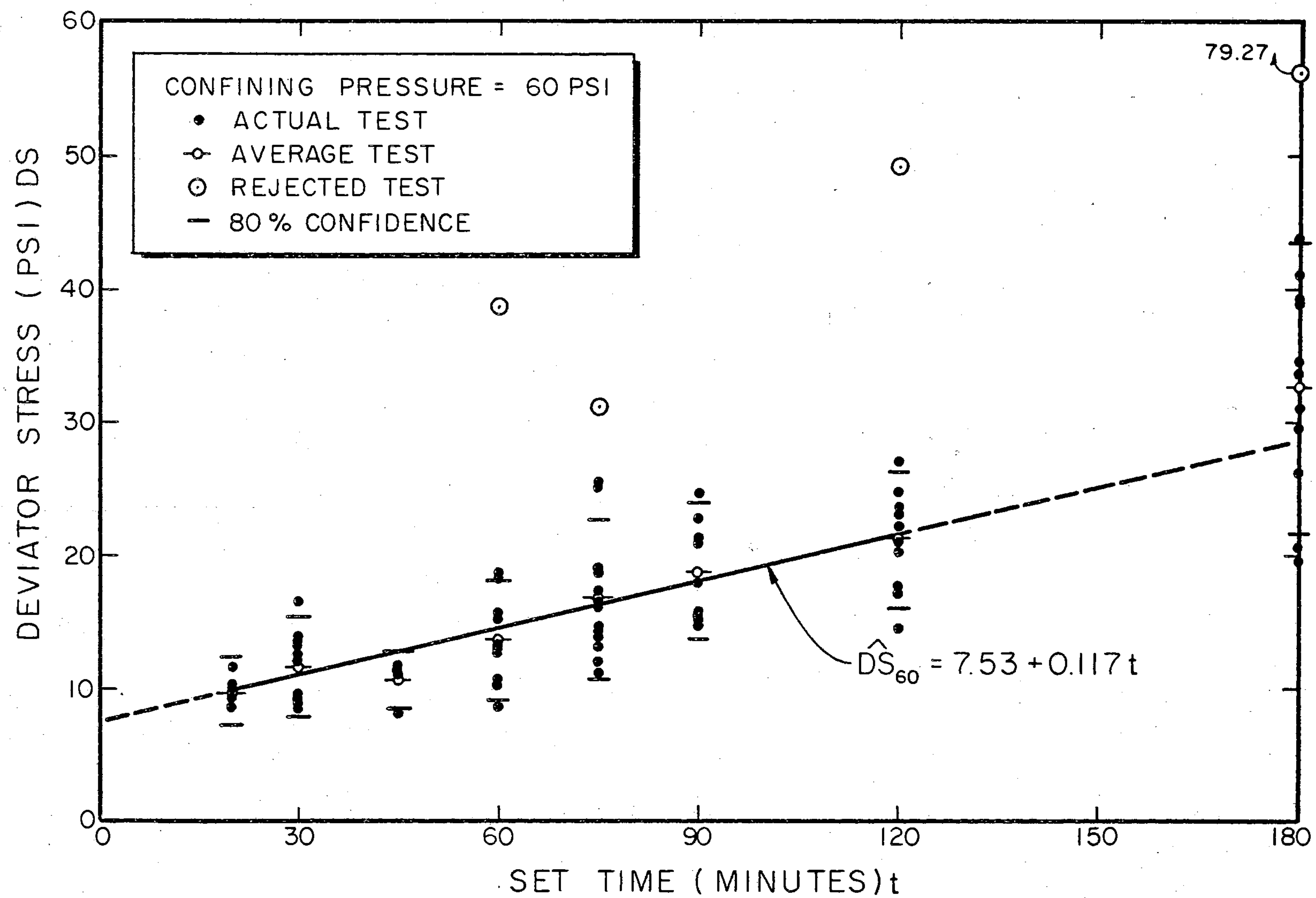

Figure 13. Deviator Stress Versus Set Time (CP = 60 PSI)

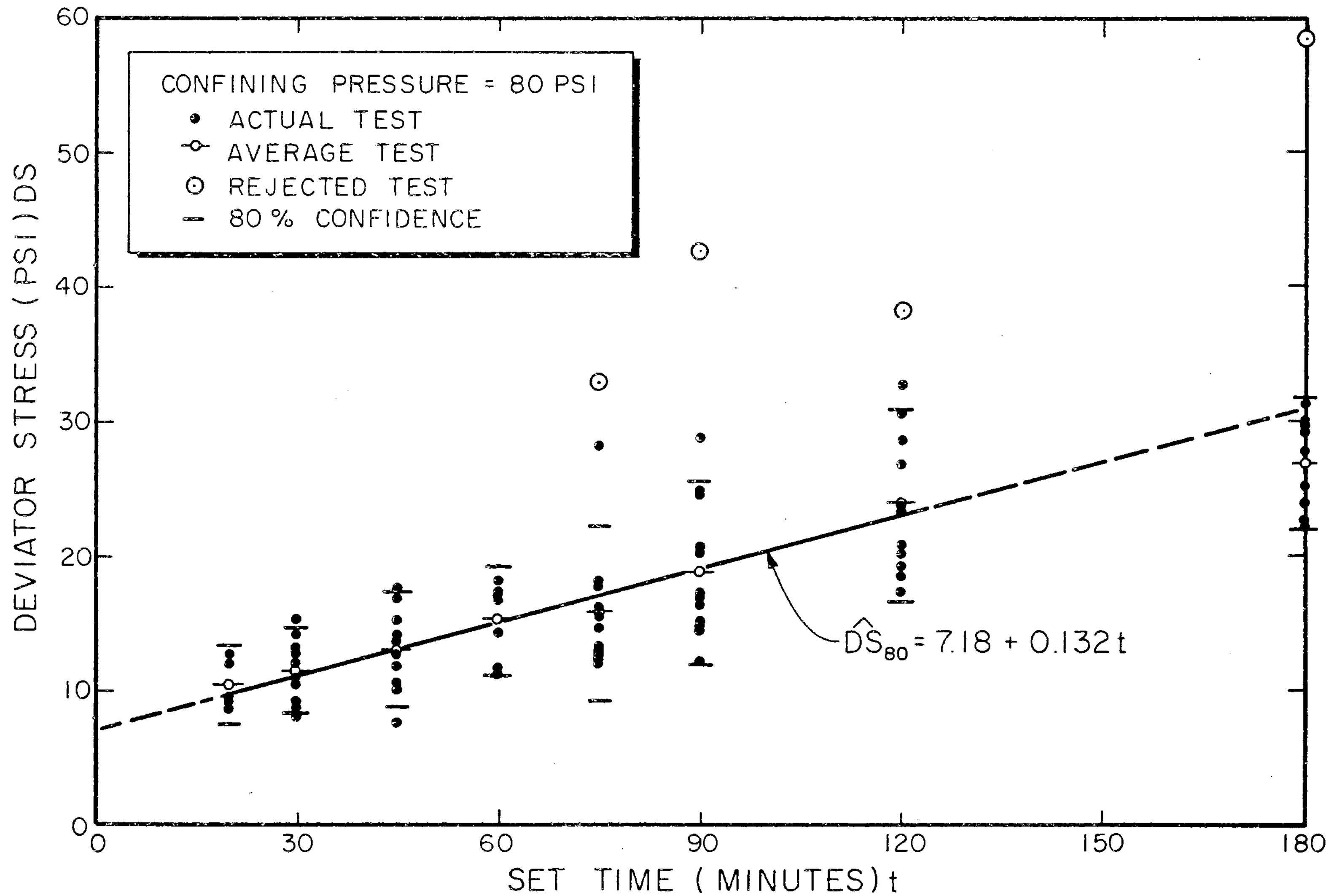

Figure 14. Deviator Stress Versus Set Time (CP = 80 PSI)

TABLE I

SUMMARY OF THE DEVIATOR STRESSES RESULTS

SET TIME OF CONCRETE (MINUTES)															
20		30		45		60		75		90		120		180	
Dev. Stress	Test No.	Dev. Stress	Test No.	Dev. Stress	Test No.	Dev. Stress	Test No.	Dev. Stress	Test No.	Dev. Stress	Test No.	Dev. Stress	Test No.	Dev. Stress	Test No.
CONFINING PRESSURE = 20 PSI															
5.73	235	5.48	138	6.25	139	6.25	180	5.72	33	10.09	34	13.23	35	25.58	187
6.09	245	5.96	236	7.07	192	7.48	183	9.84	184	12.24	65	14.01	285	30.08	141
6.77	286	5.97	246	7.81	32	8.39	140	11.18	288	12.52	185	15.50	297	30.27	63
7.54	262	8.44	31	8.28	62	8.58	153	11.62	156	13.20	20	15.66	21	31.48	66
7.92	283	8.94	263	10.49	71	8.82	287	12.21	181	13.61	5	17.09	6	31.59	122
		9.17	70	12.22	101	10.69	264	12.42	154	14.14	157	17.14	271	32.83	220
		10.96	100			13.39	229	13.99	64			20.67	158	35.16	273
		11.53	94			13.48	293	17.44	102			21.13	186	36.52	36
		11.78	61			15.96	19					21.88	121	36.97	117
						16.10	4					22.84	182	37.90	272
						18.09	72							51.43	168
CONFINING PRESSURE = 40 PSI															
		8.67	227	5.06	136	8.41	137	9.95	193	12.10	149	13.59	47	22.81	95
7.39	243	8.71	244	8.52	77	10.99	165	10.09	254	12.75	201	14.70	191	23.53	96
9.88	274	10.94	135	9.87	50	11.07	28	11.06	148	13.27	2	17.60	3	23.67	97
11.34	289	11.33	238	10.69	199	11.74	253	13.39	276	13.58	255	18.20	194	24.75	116
12.32	237	12.25	49	10.81	38	12.28	275	13.61	109	14.47	284	18.43	24	26.38	48
17.23	250	12.38	76	11.94	208	12.55	290	16.63	222	15.15	46	21.21	202	30.89	131
		13.61	82	12.65	251	12.63	200	16.80	218	16.28	23	21.84	126	34.22	174
		14.49	37	13.04	83	12.65	221	17.07	124	17.73	125	28.03	111	35.05	142
						13.07	147	17.55	166	22.44	110			35.41	170
						14.41	215	18.59	252	23.91	167				
						18.27	1	24.37	78						
						28.58	22	26.98	39						
						30.59	51								
CONFINING PRESSURE = 60 PSI															
8.63	239	8.72	144	8.26	145	8.75	127	11.44	190	14.86	178	14.68	9	19.94	104
9.50	268	8.75	198	10.81	53	10.28	189	12.09	128	15.16	26	17.32	206	20.51	203
9.69	233	9.18	88	11.40	41	10.72	260	13.35	204	15.83	205	17.76	27	26.41	267
9.99	259	9.49	240	11.49	89	12.97	13	14.18	294	15.90	129	20.40	15	29.72	130
11.86	292	12.12	212	11.66	89	13.32	80	14.33	261	18.04	14	21.06	133	31.18	169
		12.77	234			13.34	146	14.73	177	21.15	223	22.39	265	33.81	266
		13.21	207			15.26	213	16.08	74	21.27	107	23.08	226	34.60	175
		13.78	55			15.73	7	16.70	161	22.89	8	23.76	108	39.00	99
		13.85	40			18.45	224	17.47	214	24.63	155	24.99	179	39.07	115
		16.68	52			18.71	25	18.89	291			27.06	270	41.03	143
						38.60	42	19.10	269			49.10	57	43.80	98
								25.12	81					79.27	54
								25.22	106						
								31.07	90						
CONFINING PRESSURE = 80 PSI															
8.80	241	8.21	112	7.56	68	11.41	197	12.13	278	12.11	281	17.33	12	22.04	105
8.84	277	8.96	195	10.20	196	11.65	159	12.26	280	14.74	257	18.49	282	22.71	103
9.64	230	9.04	162	10.43	44	14.45	114	12.41	150	14.86	296	19.23	279	23.90	132
12.12	247	10.62	85	11.90	163	16.84	16	12.94	118	15.19	59	20.08	258	25.01	188
12.88	295	11.08	231	12.58	113	17.22	249	13.23	86	16.33	172	20.92	120	27.78	92
		12.22	209	13.77	210	17.41	164	14.57	211	16.90	119	23.26	152	29.17	93
		13.00	242	14.11	228	18.08	10	15.71	171	17.39	225	23.51	232	29.68	134
		13.07	43	15.22	248			15.80	58	20.29	219	26.85	217	29.74	176
		14.15	91	16.94	73			17.84	256	20.57	11	28.59	173	31.31	123
		15.19	67	17.54	79			18.10	216	24.65	87	30.54	30	58.43	84
								28.35	160	24.74	17	32.83	18		
								32.97	45	28.75	151	38.39	60		
										42.83	29				

TESTS OMITTED BECAUSE OF THE TWO STRESS DEVIATOR CRITERIA

168 (20-180)
136 (40-45), 51 (40-60), 39 (40-75)
42 (60-60), 90 (60-75), 57 (60-120), 54 (60-180)
45 (80-75), 29 (80-90), 60 (80-120), 84 (80-180)

TESTS OMITTED BECAUSE OF ERRORS IN TESTING TECHNIQUE

34 (20-90), 250 (40-20), 69 (60-180, 75 (60-180)

TABLE II

SUMMARY OF CP-ST COMBINATION RESULTS*

Confining Pressure (PSI)	SET TIME OF CONCRETE (MINUTES) 20	30	45	60	75	90	120	180
	Average Deviator Stresses (psi)							
20	6.81	8.69	8.68	11.56	11.80	13.14	17.91	32.84
40	10.23	11.54	11.07	13.89	15.37	16.17	19.20	28.52
60	9.93	11.86	10.72	13.75	16.82	18.86	21.25	32.64
80	10.45	11.55	13.02	15.29	15.76	18.88	23.78	26.82
	Average Associated Strains (%)							
20	18.6	18.3	18.7	17.4	16.4	9.6	8.1	7.6
40	16.8	15.3	15.1	14.5	12.5	9.6	6.3	3.3
60	18.2	15.2	15.8	13.8	11.6	8.3	4.8	3.4
80	15.2	16.8	13.8	11.0	10.7	9.3	7.8	2.7
	Deviator Stress Coefficient of Variation							
20	13.5	28.2	25.8	34.8	28.3	11.2	19.3	11.6
40	21.1	18.1	14.4	37.2	28.0	25.1	26.6	18.7
60	11.9	22.8	13.2	24.2	26.0	19.5	17.7	24.5
80	18.3	20.3	24.0	18.4	29.8	26.5	22.0	12.8
	Number of Tests							
20	5	9	6	11	8	5	10	10
40	4	8	7	12	11	10	8	9
60	5	10	5	10	13	9	10	11
80	5	10	10	7	11	12	11	9

*Not including eliminated tests.

slightly over and had eleven and twelve tests, respectively, their results were considered satisfactory. Overall, the coefficients of correlation were very satisfactory considering the many factors which were involved. The average coefficient of correlation for all tests was 21.7 per cent. The major factor was the human one involved in measuring, mixing, and consolidating the concrete samples.

On Figures 11, 12, 13 and 14 the average deviator stress and an eighty per cent confidence interval were superimposed on the previous individual tests.

In this analysis three assumptions were made:

1) As the sample deformed, the cross-sectional area remained uniform throughout the height.

2) The concrete sample did not slump after the compaction mold was removed until the time of testing.

3) The membrane did not contribute to the total strength of the specimen.

Appendix D gives methods to correct for these assumptions if desired.

CHAPTER III

ANALYSIS OF EXPERIMENTAL RESULTS

3.1 Triaxial Test Results

Concrete is a composite material consisting of a binding medium (cement paste) within which aggregate is embedded. The strength is inherent in the binding medium used whereas the primary function of the aggregate is to provide an inexpensive filler material. The concrete has virtually no shear strength immediately after it is produced by mixing the water with the cement and aggregate. However, as the cement combines with the water the process of hydration occurs and the cement paste hardens. Associated with this is a corresponding increase in shear strength. Herein lies one of the main points of this investigation.

In Chapter II the deviator stress was plotted as a function of set time for each of the four confining pressures (see Figures 11, 12, 13 and 14). Straight line characteristics were evident in these plots; thus a linear regression of the average deviator stress and set time was computed for each. The general equation of a linear regression is

$$\hat{D} = \beta_0 + \beta_1 T \tag{3-1}$$

and

$$\beta_1 = \frac{\Sigma dt}{\Sigma t^2} = \frac{\Sigma DT - \frac{\Sigma D \Sigma T}{h}}{\Sigma T^2 - \frac{(\Sigma T)^2}{h}} \tag{3-2}$$

$$\beta_0 = \bar{D} - \beta_1 \bar{T} \tag{3-3}$$

$$\bar{D} = \frac{\Sigma D}{h} \tag{3-4}$$

$$\bar{T} = \frac{\Sigma T}{h} \tag{3-5}$$

where

$\hat{D}$ = estimate of the deviator stress from the assumed linear relationship (psi)

D = individual deviator stress values from each test (psi)

T = individual set times values for each test (minutes)

β_0 = constant of the linear equation (psi)

β_1 = slope of the linear equation (psi per minute)

$\bar{D}$ = average deviator stress for all values (psi)

$\bar{T}$ = average set time for all values (minutes)

n = total number of deviator stress values (—).

The deviator stress results for the set time of 180 minutes were somewhat irregular for the four confining pressures; thus the regressions were based solely on the set times between 20 minutes and 120 minutes. These regression lines were superimposed on the deviator stress versus set time plots. The equations of these lines are

$$\hat{DS}_{20} = 4.89 + 0.102t \tag{3-6}$$

$$\hat{DS}_{40} = 8.29 + 0.090t \tag{3-7}$$

$$\hat{DS}_{60} = 7.53 + 0.117t \tag{3-8}$$

$$\hat{DS}_{80} = 7.18 + 0.132t \tag{3-9}$$

where

$\hat{DS}_n$ = deviator stress obtained by linear regression at the confining pressure, n (psi)

t = set time of the concrete (minutes).

From these plots it was evident that the set time was very influential in determining the concrete strength. The influence of the confining pressure, however, was considerably less.

A confidence interval on each of these regression lines can be found to determine their relative precision. This results in a band width which centers on the regression line. The standard deviation of the deviator stress for a fixed set time must first be computed using the equation

$$S_{DT} = \sqrt{\frac{\Sigma(D - \hat{D})^2}{n - 2}} = \sqrt{\frac{\Sigma d^2 - \frac{(\Sigma dt)^2}{\Sigma t^2}}{n - 2}} \tag{3-10}$$

and

$$\Sigma d^2 = \Sigma D^2 - \frac{(\Sigma D)^2}{n} \tag{3-11}$$

$$\Sigma dt = \Sigma DT - \frac{\Sigma D \Sigma T}{n} \tag{3-12}$$

$$\Sigma t^2 = \Sigma T^2 - \frac{(\Sigma T)^2}{n} \tag{3-13}$$

where

S_{DT} = standard deviation of the deviator stress for fixed set time (psi)

$\hat{D}$ = deviator stress given by the regression equations (psi)

D = individual deviator stress values from each test (psi)

T = individual set time values from each test (minutes)

n = total number of deviator stress values (—).

A confidence interval for each set time of 90 per cent is then computed by the equation

$$CI\ (\hat{D})\ 0.90 = \hat{D} \pm t_{0.10}\ S_{DT}\sqrt{\frac{1}{n} + \frac{(T - \bar{T})^2}{\Sigma t^2}} \qquad (3\text{-}14)$$

where

t_x = student "t" value for the probability of a larger value than x with n-2 degrees of freedom (—)

$\bar{T}$ = average set time of all the values (minutes).

The interval must be computed for a number of set time values to describe the band width. Figures 15, 16, 17 and 18 show the band with 90 per cent confidence interval for confining pressures of 20, 40, 60 and 80 psi, respectively.

A generalized equation was developed for the deviator stress as a function of set time and confining pressure. This equation is

$$DS = 0.1P + 0.135t \qquad (3\text{-}15)$$

where

DS = deviator stress (psi)

P = confining pressure (psi)

t = set time of concrete (minutes).

For each set of the four confining pressures this generalized relationship was superimposed on the confidence interval bands (see Figures 15, 16, 17 and 18). Note that the generalized equation is a valid representation for all four

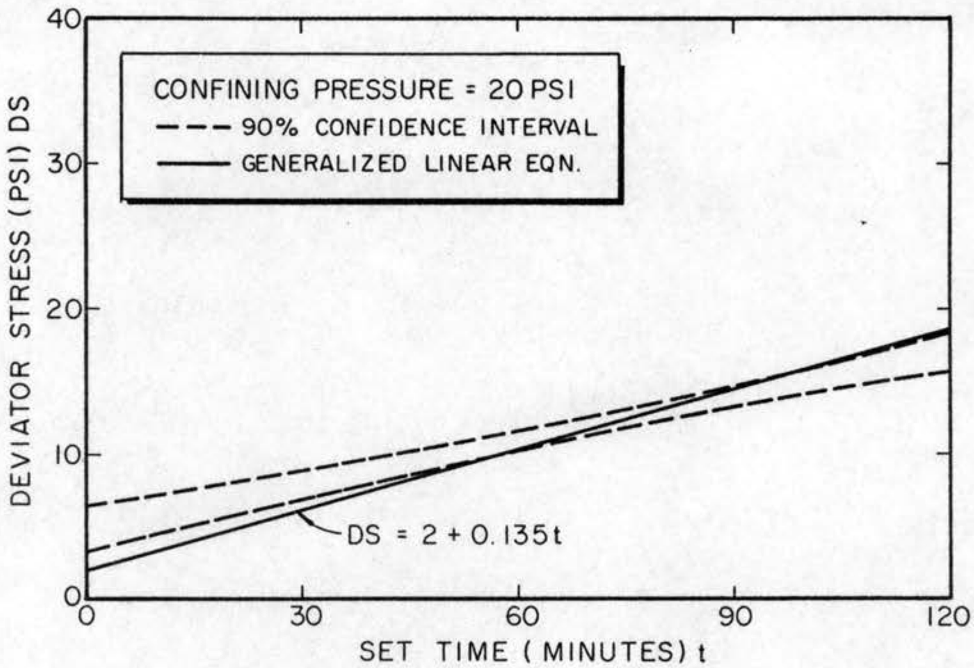

Figure 15. Deviator Stress Confidence Interval (CP = 20 PSI)

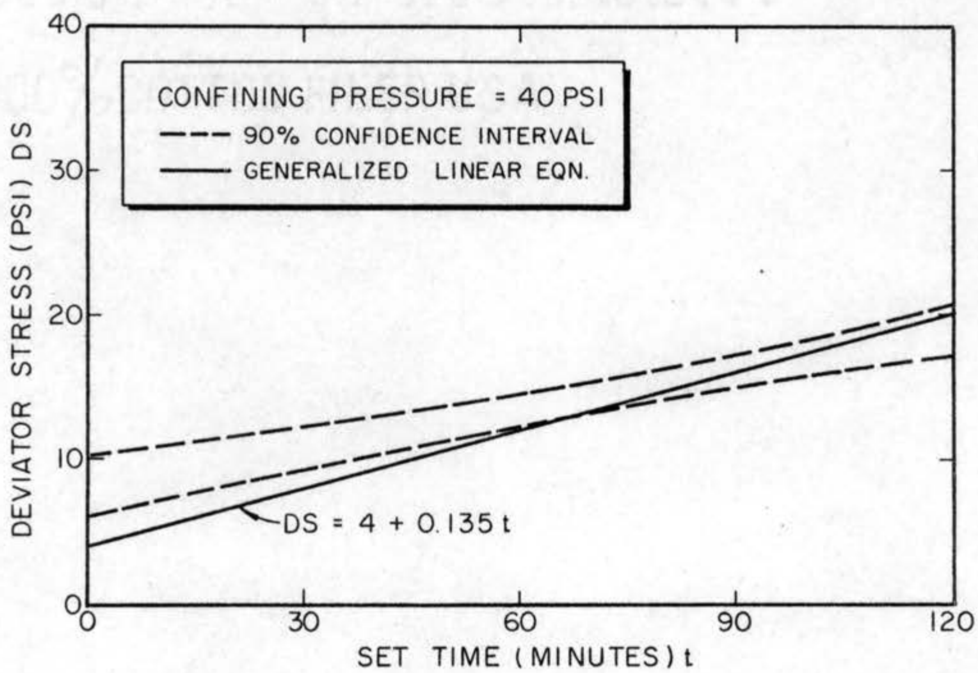

Figure 16. Deviator Stress Confidence Interval (CP = 40 PSI)

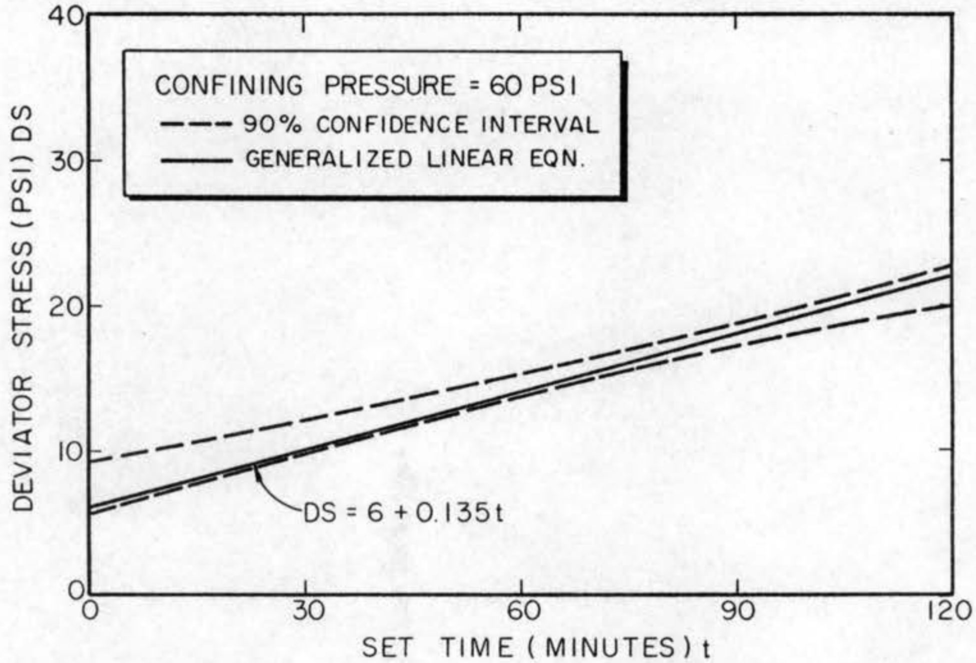

Figure 17. Deviator Stress Confidence Interval (CP = 60 PSI)

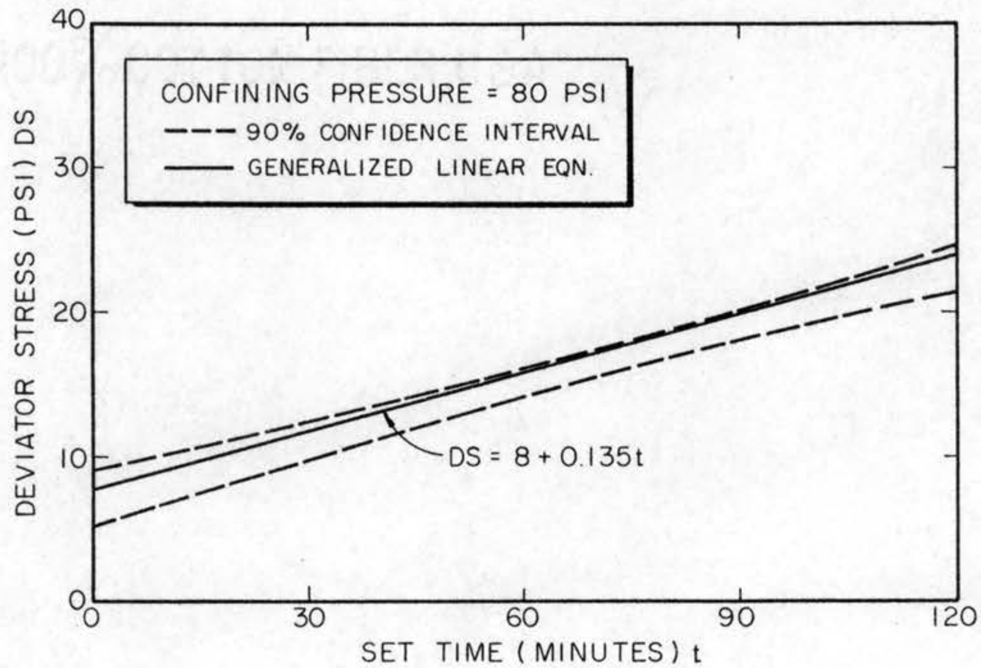

Figure 18. Deviator Stress Confidence Interval (CP = 80 PSI)

confining pressures.

While developing the shear strength, the concrete sample underwent a subsequent amount of deformation. This deformation was recorded and computed in the form of axial strain. Thus, when the deviator stress occurred there was an associated strain produced in the concrete. To have a more complete picture of the concrete shear strength it is also necessary to understand the relationship between this associated strain and set time. Therefore, the associated strain was plotted as a function of set time and a linear regression was performed for each of the four confining pressures. Since the result for 180 minutes was discarded in the calculations for the deviator stress regression lines they were also discarded for the associated strain regression lines. These plots are shown in Figures 19, 20, 21 and 22. The equations obtained for the associated strain versus set time were

$$\hat{\epsilon}_{20} = 22.54 - 0.1152t \tag{3-16}$$

$$\hat{\epsilon}_{40} = 19.34 - 0.1030t \tag{3-17}$$

$$\hat{\epsilon}_{60} = 20.70 - 0.1299t \tag{3-18}$$

$$\hat{\epsilon}_{80} = 17.64 - 0.0884t \tag{3-19}$$

where

$\hat{\epsilon}_n$ = associated strain at the confining pressure, n(psi)

t = set time of the concrete (mix).

A generalized equation was established for the associated strain in terms of the confining pressure and set time. This equation is

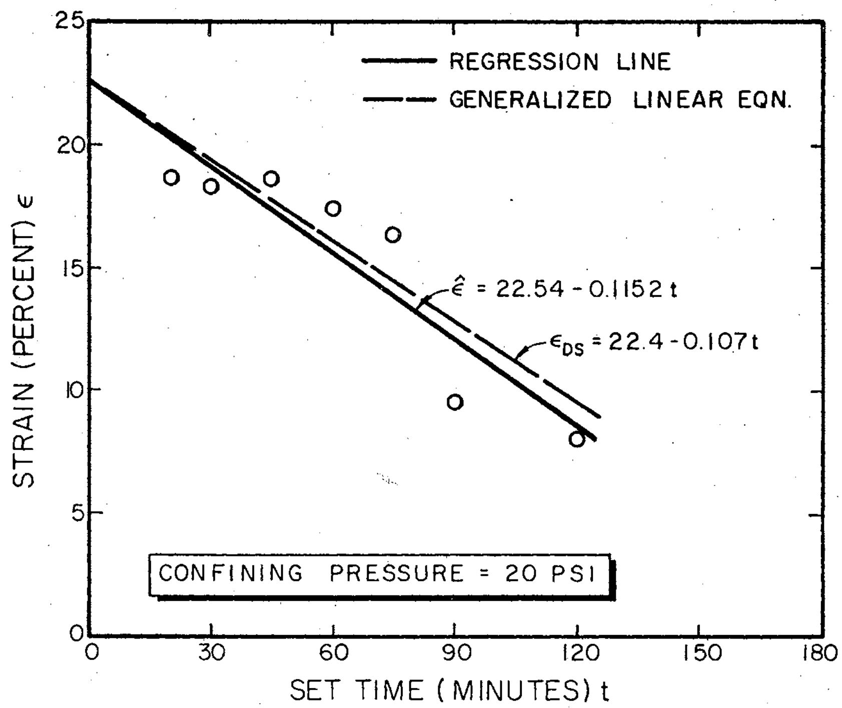

Figure 19. Associated Strain Versus Set Time (CP = 20 PSI)

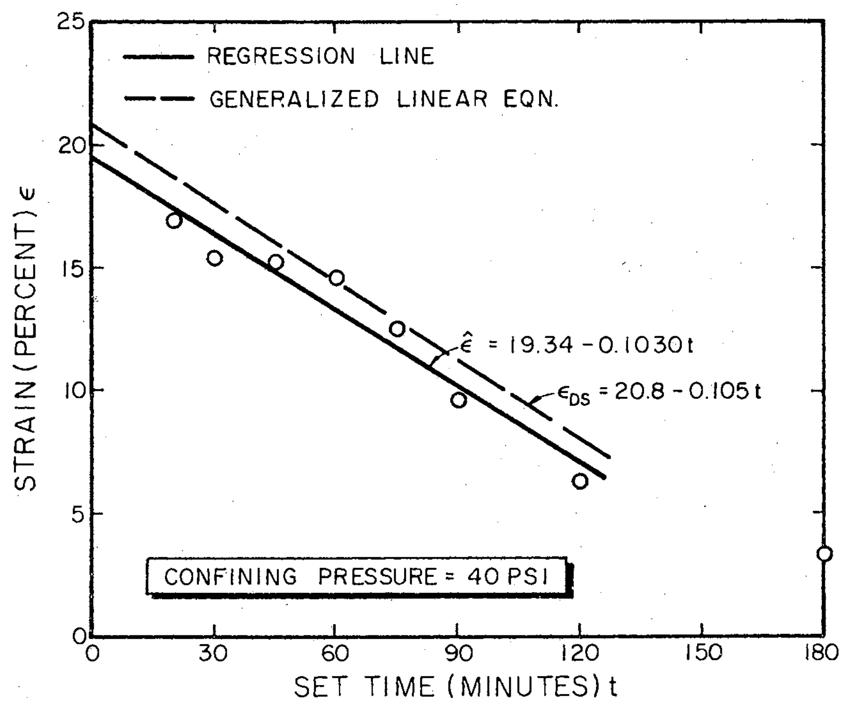

Figure 20. Associated Strain Versus Set Time (CP = 40 PSI)

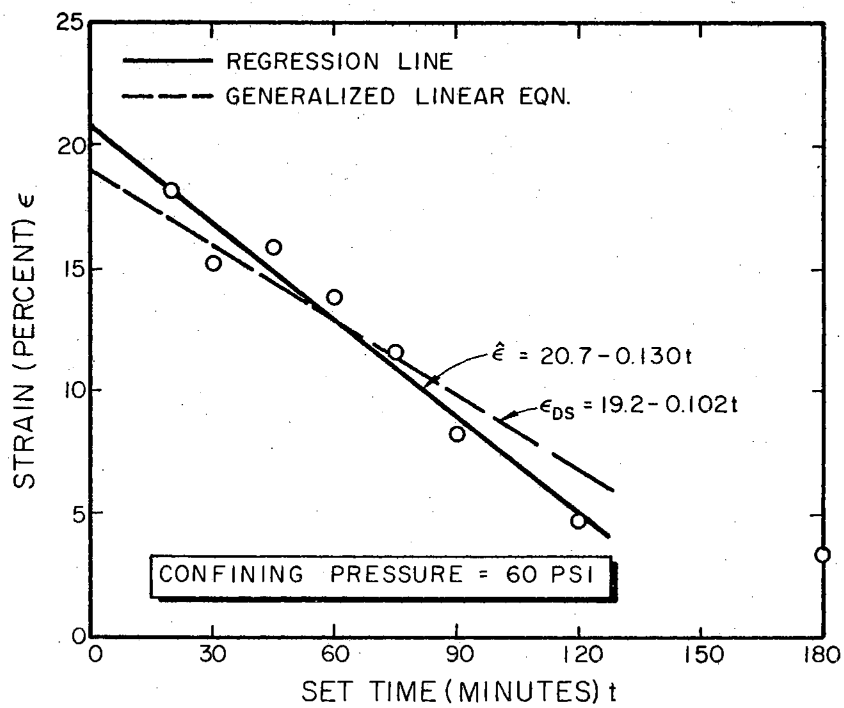

Figure 21. Associated Strain Versus Set Time (CP = 60 PSI)

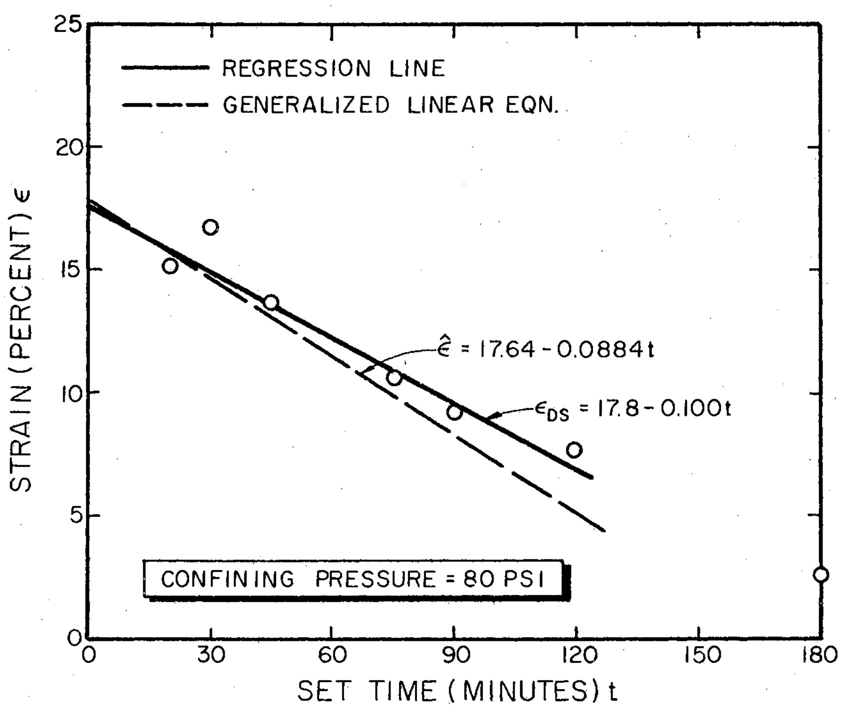

Figure 22. Associated Strain Versus Set Time (CP = 80 PSI)

$$\epsilon_{DS} = (24.0 - 0.08P) - (0.11 - 0.00013P)t \qquad (3\text{-}20)$$

where

ϵ_{DS} = strain associated with the maximum deviator stress (per cent)

P = confining pressure (psi)

t = set time of the concrete (minutes).

This generalized equation was superimposed on the associated strain versus set time plots for the four confining pressures (see Figures 19, 20, 21 and 22). This equation agrees with the actual regression lines reasonably well in all four cases.

An original assumption was that concrete, while in a plastic state, obeyed the Mohr-Coulomb Rupture theory. To see if this assumption was justifiable a Mohr diagram was constructed for each of the eight set times (see Figures 23-30). In each diagram there were four stress circles at failure, one for each of the confining pressures. The diameters of these circles were the average deviator stresses shown in Table II. A straight line was then drawn tangent to these circles and constituted the failure envelope.

In all the Mohr diagrams, with the exception of that for 180 minute set time, the failure envelopes were tangent to at least three of the four stress circles. From this the Mohr-Coulomb theory was seen to apply to concrete. In the case of 180 minutes the stress circles for 40 and 60 psi were used for it appeared the resulting line was consistent with the seven other failure envelopes. The resulting Coulomb equations for the eight set times were:

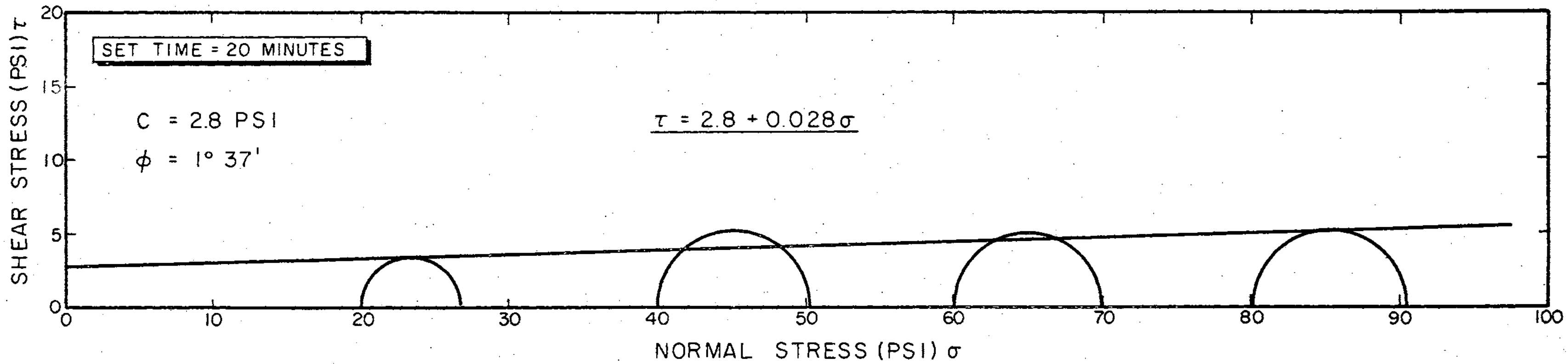

Figure 23. Mohr Diagram (Set Time = 20 MIN)

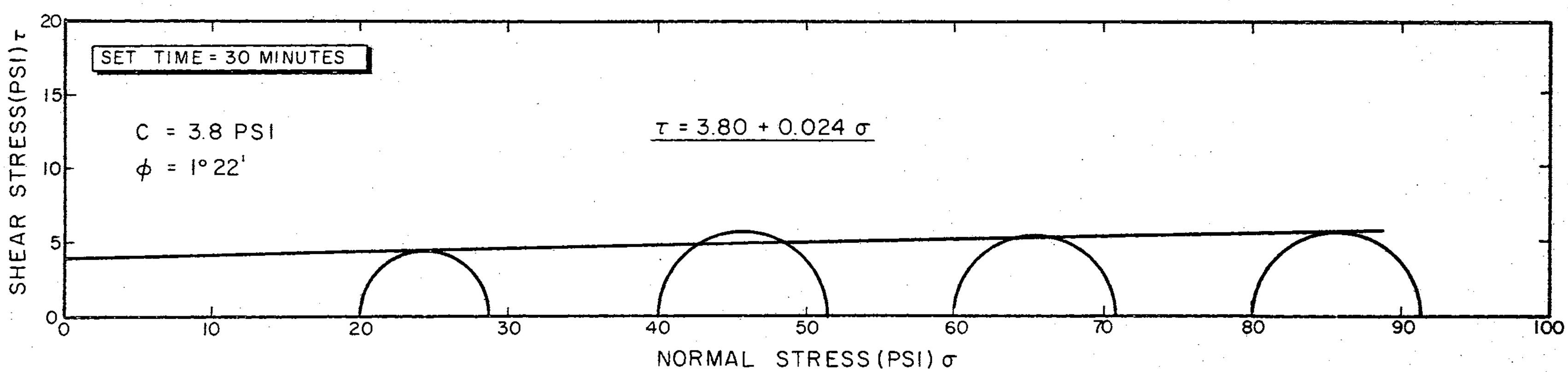

Figure 24. Mohr Diagram (Set Time = 30 MIN)

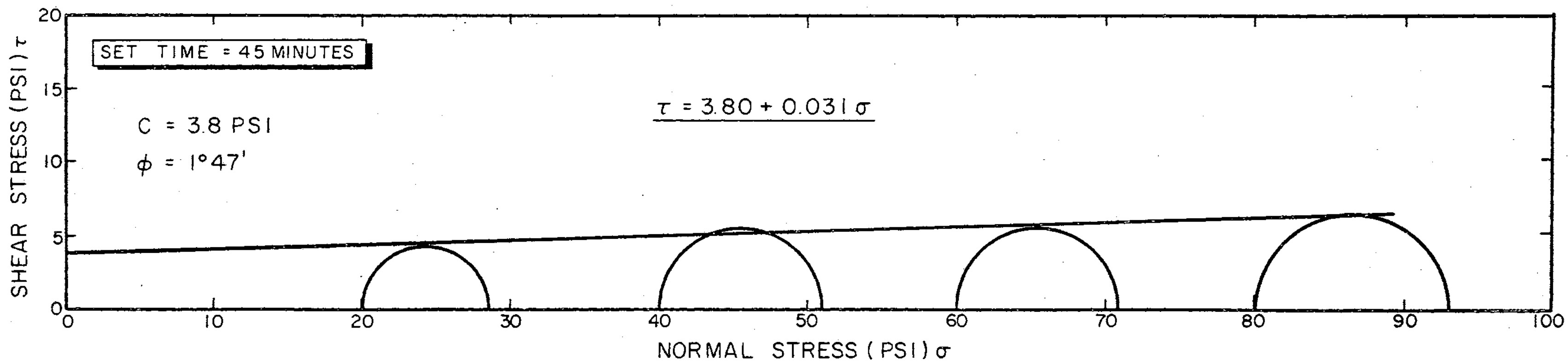

Figure 25. Mohr Diagram (Set Time = 45 MIN)

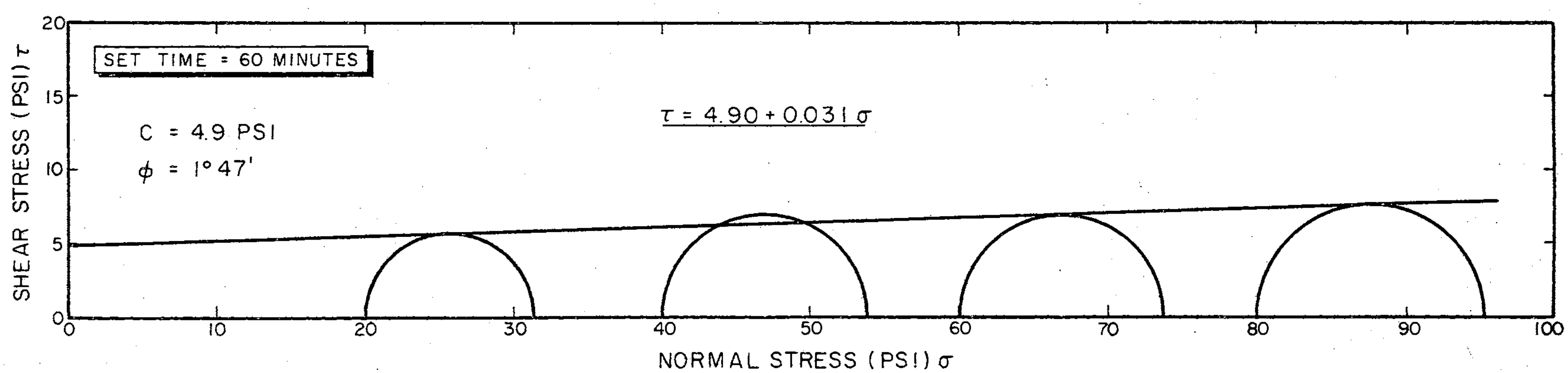

Figure 26. Mohr Diagram (Set Time = 60 MIN)

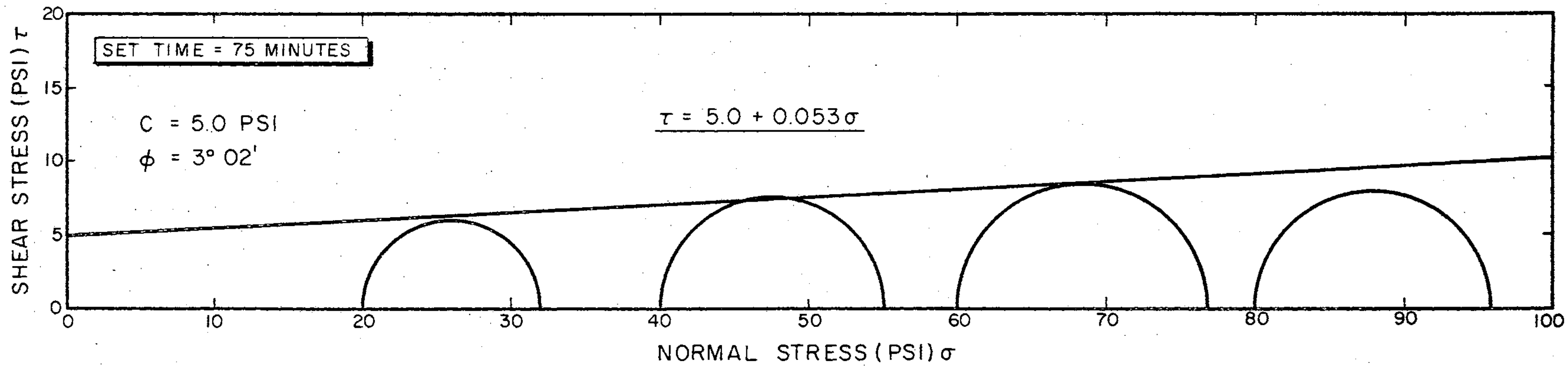

Figure 27. Mohr Diagram (Set Time = 75 MIN)

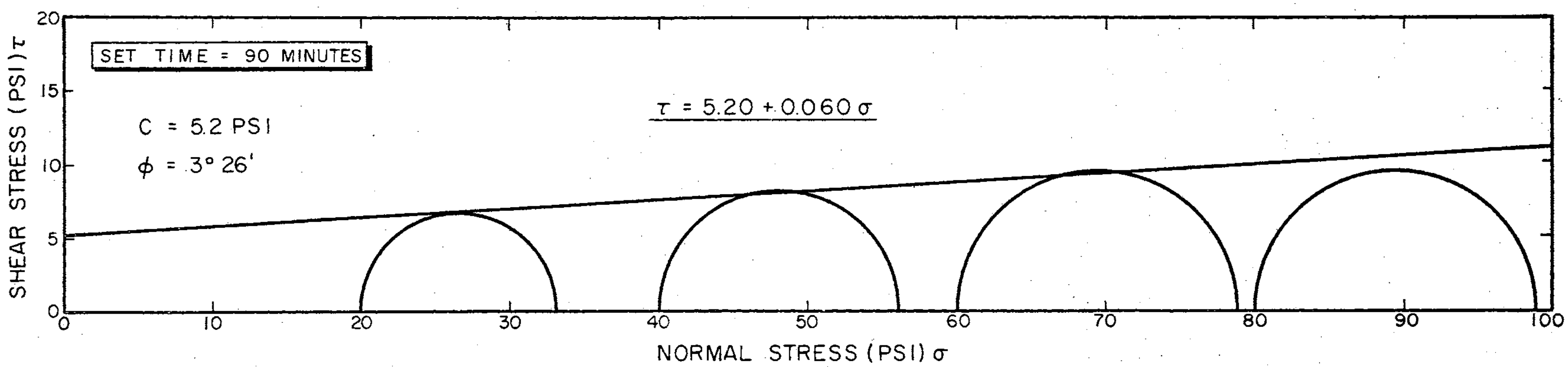

Figure 28. Mohr Diagram (Set Time = 90 MIN)

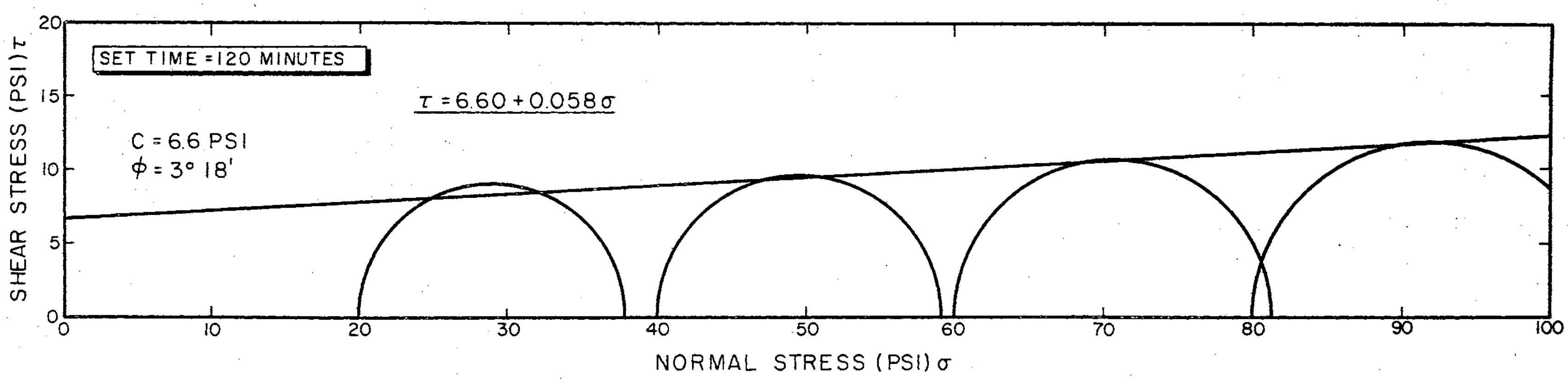

Figure 29. Mohr Diagram (Set Time = 120 MIN)

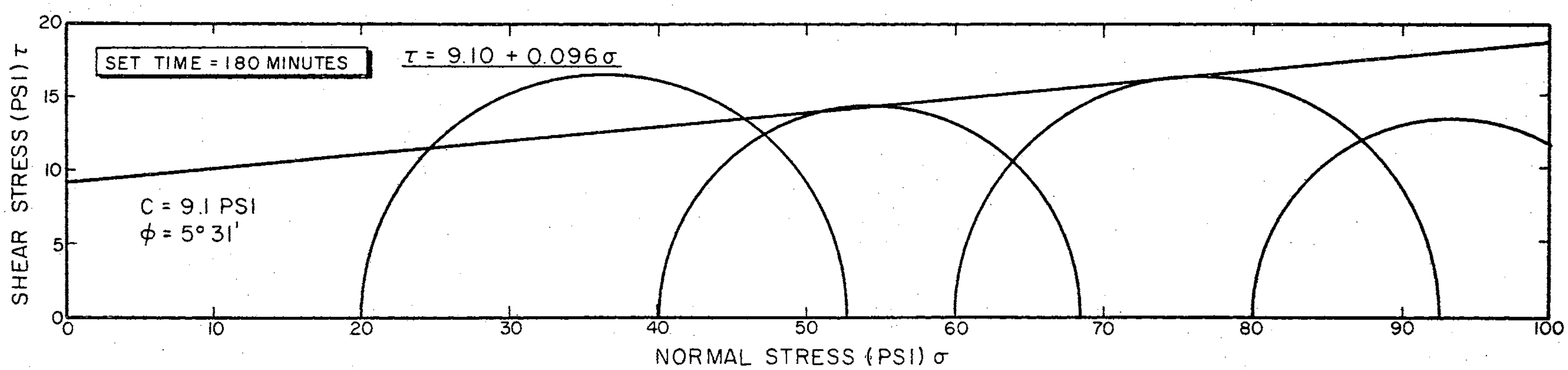

Figure 30. Mohr Diagram (Set Time = 180 MIN)

For 20 minute set time, $\tau = 2.8 + 0.028\ \sigma$ (3-22)

For 30 minute set time, $\tau = 3.8 + 0.024\ \sigma$ (3-23)

For 45 minute set time, $\tau = 3.8 + 0.031\ \sigma$ (3-24)

For 60 minute set time, $\tau = 4.9 + 0.031\ \sigma$ (3-25)

For 75 minute set time, $\tau = 5.0 + 0.053\ \sigma$ (3-26)

For 90 minute set time, $\tau = 5.2 + 0.060\ \sigma$ (3-27)

For 120 minute set time, $\tau = 6.60 + 0.058\ \sigma$ (3-28)

For 180 minute set time, $\tau = 9.10 + 0.096\ \sigma$ (3-29)

where

τ = concrete shear stress (psi)

σ = concrete normal stress (psi).

Figure 31 shows the failure envelopes in a three dimensional plot with the third dimension set time. The cohesion and internal friction angle for each set time were calculated and are shown in Table III. These were also plotted as a function of set time on Figures 32 and 33, respectively.

TABLE III

CONCRETE STRENGTH PARAMETERS VERSUS SET TIME

Set Time minutes	Cohesion psi	Internal Friction Angle degrees - minutes
20	2.8	1° 34'
30	3.8	1° 22'
45	3.8	1° 47'
60	4.9	1° 47'
75	5.0	3° 02'
90	5.2	3° 26'
120	6.6	3° 18'
180	9.1	5° 31'

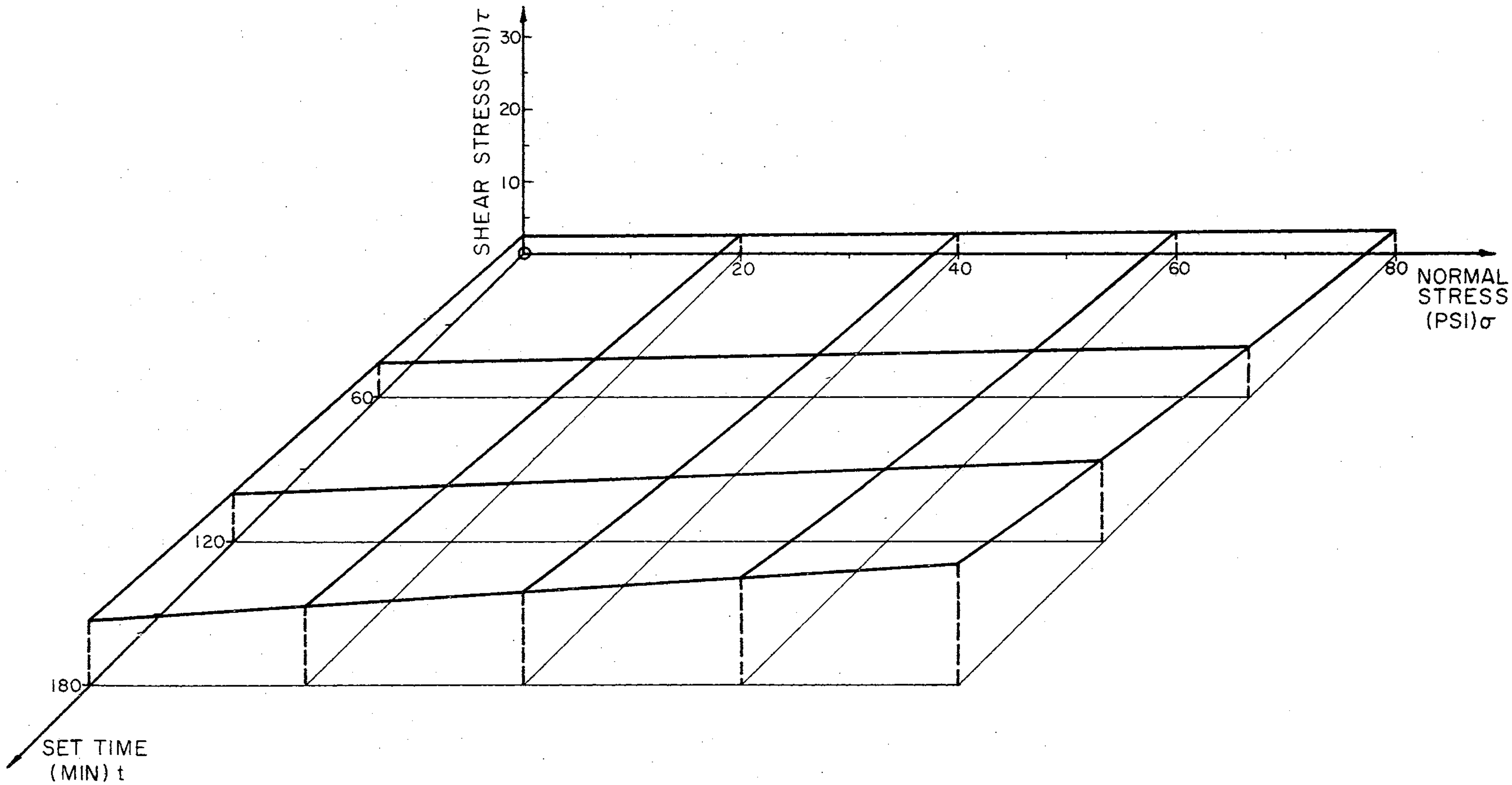

Figure 31. Mohr Diagrams with Time Variation

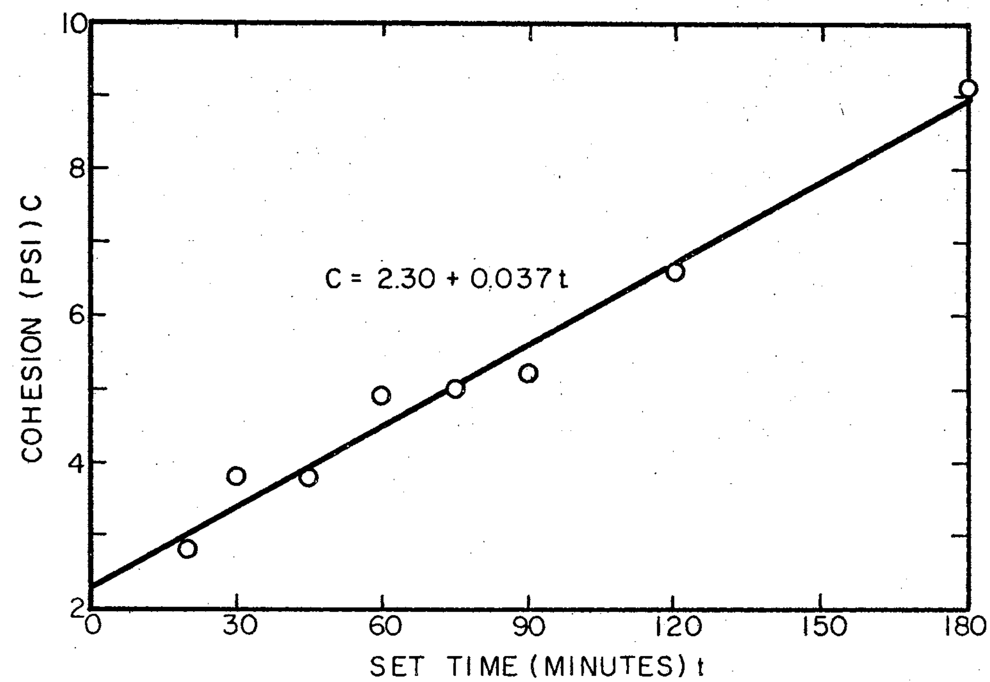

Figure 32. Cohesion Versus Set Time

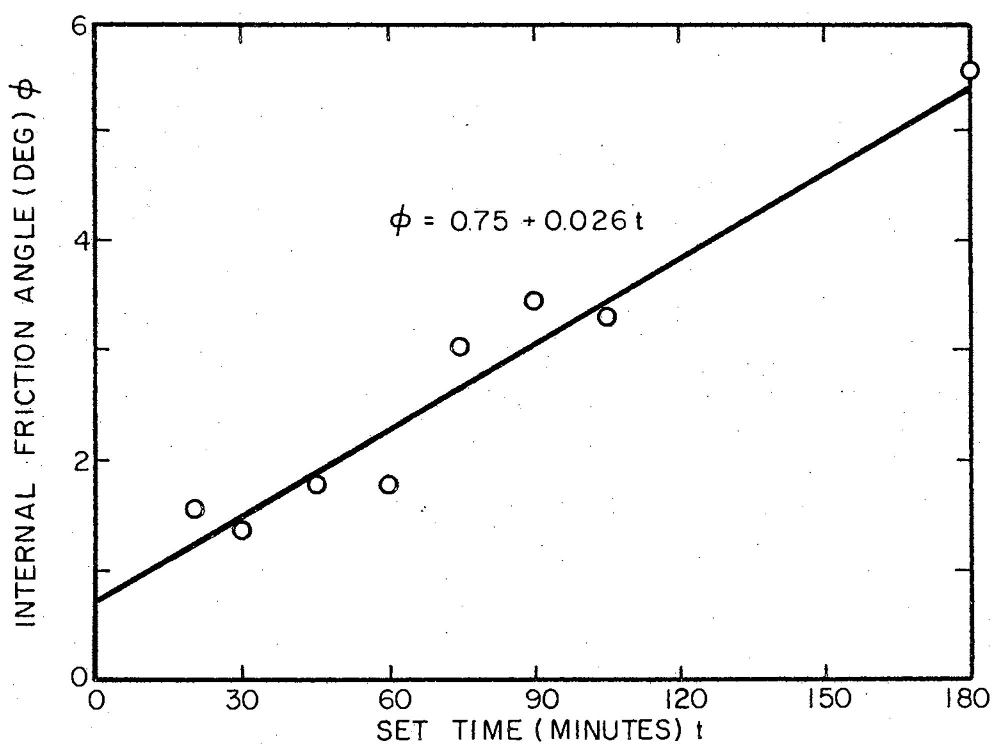

Figure 33. Internal Friction Angle Versus Set Time

From the Mohr-Coulomb diagrams it is evident that the shear strength consists mainly of cohesive bond in the cement paste for the lower set times. As the concrete continues to set the paste becomes less plastic and the mobility of the aggregate particles decreases. This causes the internal friction angle to increase. The cohesion continues to steadily increase as the set time increases indicating that the cement is combining with the water to form the binding medium.

CHAPTER IV

CALCULATION OF CONCRETE PRESSURES

4.1 Development of a New Method Relating Concrete Shear Strength to Lateral Pressures

This study has been concerned with the determination of concrete shear strength. A method will now be developed which relates the shear strength of the concrete to the lateral pressures of the concrete on formwork. It should be noted that the concrete shear strength was determined using only one mix, thus the method has limited application. Later in the chapter, pressures obtained from this new method will be compared with the pressures calculated from the ACI and CERA formulas. The new method combines theory, test results, and field experience in an intuitive approach to the problem.

Based on Rankine's Theory, a linear pressure distribution could have been assumed; however, the boundary conditions associated with concrete formwork did not permit this. The tie action in the formwork provides intermediate constraints which do not allow deflections necessary to justify a linear pressure distribution. The work of Terzaghi and Peck (23) on braced cuts in soft Chicago clay was investigated. Because the properties of both soft clay and plastic concrete have similar characteristics and the boundary conditions associated with wall formwork and braced cuts are also similar, it was decided to incorporate the formula they recommended for the lateral pressure. This formula is

$$P = \gamma h - 2q_u \tag{4-1}$$

where

P = pressure on wall

γ = unit weight of the material

h = height of wall

q_u = unconfined compressive strength of the material.

The unconfined compressive strength for the concrete and the associated strain can be evaluated by Equations 3-15 and 3-20, setting P equal to zero.

With this relationship it was necessary for the concrete to deform a certain amount to develop the full shear strength. This was defined as shear strength mobilization. In a braced cut even the most workmanlike procedure of installing the bracing produces enough deformation to develop full shear mobilization. This is not the case in concrete formwork. To develop full shearing strength, the concrete must undergo an axial strain of anywhere from 3 to 20 per cent depending on the amount of set time. However, in concrete work the formwork limits the amount of axial strain to about 2 per cent since maximum allowable deflections are established. Because of this it was necessary to reduce the amount of shear strength mobilized by the axial strain of the concrete. This was done by implementing the stress-strain relationship of the concrete for the specified time of set. The first step was to develop a relationship between the deflections in the formwork and this strain.

In Figure 34, a typical concrete wall section is shown with a linear deflection pattern assumed for ease of calculations. Also, assuming that the concrete did not expand or contract appreciably, the area of A must be equal to the area of the two triangles B. Noting that the deflection of a linear approximation was greater than the actual value, a multiplier of 1.3 was added to account for the difference.

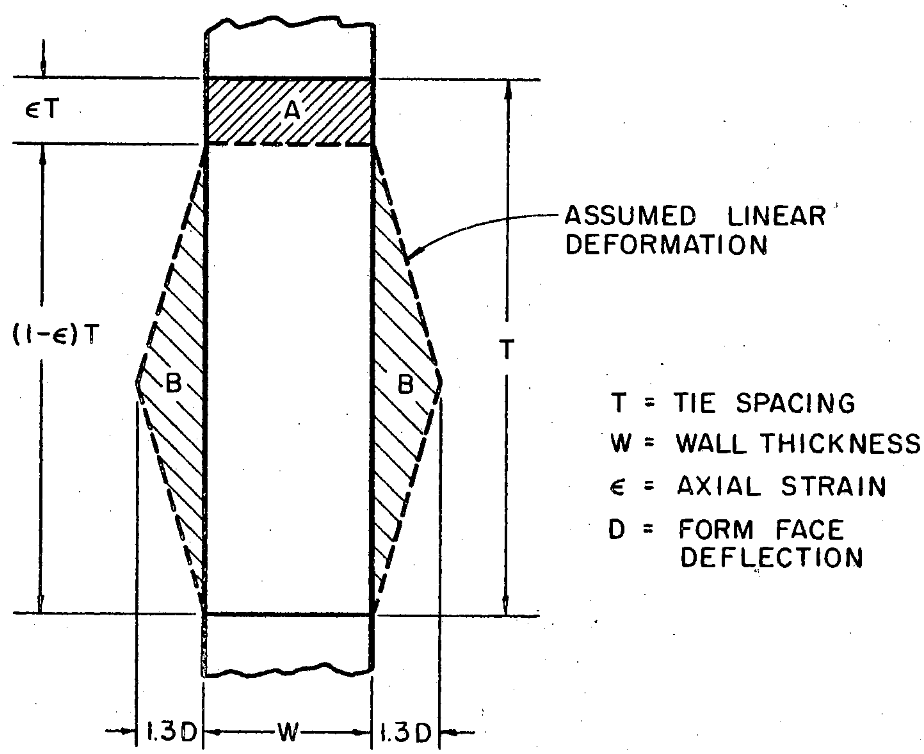

Figure 34. Typical Concrete Wall Section

The value of this multiplier was determined by approximating the observed deflections in actual formwork. When this was done the equation for the strain became

$$(\epsilon T)W = 2 \times \frac{1.3D(1 - \epsilon)T}{2}$$

which reduces to

$$\epsilon = \frac{1.3D}{W + 1.3D} \qquad (4\text{-}2)$$

where

ϵ = actual axial strain in concrete (per cent)

D = deflection of formwork face (inches)

W = wall thickness (inches).

A question arose as to the amount of deflection the form face undergoes. This depends on the rigidity of the formwork, the elongation of the tie rods, and the tightness associated with the bearing and connections. The first two could be fairly well established as a function of load. For instance in the case of a 1000 psf concrete pressure, these together were approximately 0.10 inches. The tightness, however, was very difficult, if not impossible to determine. For this reason another approach had to be taken. Under extreme conditions, a 1/4 inch deflection was thought to be reasonable. Also noting that the deflection should be a direct function of the load the following relationship was established,

$$D = 0.02P \qquad (4\text{-}3)$$

where

D = deflection of the formwork face (inches)

P = formwork pressure (psi).

It should be pointed out that this equation has been arrived at intuitively; however, the deflections obtained are conservative. A method for obtaining greater refinement of this equation is suggested in the next chapter.

Using Equation 4-3, a 1500 psf concrete pressure in a 12 inch wall would give a deflection of 0.20 inches in the formwork face. With this deflection a strain of 2.1 per cent in the vertical direction was calculated using Equation 4-2.

To determine the concrete pressure it was helpful to first evaluate the coefficient of the concrete pressure in relation to a totally fluid action with a concrete unit weight of 150 pounds per cubic foot. In doing this the coefficient relationship became

$$K_{max} = \frac{150h - 288q_{max}}{150h} \qquad (4\text{-}4)$$

where

K_{max} = coefficient of concrete pressure at failure(—)

h = height of concrete head (ft)

q_{max} = shear strength of the concrete at failure (psi).

The stress-strain relationship was assumed to be linear between the origin and the point of failure (see Figure 3). However, for low values of strain, this linear relationship decidedly underestimated the concrete shear strength. This was particularly true for low set times. For example, at a 30 minute set time and 60 psi confining pressure the shear strength at two per cent given by the linear approximation was 1.2 psi whereas the actual shear strength was 2.9 psi (see Figure B.4 in Appendix B). To more accurately represent the stress-strain relationship a third order curve was drawn between the origin and point of failure in the concrete. See Figure 35 for a general plot illustrating this relationship. The value of coefficient K includes the negative of the shear strength (Equation 4-4), thus when K was plotted versus the vertical strain in the concrete, the plot became concave upward.

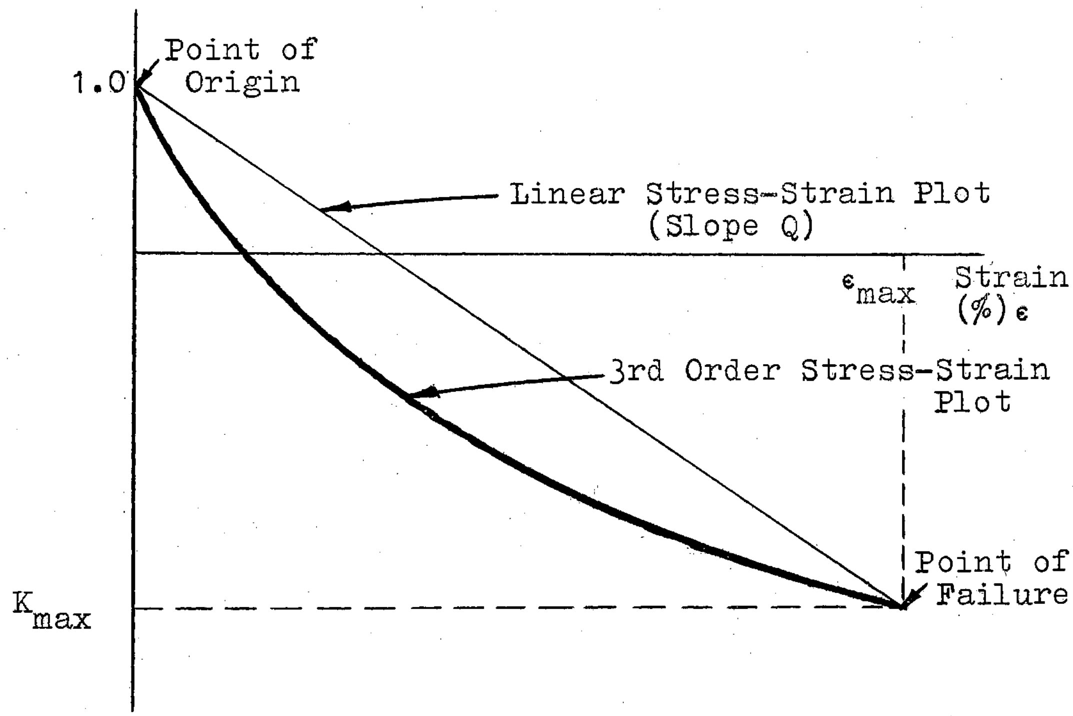

Figure 35. Coefficient Versus Strain

Negative values of K indicate that the concrete, with a certain amount of deformation, would be able to support the concrete head without any additional lateral support. Because of the absence of shear strength immediately after mixing the coefficient became one which depicts a fluid head action.

The slope at the origin was greater than that given by a linear approximation and at failure it was less. Also from the actual stress-strain curves it could be noted that the per cent increase in the slope at the origin was approximately twice the magnitude of the per cent decrease of the slope at failure. The equation used to calculate K was

$$K = A\epsilon^3 + B\epsilon^2 + C\epsilon + D \tag{4-5}$$

and the boundary conditions were

$$K\ (@\epsilon = 0) = 1.0$$

$$\frac{dK}{d\epsilon}\ (@\epsilon = 0) = (1 + \rho_1)Q$$

$$K\ (@\epsilon = \epsilon_{max}) = K_{max}$$

$$\frac{dK}{d\epsilon}\ (@\epsilon = \epsilon_{max}) = (1 - \rho_2)Q$$

where

$Q = -\dfrac{1 - K_{max}}{\epsilon_{max}}$ = slope of the linear $K - \epsilon$ plot (see Figure 35)

K_{max} = coefficient of concrete pressure at failure(—)

ϵ_{max} = axial strain of the concrete at failure (per cent) Equation 3-20

K = coefficient of concrete pressure at the concrete strain (—)

ϵ = actual amount of vertical strain in the concrete (per cent)(Equation 4-2)

ρ_1 = fractional increase in stress-strain slope from linear plot at origin (—)

ρ_2 = fractional decrease in stress-strain slope from linear plot at failure (—).

The constants A, B, C and D used in Equation 4-5 to calculate K were evaluated using the boundary conditions to be

$$A = \frac{(2 + \rho_1 - \rho_2)\epsilon_{max}Q - 2K_{max} + 2}{\epsilon_{max}^3} \tag{4-6}$$

$$B = \frac{3K_{max} - 3 - (3 + 2\rho_1 - \rho_2)\epsilon_{max}Q}{\epsilon_{max}^2} \tag{4-7}$$

$$C = (1 + \rho_1)Q \tag{4-8}$$

$$D = 1. \tag{4-9}$$

The linear approximation of the stress-strain curve was much closer to the actual stress-strain curve as the set time increased. For this reason it was not possible to set an exact value on ρ_1 and ρ_2. Since the rate of pour, R, directly influences the time at which the maximum pressure was developed, it was felt that these slopes, ρ_1 and ρ_2, should be made a function of R. After an actual analysis two straight lines were established for ρ_1 in terms of R. These were

$$\text{for } R < 14 \text{ fph } \rho_1 = R/9 - 0.3 \tag{4-10}$$

$$\text{for } R \geq 14 \text{ fph } \rho_1 = R/40 + 1.0 \tag{4-11}$$

and

$$\rho_2 = 0.5\,\rho_1. \tag{4-12}$$

This indicates that for low rates of pour the maximum pressure occurs at relatively high set times and the per cent increase in the slope would be small in comparison to the linear plot. This was intuitively correct. Note that when R = 0 the slope was negative; however, this was not considered relevant for values of R less than one are never encountered in construction practice. The validity of these relationships will be shown later.

In Equation 4-4, the variable h was given as the height of concrete head and was equal to the rate of pour multiplied by the elapsed time since the placing operation had begun. In the same equation the shear strength of the concrete is also a function of time but it commences when the concrete was mixed and began to set. This difference was defined as the time of initial set and was equal to the elapsed time between the commencement of mixing and placing operations. It may be recalled that the concrete shear strength consists mainly of cohesive bond in the cement paste and not interlocking forces created by the aggregates. It was therefore necessary to consider this fact and include it in the analysis. Varying amounts of initial set time were considered and the results will be discussed later. It should be noted that this breakdown of interlocking friction was considered by the addition of a vibration factor which was computed in relation to wall thickness. In narrow walls the vibration pressure is greatly intensified, therefore, a higher vibration pressure was necessary. The CERA method added 200 psf for vibration to all pressures. An equation was developed to let the vibration pressure be approximately 200 psf for a 6 inch wall and then gradually reduce downward for wider walls. The equation is

$$P_{vib} = 100 + 300(1 - W/10) \qquad (4\text{-}13)$$

where

P_{vib} = concrete pressure due to vibration and impact factors (psf)

W = wall thickness (inches).

With this equation the vibration pressure added for a 12 inch wall was 40 psf, whereas for a 6 inch wall, the pressure was 220 psf.

The height of the actual wall being considered must be included in the analysis. This is particularly evident for low walls with high rates of pour. The time in which concrete develops its maximum pressure is usually on the order of one hour after placing. If an 8 foot high wall was poured at 12 feet per hour, the total time of pouring would only be 40 minutes. Most formulas consider this case as an equivalent fluid head of concrete; however, this method completely disregards the fact that the concrete has already had 40 minutes of set (plus any initial set time) to develop shear strength. Thus in Equation 4-4, the value of h was set equal to the lesser of the wall height or rate of pour times the time of placing.

In the actual pressure analysis the equations developed for the deviator stress (Equation 3-15) and associated strain

(Equation 3-20) were used. A computer program that aided in the analysis is shown as Program V in Appendix C. With this program the pressures could be evaluated by the new method for different amounts of placement times. A pressure was calculated and then compared with the largest previously calculated pressure and if it was larger, then it stored the pressure as the maximum. The following four factors were singled out for analysis with this program;

1 - Rate of pour

2 - Time of initial set

3 - Height of wall

4 - Wall thickness.

In past literature, great importance was placed on the rate of pour; therefore, it was included together with each of the other three factors. This was also necessitated by the rate of pour's interrelationship with each of the other factors.

To begin the program it was necessary to have an estimate of the pressure in order to calculate the form face deflection. The ACI formula was used for this estimate.

A summary of the calculations to find the pressures by the new method follows. For each of the placement times,

1 - Calculate the ACI pressure using Equations 1-4 and 1-7:

for $R < 7$ fph whichever is lesser of

$$P = 150 + \frac{9000R}{T} \text{ or } 150\,h \qquad (1\text{-}4)$$

for $R \geq 7$ fph whichever is the least of

$$P = 150 + \frac{43400}{T} + \frac{2800R}{T} \text{ or } 150\ h \text{ or } 2000 \text{ psf} \quad (1\text{-}7)$$

2 - Calculate the deflection using the ACI pressure of 1 in Equation 4-3:

$$D = 0.02P \quad (4\text{-}3)$$

3 - Calculate the axial strain in the concrete by Equation 4-2:

$$\epsilon = \frac{1.3D}{W + 1.3D} \quad (4\text{-}2)$$

4 - Calculate the minimum head of concrete by comparing the wall height with the rate of pour multiplied by the placement time:

$$h_{con} = (R)tp \text{ or } hm \text{ whichever is less}$$

5 - Calculate the slopes of the K - ϵ curves using Equations 4-10, 4-11 and 4-12:

$$\text{for } R < 14 \text{ fph } \rho_1 = \frac{R}{9} - 0.3 \quad (4\text{-}10)$$

$$\text{for } R \geq 14 \text{ fph } \rho_1 = \frac{R}{40} + 1.0 \quad (4\text{-}11)$$

$$\rho_2 = 0.5\rho_1 \quad (4\text{-}12)$$

6 - Calculate the time of set by adding the initial set time to the placement time:

$$t_s = t_p + t_{is}$$

7 - Calculate the coefficient for the maximum deviator stress (Equation 3-15) using Equation 4-4 and the associated strain using Equation 3-20:

$$q_{max} = 0.1P + 0.135\ t_s \quad (3\text{-}15)$$

$$K_{max} = \frac{150h - 288\ q_{max}}{150h} \quad (4\text{-}4)$$

$$\epsilon_{max} = (24.0 - 0.8P) - (0.11 - 0.00013P)t_s \quad (3\text{-}10)$$

8 - Calculate the actual coefficients using the results previously evaluated in Equations 4-5 through 4-9:

$$K = A\epsilon^3 + B\epsilon^2 + C\epsilon + D \tag{4-5}$$

where

$$A = \frac{(2 + \rho_1 - \rho_2)\epsilon_{max}Q - 2K_{max} + 2}{\epsilon_{max}^3} \tag{4-6}$$

$$B = \frac{3K_{max} - 3 - (3 + 2\rho_1 - \rho_2)\epsilon_{max}Q}{\epsilon_{max}^2} \tag{4-7}$$

$$C = (1 + \rho_1)Q \tag{4-8}$$

$$D = 1.0 \tag{4.9}$$

$$Q = -\left[\frac{1 - K_{max}}{\epsilon_{max}}\right]$$

9 - Calculate the vibration pressure using Equation 4-13:

$$P_{vib} = 100 + 300\ (1 - W/10) \tag{4-13}$$

10 - Calculate the concrete pressure for this placement time by adding the coefficient multiplied by the equivalent fluid head of concrete to the vibration pressure:

$$P_{con} = 150K\ h_{con} + P_{vib}$$

11 - Compare the pressure of this placement time with that of the previous placement times and store the highest pressure. After all placement times have been evaluated the maximum pressure is punched out.

12 - Then change to the next desired input conditions (wall thickness, wall height, rate of pour and initial set time) and go through all calculations once more.

For a more thorough summary see the flow chart for Program V in Appendix C.

4.2 Comparison of Concrete Pressures Determined by the New Method with Those of the ACI and CERA Methods

As mentioned previously concrete pressures were calculated using the ACI and CERA formulas in order to correlate the pressures computed in this new method. It was realized that only one concrete mix was used and no prototype tests have been performed, thus all comparisons between the ACI, CERA, and new methods were limited in scope.

The American Concrete Institute formula was chosen because it is the recognized method in the United States and the CERA method because of the extensive investigation that was performed on actual formwork in its development. In order to use these methods, it was necessary to establish certain values to make the equations consistent with the testing conditions of the new method. For the CERA method both the concrete mix and temperature characteristics were corrected. Also the temperature of 75°F had to be included in the ACI formulas. After this, each of the three factors (initial set time, wall width and height) was combined with the rate of pour and analyzed separately.

Before these four factors are discussed, an interest of note concerning the slopes, ρ_1 and ρ_2 of the K - ϵ plots should be pointed out. The actual value of ρ_1 was established from the appropriate stress-strain relationship using the time at which the maximum pressure occurred. These actual values compared very closely to those estimated by Equations 4-10, 4-11 and 4-12 which use R as a basis. The comparison of the ρ_1 values is shown in Table IV.

TABLE IV

COMPARISON OF ESTIMATED AND ACTUAL ρ_1 VALUES

Rate of Pour	Estimated ρ_1 Value	Time of Maximum Pressure	Actual ρ_1 Value
2	0	140	0.3
6	0.35	140	0.3
10	0.8	110	0.6
14	1.25	90	0.6
18	1.45	70	1.3

Varying Initial Set Times.—To understand the influence of the initial set alone, the wall height and width were kept at 12 feet and 12 inches, respectively. Both twenty different placement times increasing from zero to two hundred and five different rates of pour ranging from 2 to 18 feet per hour were analyzed. The results obtained for five initial set times (0, 10, 20, 30 and 40 min.) are shown in Table V. These pressures were plotted as a function of rate of pour for each of these initial set times in Figures 36 to 40. It was interesting to note that the CERA method takes into account this factor of initial set time. When the initial set time was zero the pressures developed by both the new and CERA methods were greater than that for the ACI method. On the other hand, for a large initial set time

TABLE V

CONCRETE PRESSURES WITH VARYING TIME OF INITIAL SET

RATE OF POUR (FT/HR)	ACI[1]		TIME OF INITIAL SET (MIN) 0	10	20	30	40
2	390	N.M.[2]	409	359	309	259	209
		CERA[3]	502	490	460	419	371
6	870	N.M.	1088	938	788	638	488
		CERA	916	879	790	665	522
10	1101	N.M.	1293	1189	1069	931	769
		CERA	1300	1268	1120	912	674
14	1251	N.M.	1336	1224	1099	956	794
		CERA	1380	1380	1380	1158	825
18	1400	N.M.	1408	1288	1155	1055	836
		CERA	1460	1460	1460	1405	976

All pressures in psf.
Wall Thickness = 12 in.
Wall Height = 12 ft.
Concrete Temperature = 75°F.

[1]American Concrete Institute.

[2]New Method.

[3]Civil Engineering Research Association.

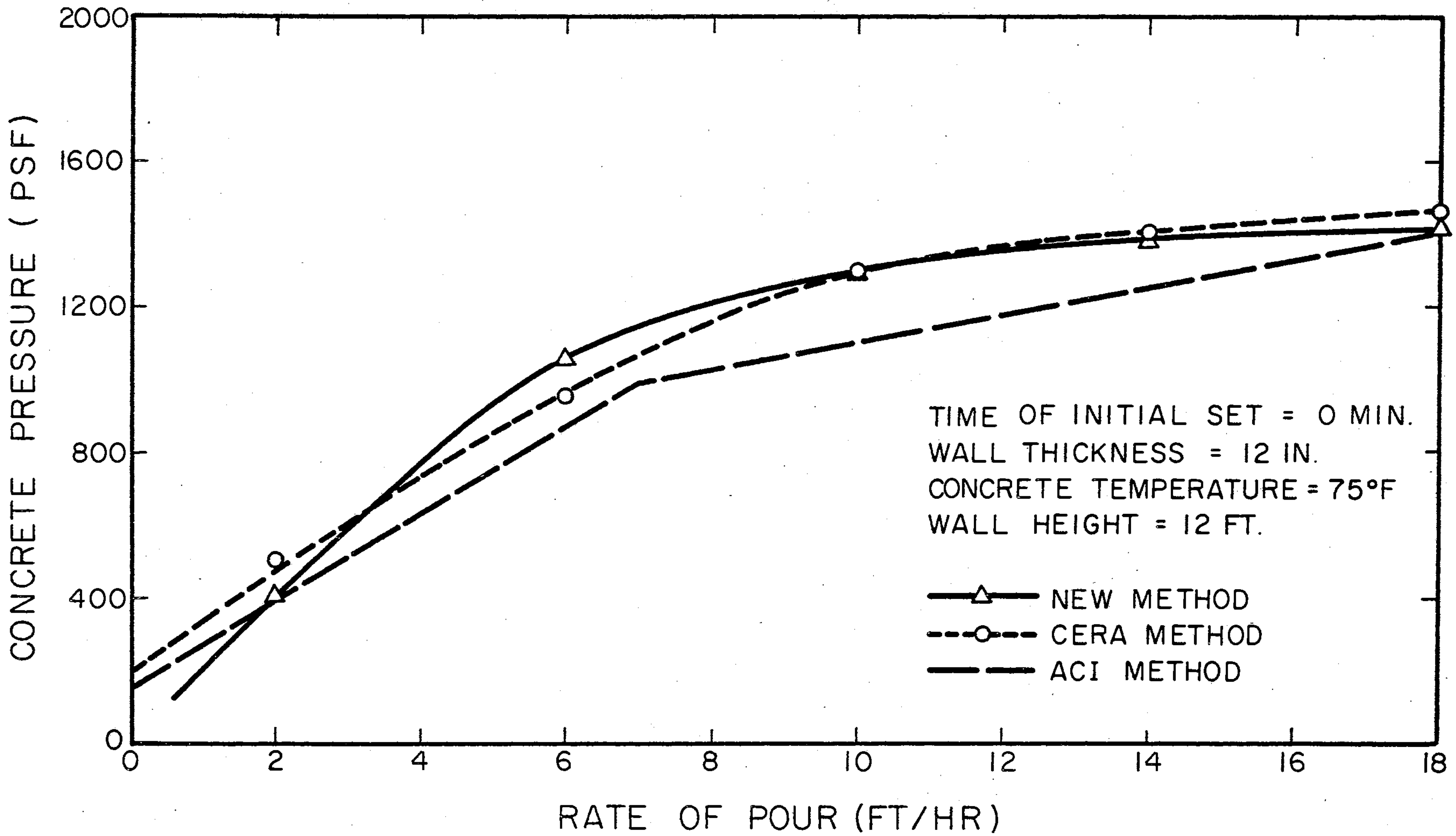

Figure 36. Concrete Pressure Versus Rate of Pour (T.I.S. = 0 MIN)

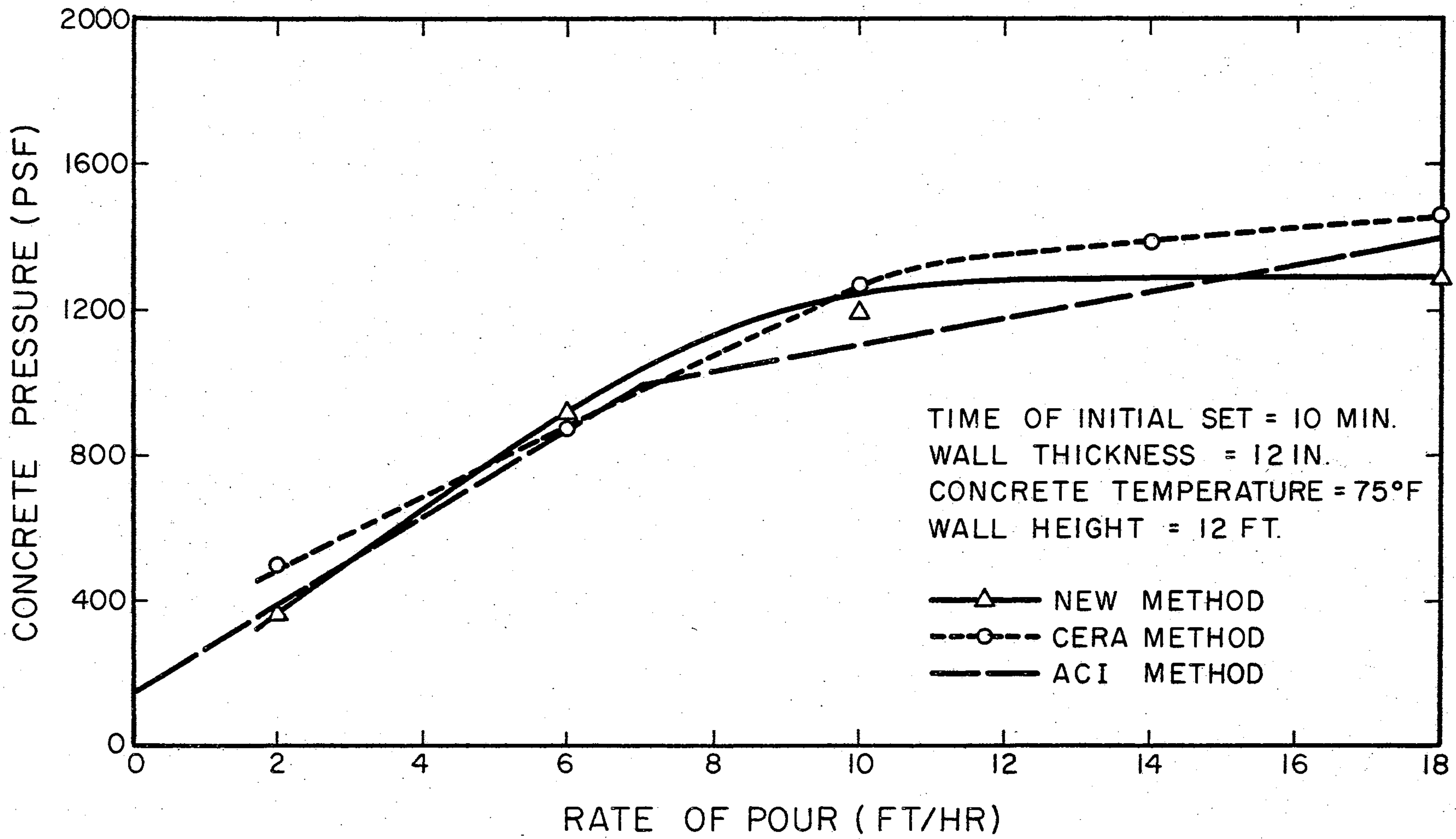

Figure 37. Concrete Pressure Versus Rate of Pour (T.I.S. = 10 MIN)

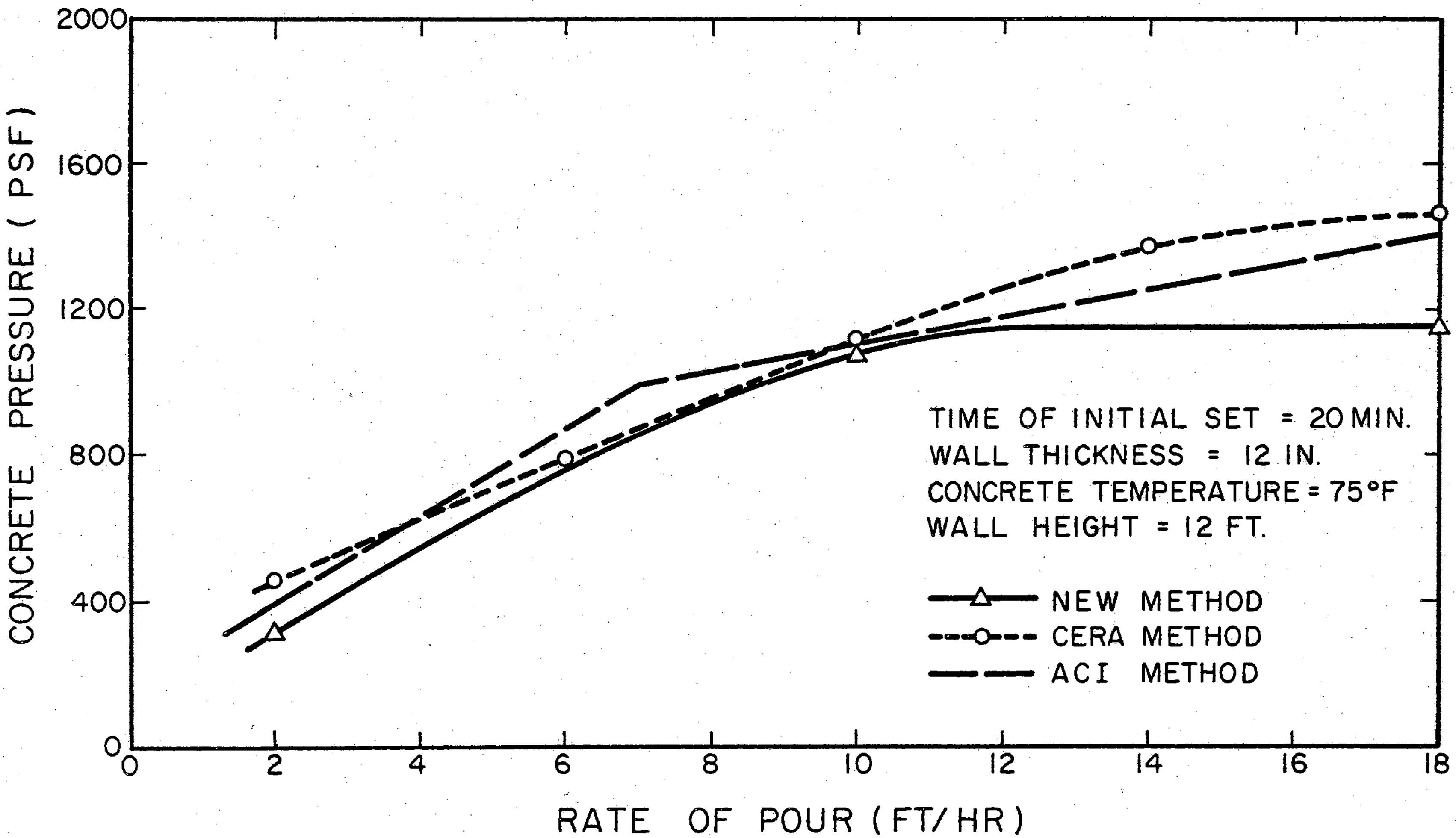

Figure 38. Concrete Pressure Versus Rate of Pour (T.I.S. = 20 MIN)

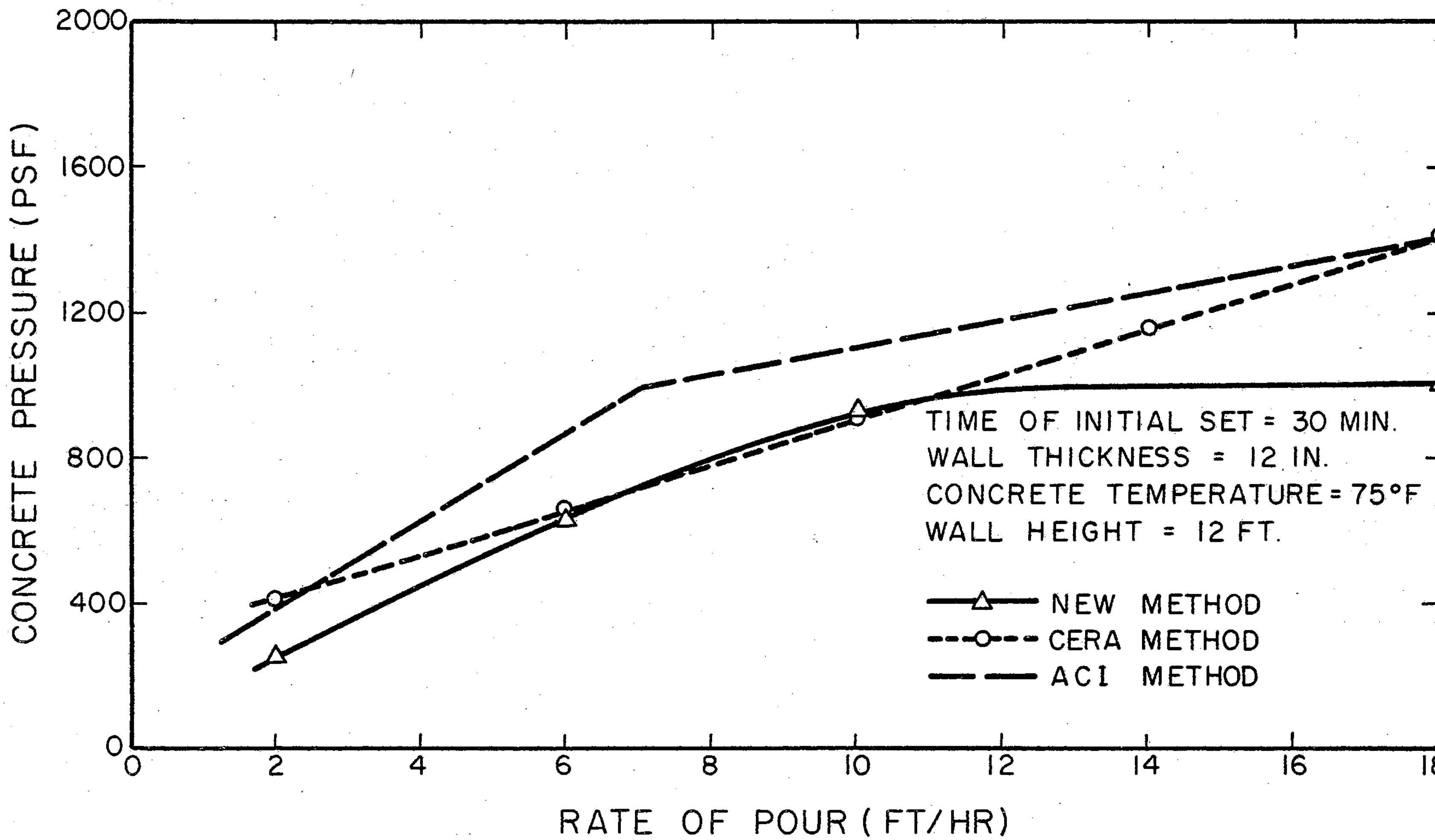

Figure 39. Concrete Pressure Versus Rate of Pour (T.I.S. = 30 MIN)

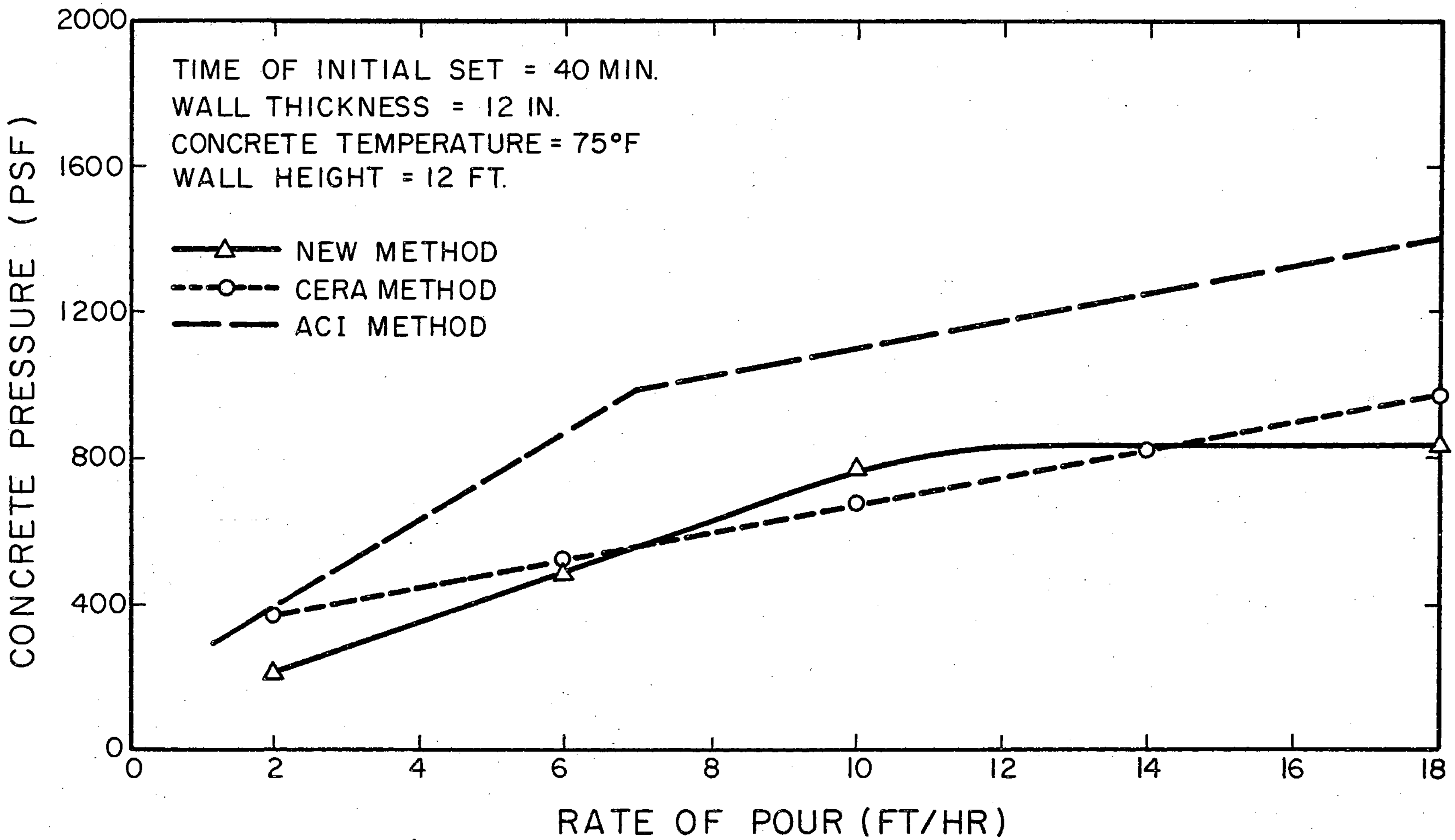

Figure 40. Concrete Pressure Versus Rate of Pour (T.I.S. = 40 MIN)

of 40 minutes, the pressures of the new and CERA methods were less than the ACI pressures. This is quite understandable for the longer the concrete is allowed to set before placement the less mobile it will become and subsequently the greater the shear strength. Also noted was the fact that the new method was in close agreement with the CERA method for almost all the initial set times except when the rate of pour was over 14 feet per hour in the 12 foot wall. This factor will be covered as part of the wall height discussion in the next section. It would seem that the ACI method was deficient in not considering the initial set time factor. However, this factor has indirectly been considered by ACI, for on most jobs, the minimum amount of initial set time is 20 minutes. On investigating the graph for 20 minutes initial set time (Figure 38), the ACI formula was seen to agree very well with the new and CERA methods. However, for higher initial set times (30 minutes) which are more common on concrete projects, the ACI method appears to overestimate the pressure by about 25 per cent for this factor alone.

Using the same results the pressures were then plotted versus initial set times for each of the rates of pour. The primary purpose in doing this was to establish the initial set time which gave the best correlation between the methods for each rate of pour. These plots are shown in Figures 41 through 45, inclusive. For rates of pour of six feet per hour (Figure 42) and ten feet per hour (Figure 43) very close correlation between the new and CERA methods was

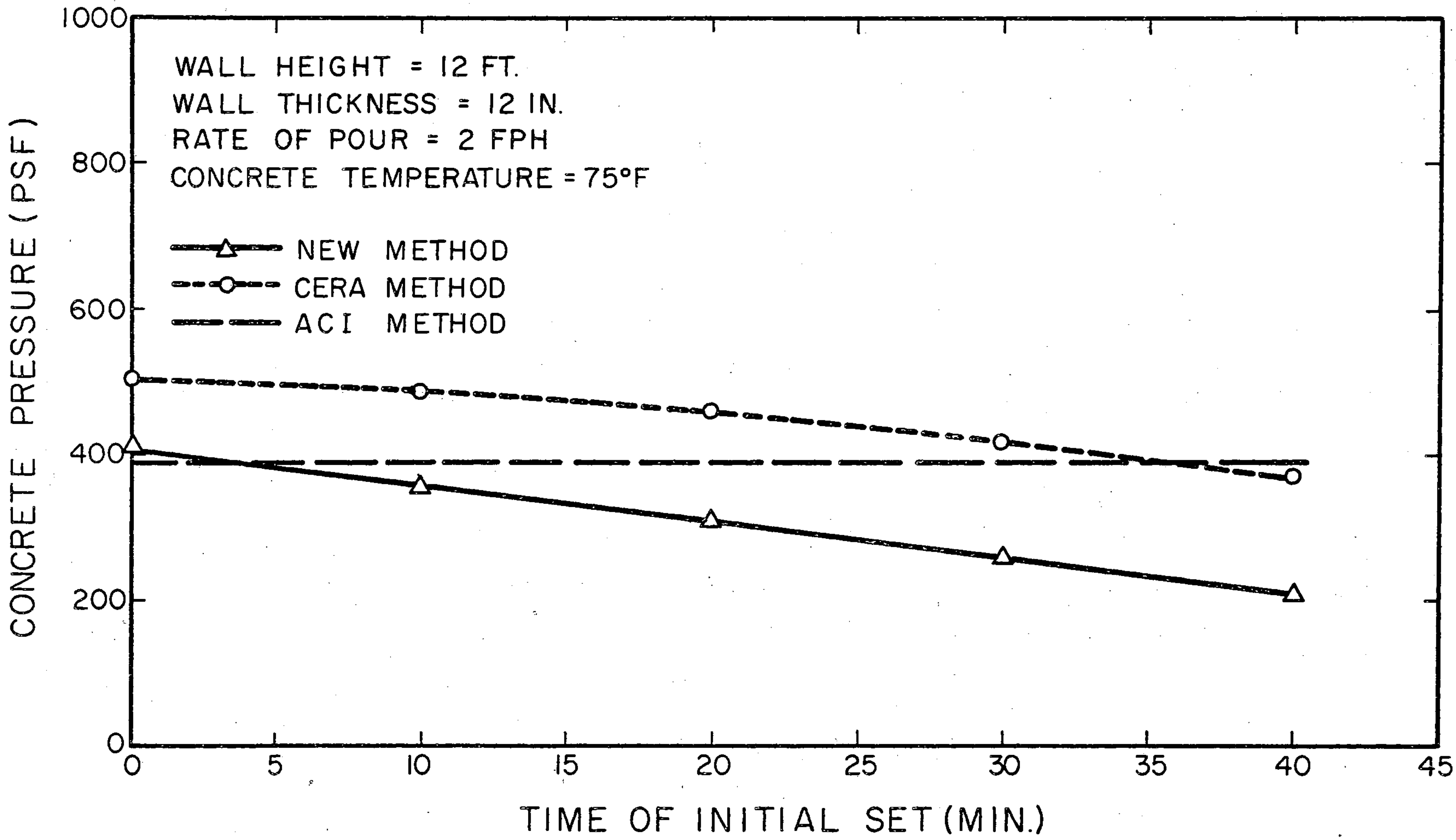

Figure 41. Concrete Pressure Versus Time of Initial Set (R = 2 FPH)

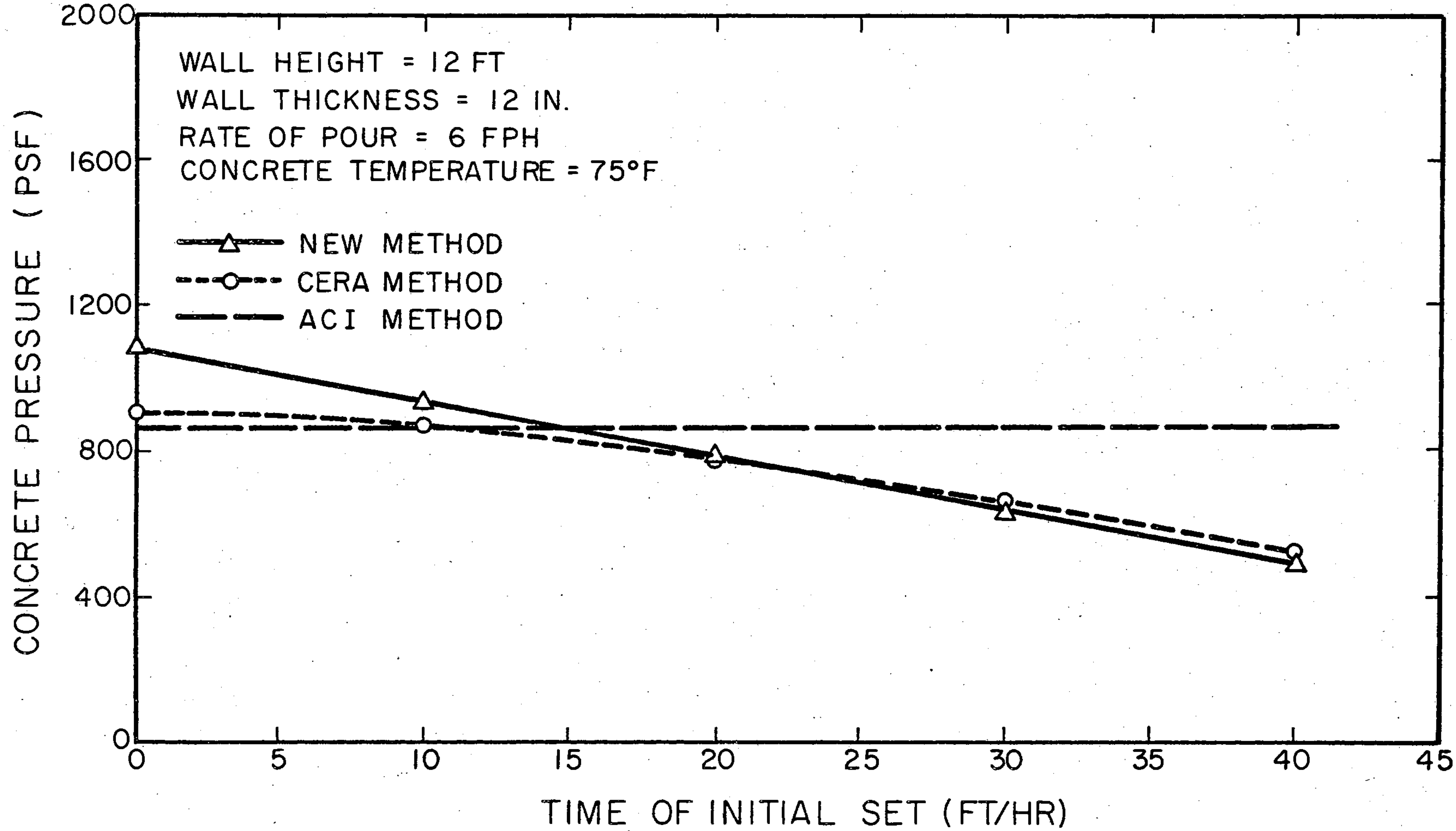

Figure 42. Concrete Pressure Versus Time of Initial Set (R = 6 FPH)

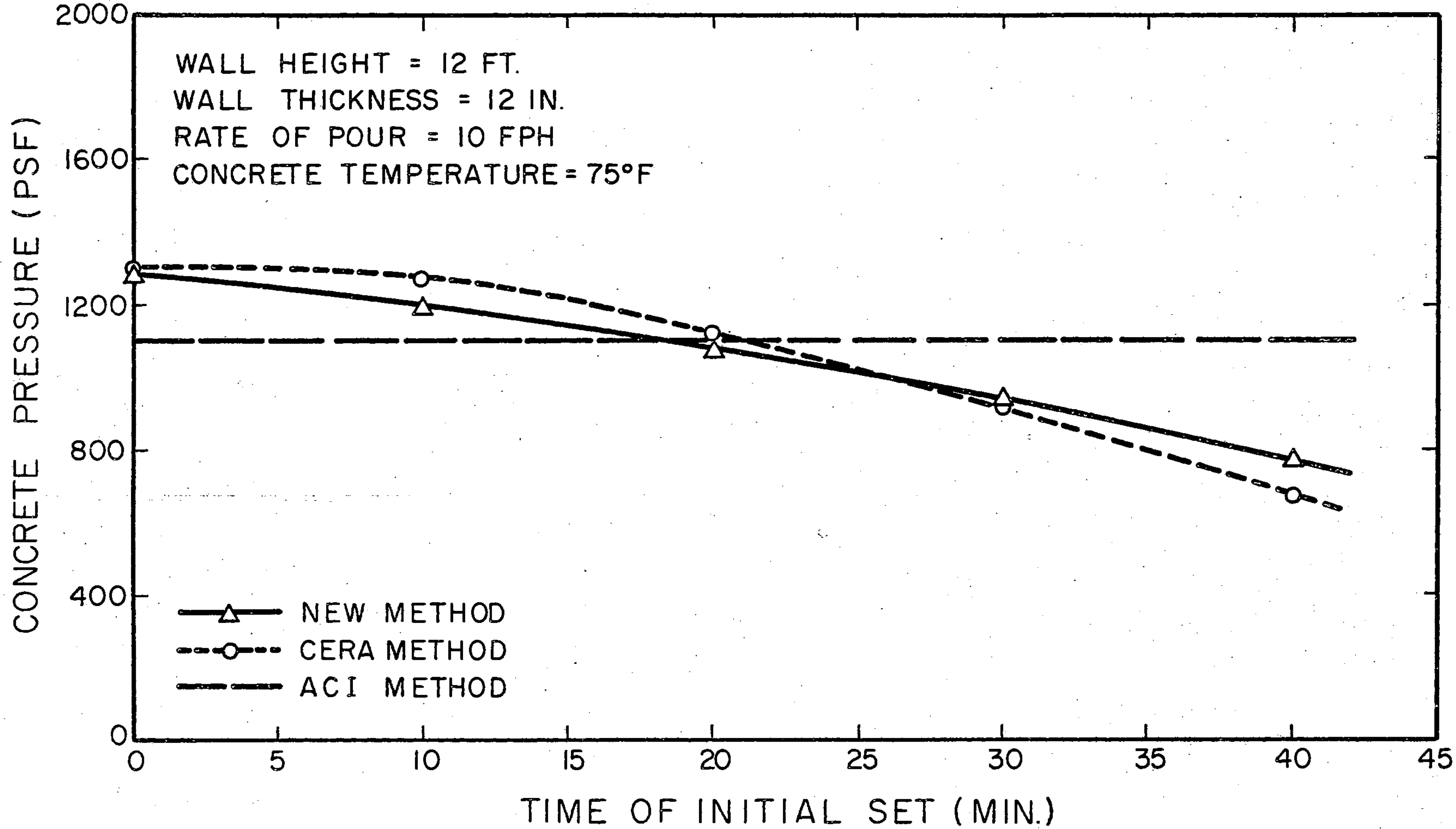

Figure 43. Concrete Pressure Versus Time of Initial Set (R = 10 FPH)

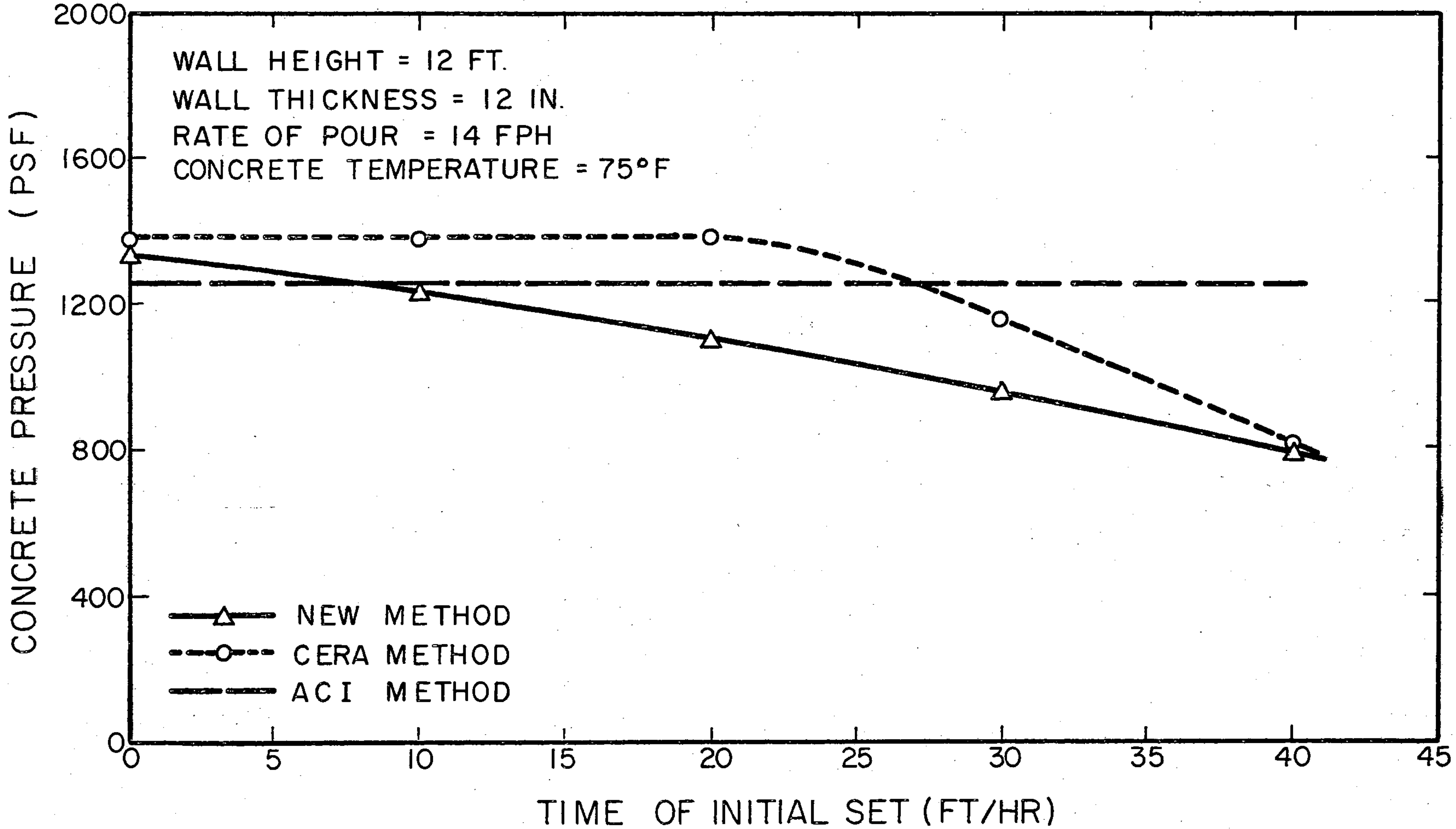

Figure 44. Concrete Pressure Versus Time of Initial Set (R = 14 FPH)

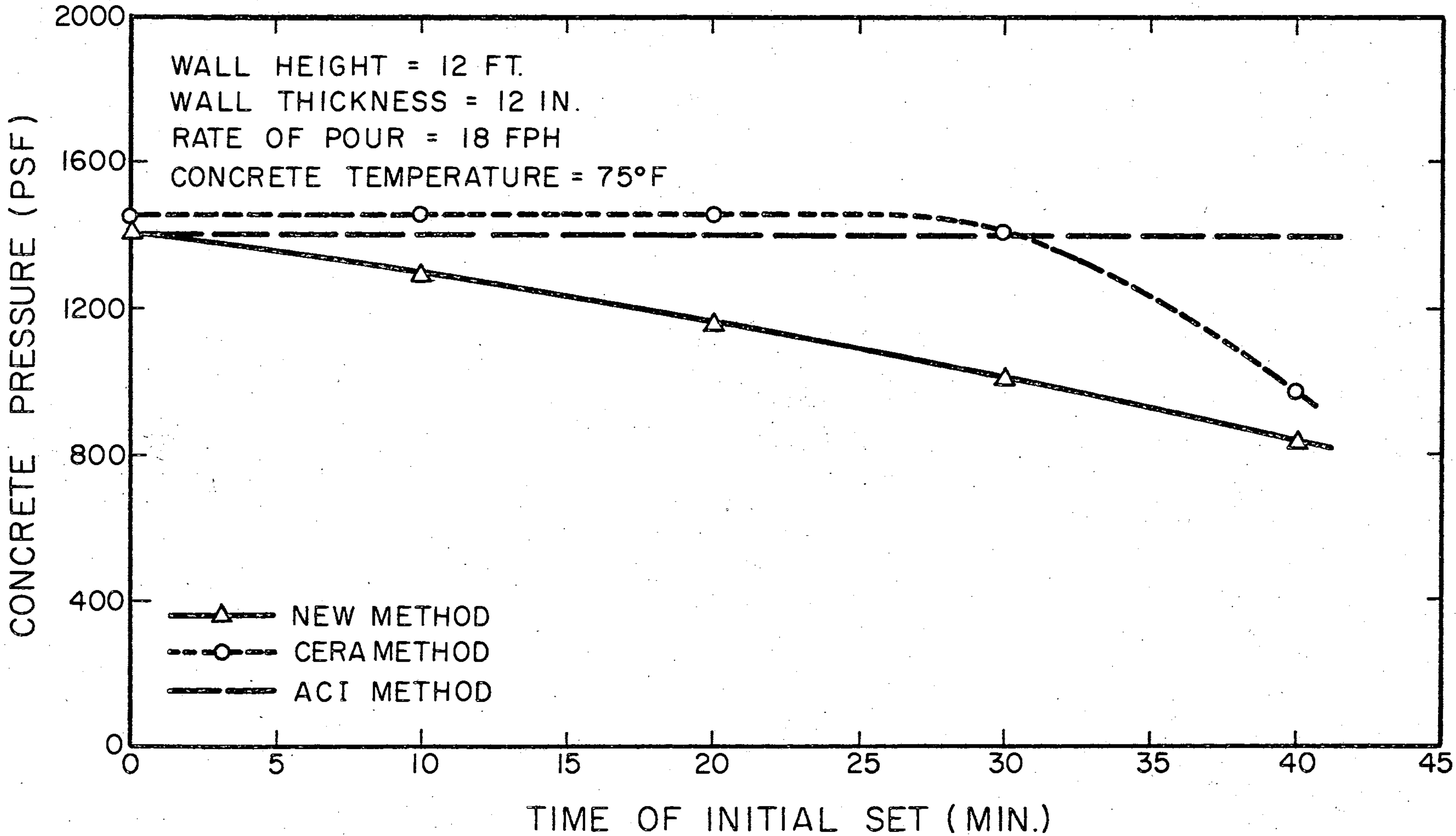

Figure 45. Concrete Pressure Versus Time of Initial Set (R = 18 FPH)

observed. This was particularly true for initial set times between 20 and 30 minutes. For this reason and that mentioned in the preceding paragraph, an initial set time of 20 minutes was chosen as being representative and was used in the analysis of the other two factors.

Varying Heights of Wall.—Modern methods of concrete placement have continually been improving and because of this the rates of pour in walls have steadily increased. Because of these higher rates of pour the pressures obtained using the ACI and CERA methods are sometimes greater than a concrete pressure with an equivalent fluid head (150 lbs. per cu. ft.). In these cases both ACI and CERA specify the equivalent fluid head of concrete to be used. However, they do not consider any shear strength developed during the initial set and placement time. To actually see how height affects the pressures five different heights were investigated (6, 10, 12, 16 and 20 feet). These were done for an initial set time of 20 minutes and a wall thickness of 12 inches. The rates of pour again ranged from 2 to 18 feet per hour and there were 20 placement times for each rate of pour. The results of the program are seen in Table VI. Figure 46 gives the pressures as a function of rate of pour for a six foot high wall. This plot is interesting in that the result using an equivalent fluid head of concrete gives 900 psf whereas CERA gives a value even higher due to their addition for vibration. Note that in this case the new method gives pressures on the order of 2/3 the hydrostatic

TABLE VI

CONCRETE PRESSURES WITH VARYING WALL HEIGHTS

RATE OF POUR (FT/HR)		HEIGHT OF WALL (FT) 6	10	12	16	20
2	ACI[1]	390	390	390	390	390
	N.M.[2]	309	309	309	309	309
	CERA[3]	460	460	460	460	460
6	ACI	870	870	870	870	870
	N.M.	574	788	788	788	788
	CERA	790	790	790	790	790
10	ACI	900	1101	1101	1101	1101
	N.M.	611	939	1069	1269	1344
	CERA	1042	1120	1120	1120	1120
14	ACI	900	1251	1251	1251	1251
	N.M.	611	939	1099	1444	1657
	CERA	1100	1320	1320	1320	1320
18	ACI	900	1400	1400	1400	1400
	N.M.	611	939	1155	1455	1843
	CERA	1100	1460	1460	1460	1460

All pressures in psf.
Wall Thickness = 12 in.
Time of Initial Set = 20 min.
Concrete Temperature = 75°F.

[1]American Concrete Institute. [2]New Method.

[3]Civil Engineering Research Association.

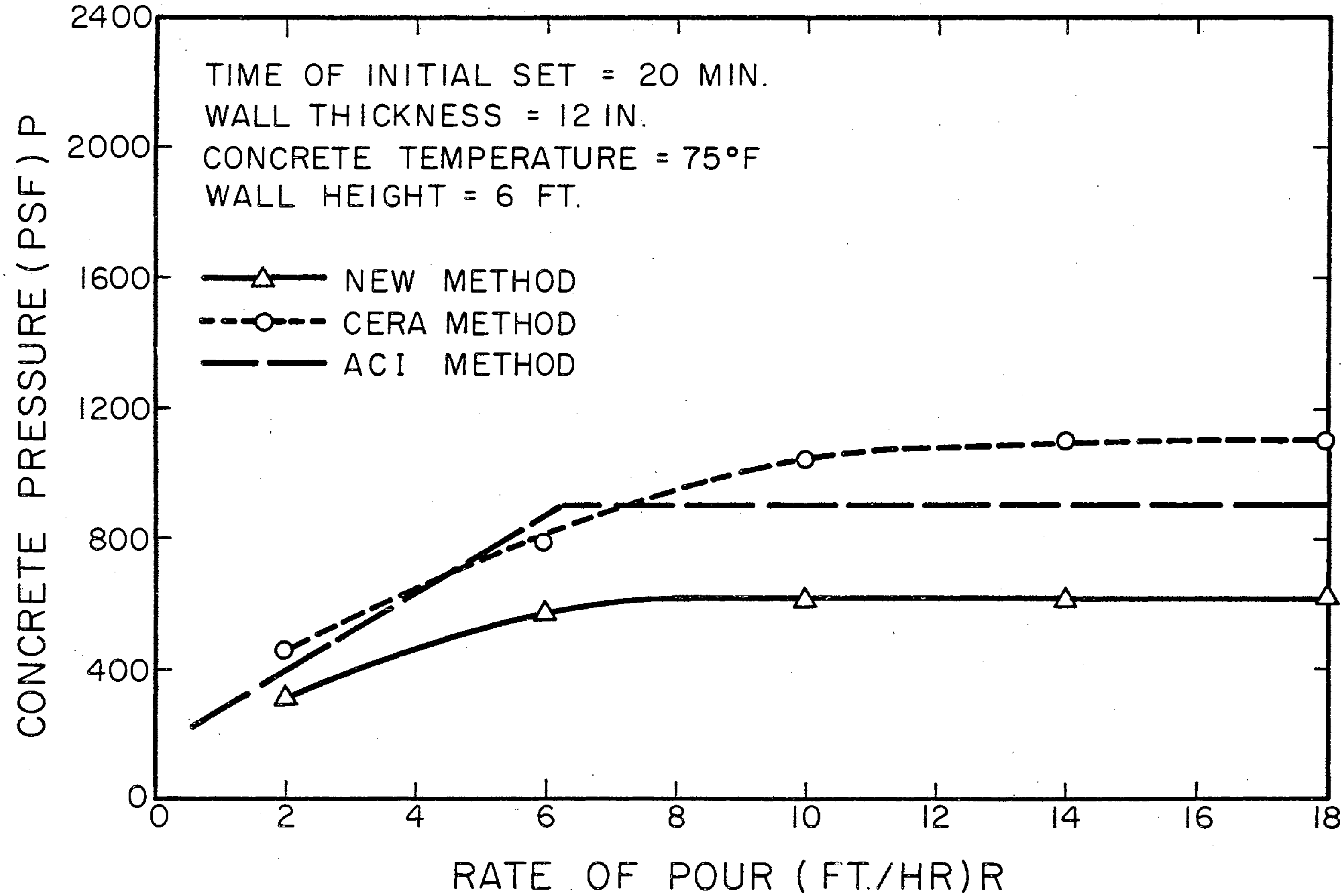

Figure 46. Concrete Pressure Versus Rate of Pour (H = 6 FT)

case which is due to the setting up of the concrete. On Figure 47, the pressures for heights of 10, 12, 16 and 20 feet are all plotted as a function of rate of pour for all the methods. In the CERA and ACI methods no distinction is made for the different wall heights but the pressures of the new method differ decidedly for high rates of pour. For rates of pour up to eight feet per hour, all three methods give very similar results. Above eight feet per hour, the height begins to influence the pressures obtained by the new method and this influence becomes greater as the rate of pour increases. For instance, the pressure difference between a ten foot and a twenty foot high wall at a rate of pour of ten feet per hour is 42 per cent, whereas at a rate of eighteen feet per hour it is 96 per cent. The reason for this variance stems from the combination of the maximum concrete head together with the shear strength in the coefficient (K) calculation in Equation 4-4. Notice that for very high walls (twenty feet), the concrete pressure is considerably higher than the pressures of the ACI and CERA methods. This is caused by the assumption of twenty minutes of initial set time, whereas the actual is more like thirty minutes. If thirty minutes is used then the resulting pressure for the twenty foot wall is much closer to ACI for high rates of pour. In this case, both the pressures of the new and CERA methods are lower than the ACI pressures for rates of pour up to ten feet per hour.

Varying Widths of Wall.—It was originally assumed in

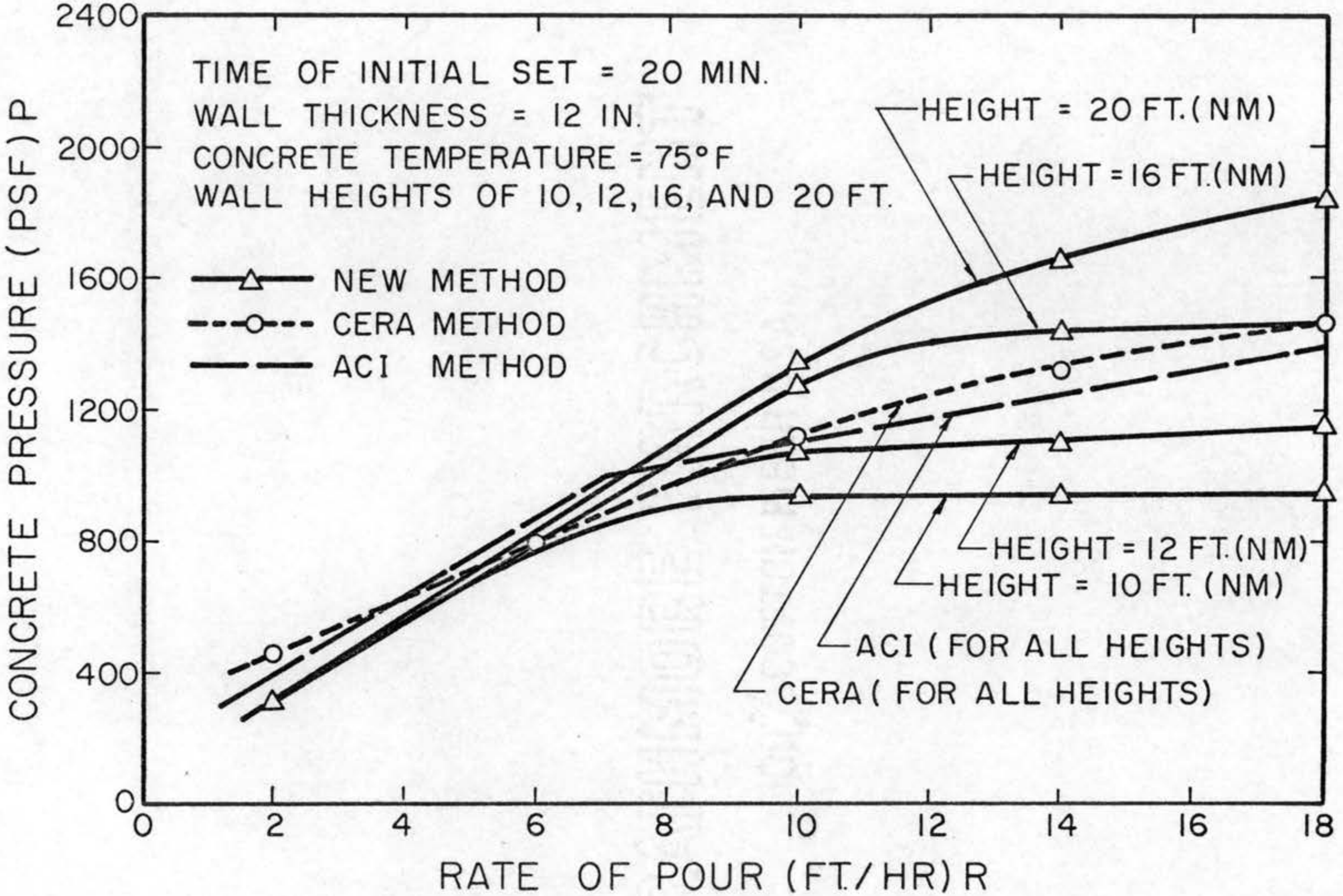

Figure 47. Concrete Pressure Versus Rate of Pour (H = 10, 12, 16 and 20 FT)

the new method that the width of wall was influential in determining the concrete pressure. This was related to the amount of shear strength that was mobilized. When all other factors are held constant, the same form face deflection causes greater concrete deformation in a narrow wall than a wide one. This in turn produces a larger amount of vertical strain and thus a higher shear strength. The CERA method also considers the wall thickness to be influential; however, it denotes arching between the form faces as the cause. Because of this the influence on the pressure is somewhat different in the new and CERA methods. As mentioned earlier, ACI does not consider the wall thickness at all. In Table VII, a summary for wall widths of 8, 12 and 16 inches is given. The initial set time is twenty minutes and the wall height, sixteen feet. The rates of pour again went from two to eighteen feet per hour. In Figure 48, the same results are shown graphically. As seen in this Figure, it is not until a rate of pour of nine feet per hour has been reached that CERA makes any allowance for wall thickness.

The selection of pressure in the CERA method depends on two criteria, stiffening and arching (see Equations 1-8 and 1-9), and they are not integrated. Therefore, it is only when the arching criteria governs that the width influences the pressure. The new method considers width influence for all rates of pour. The relative magnitude of pressures obtained from all three methods agrees very well when considering twenty minutes as the initial set time.

TABLE VII

CONCRETE PRESSURES WITH VARYING WALL THICKNESSES

RATE OF POUR (FT/HR)			WIDTH OF WALL (IN.)		
			8	12	16
	2	ACI[1]	390	390	390
		N.M.[2]	352	309	264
		CERA[3]	460	460	460
	6	ACI	870	870	870
		N.M.	686	788	889
		CERA	790	790	790
	10	ACI	1101	1101	1101
		N.M.	1096	1269	1383
		CERA	1100	1120	1120
	14	ACI	1251	1251	1251
		N.M.	1287	1444	1584
		CERA	1180	1380	1449
	18	ACI	1400	1400	1400
		N.M.	1345	1455	1584
		CERA	1260	1460	1660

All pressures in psf.
Wall Height = 16 ft.
Time of Initial Set = 20 min.
Concrete Temperature = 75°F.

[1] American Concrete Institute.

[2] New Method.

[3] Civil Engineering Research Association.

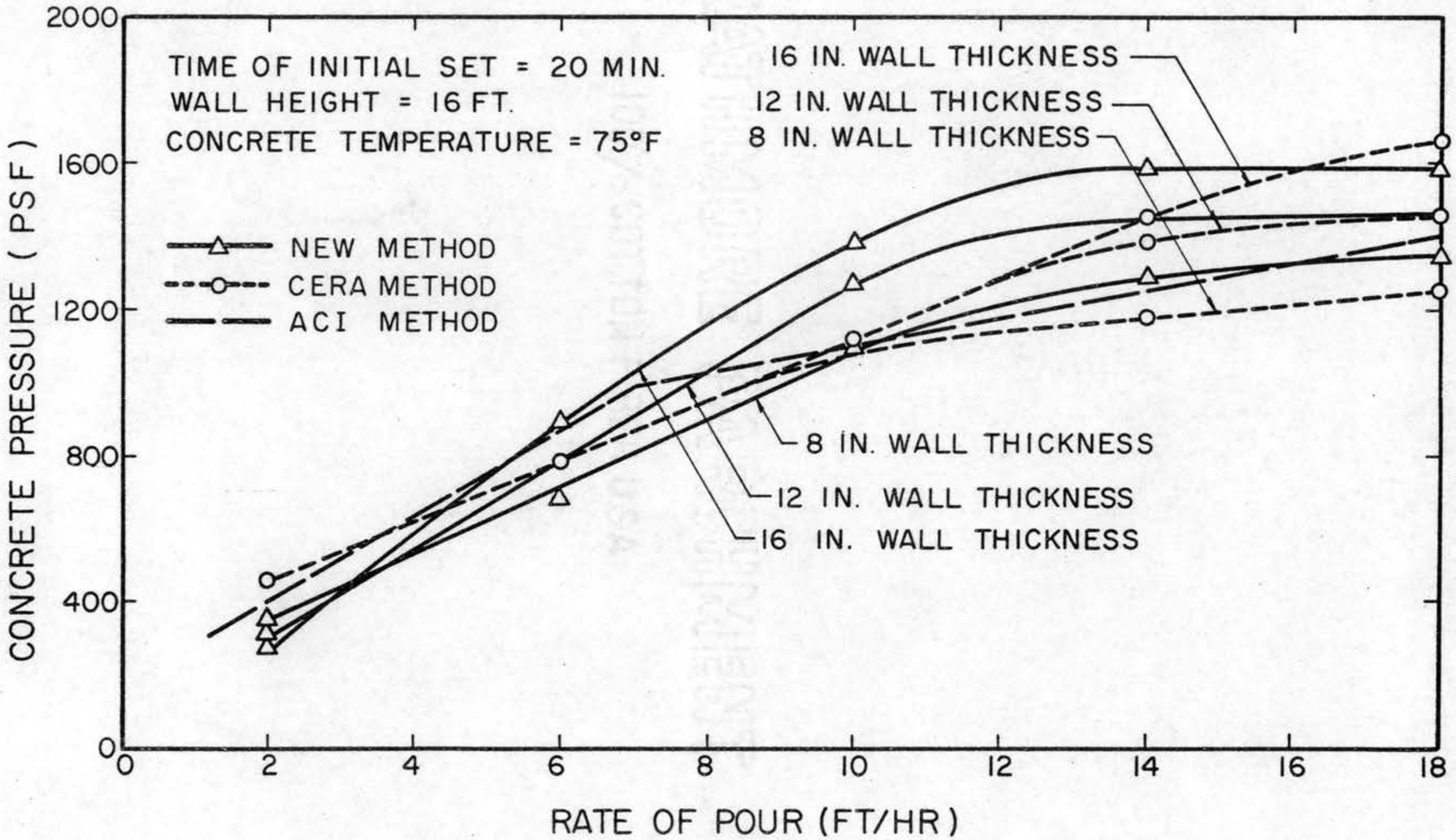

Figure 48. Concrete Pressure Versus Rate of Pour (W = 8, 10, and 12 IN)

At the rate of two feet per hour, an error may have been thought to occur because the eight inch wall gives a higher pressure than a wider wall, but this is due to the vibration intensification factor (Equation 4-13) in narrow walls being proportionally greater than that caused by the rate of pour.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

This study has centered on the shear strength variation of concrete and its relationship to lateral pressures. The shear strength was evaluated through the use of a triaxial testing arrangement. The triaxial tests were run at different concrete set times and a shear strength-set time relationship was established. Only one concrete mix and test temperature was used so that the factor of shear strength could be isolated and its effect on concrete pressures established. Under these conditions the pressures obtained by this new method were compared with pressures that were computed using methods recommended by two noted organizations, the Civil Engineering Research Association (CERA) of England and the American Concrete Institute (ACI).

5.1 Conclusions

From the results of this study the following conclusions can be drawn:

1) The shear strength parameters of concrete are a linear function of set time.
2) The Mohr-Coulomb Rupture Theory does apply to concrete while it is setting.

3) The shear strength of the concrete can be related to the pressures of concrete on formwork providing the boundary conditions associated with are considered.

4) With the factors they consider, both the ACI and CERA methods for determining the concrete pressures are reasonably accurate; however, in some areas they both are overly conservative (i.e. in some cases the assumption of the equivalent fluid height of concrete with no consideration of any shear strength which has developed during the placing operation).

5.2 Recommendations

The method used to evaluate the concrete pressures has been primarily based on a theoretical analysis. To validate the results it is suggested that prototype walls be erected and a set of full scale tests be performed. Then the actual deflections and initial set times can be recorded and the validity of this correlation can be further verified. After this has been established further triaxial tests can be performed with various mix designs and temperatures in order to make this new method more complete.

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APPENDIX A

DATA AND RESULTS OF THE INDIVIDUAL TRIAXIAL TESTS

DATE	TEST NO.	CHAMBER PRESSURE PSI	SET TIME MIN.	TEMP. °F	DEVIATOR STRESS PSI	ASSOCIATED STRAIN %	OVERALL NUMBER
1	2	3	4	5	6	7	8
1/25/67	1	40	60	80	18.87	15	1
	2	40	90	80	13.27	7	2
	3	40	120	80	17.60	4	3
1/26/67	1	20	60	74	16.10	19	4
	2	20	90	74	13.61	8	5
	3	20	120	74	17.09	5	6
	4	60	60	76	15.73	17	7
	5	60	90	74	22.89	6	8
	6	60	120	74	14.68	7	9
	7	80	60	74	18.08	13	10
	8	80	90	73	20.57	8	11
	9	80	120	73	17.33	7	12
1/27/67	1	60	60	78	12.97	8	13
	2	60	90	77	18.04	5	14
	3	60	120	76	20.40	3	15
	4	80	60	80	16.84	9	16
	5	80	90	79	24.74	7	17
	6	80	120	78	32.83	4	18
1/28/67	1	20	60	81	15.96	15	19
	2	20	90	81	13.20	10	20
	3	20	120	80	15.66	5	21
	4	40	60	82	28.58	17	22
	5	40	90	82	16.28	8	23
	6	40	120	82	18.43	6	24
	7	60	60	83	18.71	12	25
	8	60	90	83	15.16	9	26
	9	60	120	83	17.76	6	27
1/29/67	1	40	60	86	11.07	12	28
	2	40	90	86	42.83	11	29
	3	40	120	86	30.54	4	30
2/2/67	1	20	30	72	8.44	18	31
	2	20	45	72	7.81	19	32
	3	20	75	71	5.72	19	33
	4	20	90	74	10.09	18	34
	5	20	120	74	13.23	10	35
	6	20	180	74	36.52	19	36
	7	40	30	75	14.49	12	37
	8	40	45	75	10.81	19	38
	9	40	75	75	26.98	19	39
2/3/67	1	60	30	74	13.85	9	40
	2	60	45	74	11.40	16	41
	3	60	60	74	38.60	11	42
	4	80	30	78	13.07	9	43
	5	80	45	78	10.43	13	44
	6	80	75	78	32.97	19	45
	7	40	90	79	15.15	7	46
	8	40	120	79	13.59	8	47
	9	40	180	78	26.38	5	48
	10	40	30	80	12.25	11	49
	11	40	45	80	9.87	13	50
	12	40	60	80	30.59	19	51
2/4/67	1	60	30	79	16.68	7	52
	2	60	45	79	10.81	13	53
	3	60	180	79	79.27	16	54
2/7/67	1	60	30	68	13.78	15	55
	2	60	45	67	11.49	18	56
	3	60	120	67	49.10	15	57
	4	80	75	72	15.80	9	58
	5	80	90	72	15.19	12	59
	6	80	120	72	38.39	18	60
	7	20	30	74	11.78	15	61
	8	20	45	73	8.28	18	62
	9	20	180	74	30.27	17	63
	10	20	75	73	13.99	17	64
	11	20	90	75	12.24	11	65
	12	20	180	75	31.48	4	66
	13	80	30	75	15.19	19	67
	14	80	45	75	7.56	19	68
	15	60	180	74	89.14	9	69
2/8/67	1	20	30	76	9.17	19	70
	2	20	45	76	10.49	19	71
	3	20	60	76	18.09	18	72
	4	80	45	78	16.94	12	73
	5	60	75	78	16.08	7	74
	6	80	180	77	-----	--	75
	7	40	30	74	12.38	18	76
	8	40	45	74	8.52	19	77
	9	40	75	74	24.37	19	78

1	2	3	4	5	6	7	8
2/9/67	1	80	45	70	17.54	11	79
	2	60	60	69	13.32	17	80
	3	60	75	69	25.12	15	81
	4	40	30	78	13.61	12	82
	5	40	45	78	13.04	17	83
	6	80	180	78	58.43	8	84
	7	80	30	79	10.62	18	85
	8	80	75	79	13.23	11	86
	9	80	90	79	24.65	16	87
	10	60	30	80	9.18	19	88
	11	60	45	80	11.66	13	89
	12	60	75	80	31.07	19	90
	13	80	30	80	14.15	17	91
	14	80	180	80	27.78	2	92
	15	80	180	80	29.17	2	93
2/10/67	1	20	30	74	11.53	19	94
	2	40	180	75	22.81	3	95
	3	40	180	75	23.53	3	96
	4	40	180	79	23.67	2	97
	5	60	180	79	43.80	6	98
	6	60	180	79	39.00	1	99
	7	20	30	78	10.96	19	100
	8	20	45	78	12.22	18	101
	9	20	75	79	17.44	7	102
2/11/67	1	80	180	75	22.71	3	103
	2	60	180	76	19.94	1	104
	3	80	180	77	22.04	3	105
	4	60	75	75	25.22	10	106
	5	60	90	76	21.27	5	107
	6	60	120	76	23.76	4	108
2/13/67	1	40	75	83	13.61	9	109
	2	40	90	83	22.44	9	110
	3	40	120	84	28.03	5	111
	4	80	30	80	8.21	19	112
	5	80	45	80	12.58	16	113
	6	80	60	80	14.45	9	114
	7	60	180	80	39.07	3	115
	8	40	180	80	24.75	3	116
	9	20	180	80	36.97	3	117
2/14/67	1	80	75	73	12.94	18	118
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	3	80	120	75	20.92	6	120
	4	20	120	76	21.88	7	121
	5	20	180	77	31.59	4	122
	6	80	180	77	31.31	3	123
	7	40	75	80	17.07	13	124
	8	40	90	80	17.73	6	125
	9	40	120	79	21.84	5	126
	10	60	60	79	8.75	18	127
	11	60	75	80	12.09	9	128
	12	60	90	80	15.90	11	129
	13	60	180	80	29.72	2	130
	14	40	180	80	30.89	2	131
	15	30	180	80	23.90	2	132
2/15/67	1	60	120	76	21.06	3	133
	2	80	180	75	29.68	2	134
	3	40	30	72	7.70	12	135
	4	40	45	72	5.06	19	136
	5	40	60	73	8.41	19	137
	6	20	30	75	5.48	18	138
	7	20	45	75	6.25	19	139
	8	20	60	76	8.39	19	140
	9	20	180	75	30.08	5	141
	10	40	180	75	35.05	5	142
	11	60	180	75	41.03	3	143
2/16/67	1	60	30	71	8.72	19	144
	2	60	45	72	8.26	19	145
	3	60	60	71	13.34	14	146
	4	40	60	71	13.07	17	147
	5	40	75	74	11.06	18	148
	6	40	90	72	12.10	9	149
	7	80	75	72	12.41	15	150
	8	80	90	71	28.75	19	151
	9	80	120	72	23.26	9	152
	10	20	60	75	8.58	19	153
	11	20	75	75	12.42	17	154
	12	60	90	74	24.63	18	155
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	2	20	90	77	14.14	8	157
	3	20	120	77	20.67	6	158
2/18/67	1	80	60	78	11.65	6	159
	2	80	75	78	28.35	11	160
	3	60	75	78	16.70	7	161
	4	80	30	83	9.04	19	162
	5	80	45	82	11.90	10	163
	6	80	60	82	17.41	9	164
2/23/67	1	40	60	73	10.99	18	165
	2	40	75	73	17.55	9	166
	3	40	90	72	23.91	8	167
	4	20	180	74	51.43	13	168
	5	60	180	74	31.18	3	169
	6	40	180	74	35.41	4	170
	7	80	75	72	15.71	9	171
	8	80	90	72	16.33	11	172
	9	80	120	72	28.59	11	173

1	2	3	4	5	6	7	8
2/24/67	1	40	180	74	34.22	3	174
	2	60	180	74	34.60	9	175
	3	80	180	76	29.74	3	176
	4	60	75	73	14.73	17	177
	5	60	90	73	14.86	8	178
	6	60	120	72	24.99	5	179
2/25/67	1	20	60	67	6.25	19	180
	2	20	75	67	12.21	19	181
	3	20	120	68	22.84	12	182
	4	20	60	73	7.48	18	183
	5	20	75	74	9.84	19	184
	6	20	90	74	12.52	11	185
	7	20	120	77	21.13	7	186
	8	20	180	76	25.58	5	187
	9	80	180	77	25.01	4	188
	10	60	60	76	10.28	17	189
	11	60	75	76	11.44	12	190
	12	40	120	76	14.70	8	191
2/27/67	1	20	45	75	7.07	19	192
	2	40	75	75	9.95	18	193
	3	40	120	74	18.20	8	194
2/28/67	1	80	30	75	8.96	18	195
	2	80	45	75	10.20	19	196
	3	80	60	75	11.41	17	197
	4	60	30	77	8.75	18	198
	5	40	45	77	10.69	17	199
	6	40	60	78	12.63	18	200
5/30/67	1	40	90	79	12.75	9	201
	2	40	120	80	21.21	6	202
	3	60	180	79	20.51	4	203
	4	60	75	77	13.35	14	204
	5	60	90	77	15.83	7	205
	6	60	120	77	17.32	4	206
	7	60	30	77	13.21	19	207
	8	40	45	76	11.94	17	208
5/31/67	1	80	30	81	12.22	13	209
	2	80	45	81	13.77	10	210
	3	80	75	81	14.57	7	211
	4	60	30	80	12.12	9	212
	5	60	60	80	15.26	9	213
	6	60	75	80	17.47	10	214
	7	40	60	83	14.41	17	215
	8	80	75	83	18.10	6	216
	9	80	120	83	26.85	4	217
	10	40	75	85	16.80	9	218
	11	80	90	85	20.29	3	219
	12	20	180	84	32.83	2	220
	13	40	60	83	12.65	5	221
	14	40	75	83	16.63	14	222
	15	60	90	83	21.15	6	223
	16	60	60	81	18.45	10	224
	17	80	90	82	17.39	8	225
	18	60	120	82	23.08	4	226
	19	40	30	80	8.67	19	227
	20	80	45	80	14.11	16	228
	21	20	60	80	13.39	18	229
6/1/67	1	80	20	80	9.64	16	230
	2	80	30	80	11.08	17	231
	3	80	120	79	23.51	8	232
	4	60	20	79	9.69	19	233
	5	60	30	79	12.77	19	234
	6	20	20	80	5.73	19	235
	7	20	30	80	5.96	19	236
	8	40	20	80	12.32	14	237
	9	40	30	80	11.33	19	238
6/2/67	1	60	20	76	8.63	19	239
	2	60	30	76	9.49	18	240
	3	80	20	76	8.80	19	241
	4	80	30	76	13.00	19	242
	5	40	20	76	7.39	19	243
	6	40	30	76	8.71	19	244
	7	20	20	77	6.09	19	245
	8	20	30	77	5.97	19	246
6/22/67	1	80	20	79	12.12	15	247
	2	80	45	79	15.22	12	248
	3	80	60	79	17.22	14	249
	4	40	20	80	17.23	9	250
	5	40	45	80	12.65	4	251
	6	40	75	80	18.59	6	252
6/23/67	1	40	60	77	11.74	16	253
	2	40	75	77	10.09	13	254
	3	40	90	78	13.58	14	255
	4	80	75	78	17.84	6	256
	5	80	90	79	14.74	5	257
	6	80	120	79	20.08	4	258
	7	60	20	79	9.99	19	259
	8	60	60	79	10.72	16	260
	9	60	75	79	14.33	13	261
	10	20	20	80	7.54	19	262
	11	20	30	79	8.94	19	263
	12	20	60	80	10.69	15	264
	13	60	120	80	22.39	5	265
	14	60	180	81	33.81	3	266
	15	60	180	81	26.41	2	267
	16	60	20	81	9.50	15	268
	17	60	75	81	19.10	14	269
	18	60	120	81	27.06	7	270

1	2	3	4	5	6	7	8
6/24/67	1	20	120	80	17.14	6	271
	2	20	180	80	37.90	7	272
	3	20	180	80	35.16	6	273
	4	40	20	82	9.88	15	274
	5	40	60	81	12.28	12	275
	6	40	75	82	13.39	9	276
6/26/67	1	80	20	74	8.84	19	277
	2	80	75	74	12.13	13	278
	3	80	120	74	19.23	10	279
	4	80	75	75	12.26	13	280
	5	80	90	75	12.11	15	281
	6	80	120	75	18.49	19	282
	7	20	20	76	7.92	19	283
	8	40	90	76	14.47	19	284
	9	20	120	76	14.01	19	285
	10	20	20	76	6.77	17	286
	11	20	60	76	8.82	18	287
	12	20	75	76	11.18	18	288
6/27/67	1	40	20	79	11.34	19	289
	2	40	60	80	12.55	8	290
	3	60	75	80	18.89	11	291
	4	60	20	80	11.86	19	292
	5	20	60	80	13.48	13	293
	6	60	75	80	14.18	12	294
6/28/67	1	80	20	78	12.88	7	295
	2	80	90	78	14.86	3	296
	3	20	120	78	15.50	4	297

APPENDIX B

REPRESENTATIVE STRESS VERSUS STRAIN CURVES
OF EIGHT CP-ST COMBINATIONS

CP (psi)	ST (minutes)	
20	20	(Figure B.1)
60	30	(Figure B.2)
20	45	(Figure B.3)
60	60	(Figure B.4)
20	75	(Figure B.5)
20	90	(Figure B.6)
60	120	(Figure B.7)
20	180	(Figure B.8)

Figures B.1, B.2, B.3, and B.5 are examples of CP-ST combinations where no individual tests were eliminated. Figures B.4, B.7 and B.8 all have one test eliminated because these tests were greater than the average deviator stress plus two standard deviations. Figure B.6 shows an example of a combination which had a test rejected because of improper testing technique. In this plot, test number 34 was disregarded because the proving ring was not in close contact with the triaxial piston. This resulted in a stress-strain curve which gave virtually no rise in stress for the initial values of strain. On the plots the point of failure for each test is denoted by a point.

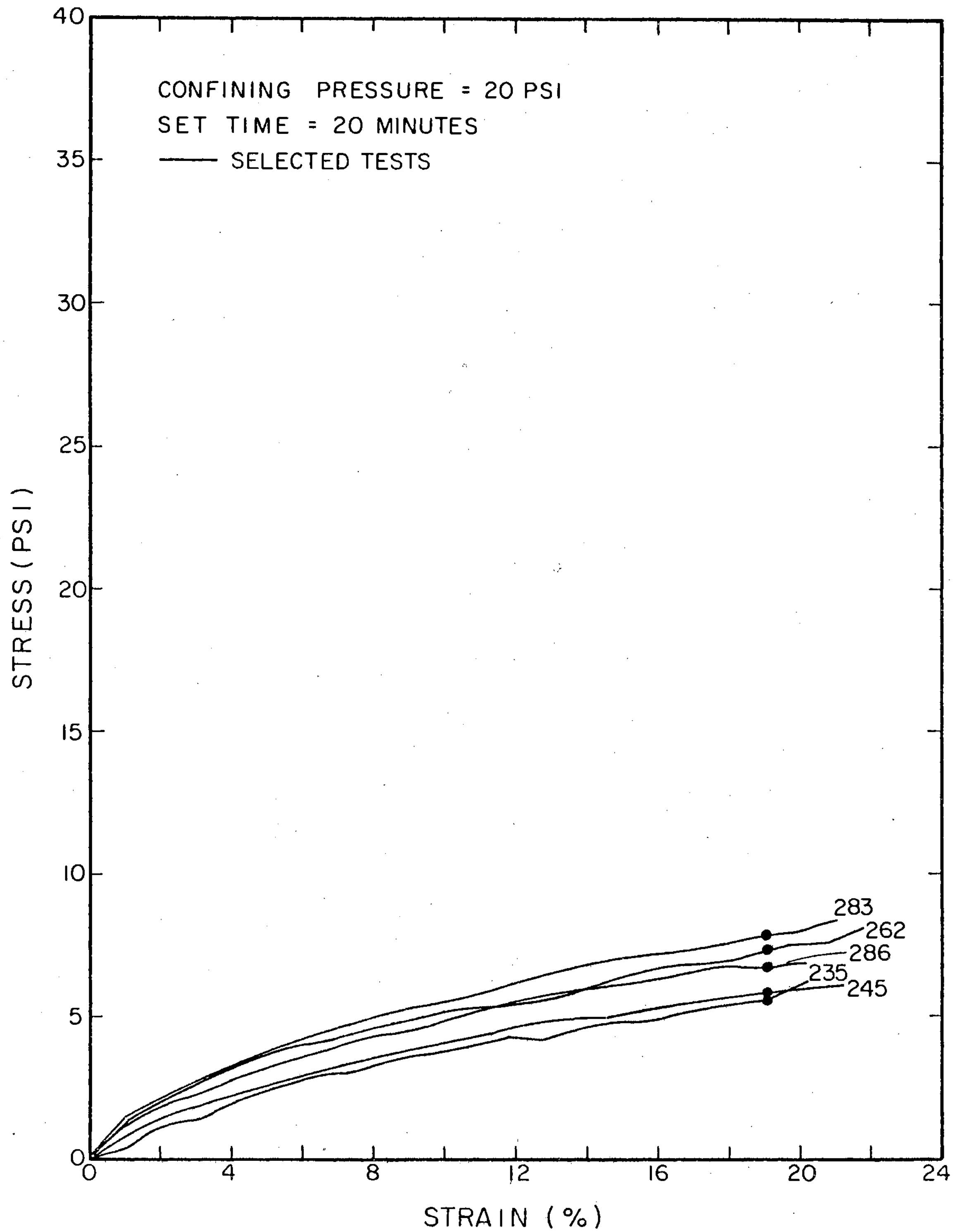

Figure B.1. Stress Versus Strain (CP = 20 PSI, ST = 20 MIN)

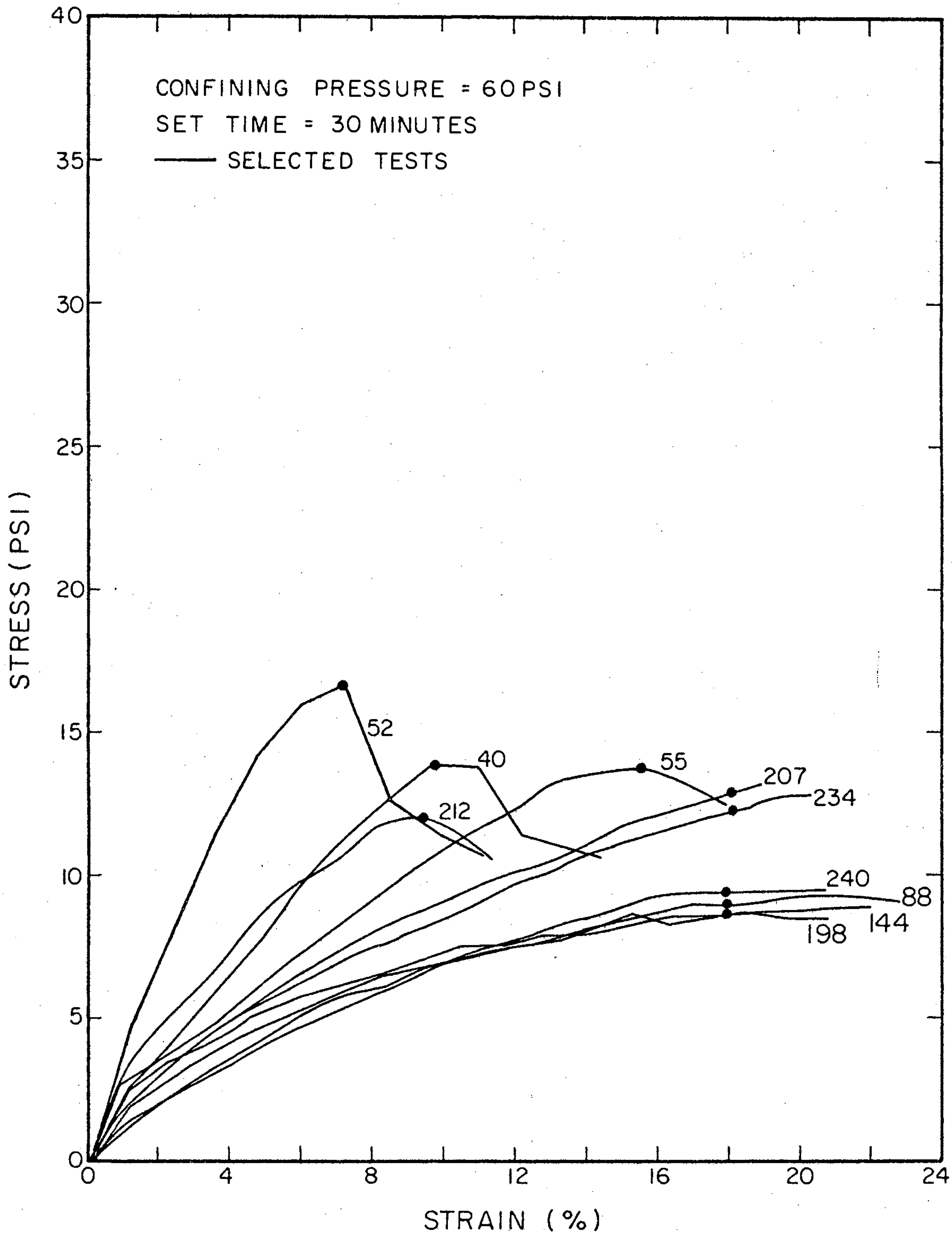

Figure B.2. Stress Versus Strain (CP = 60 PSI, ST = 30 MIN)

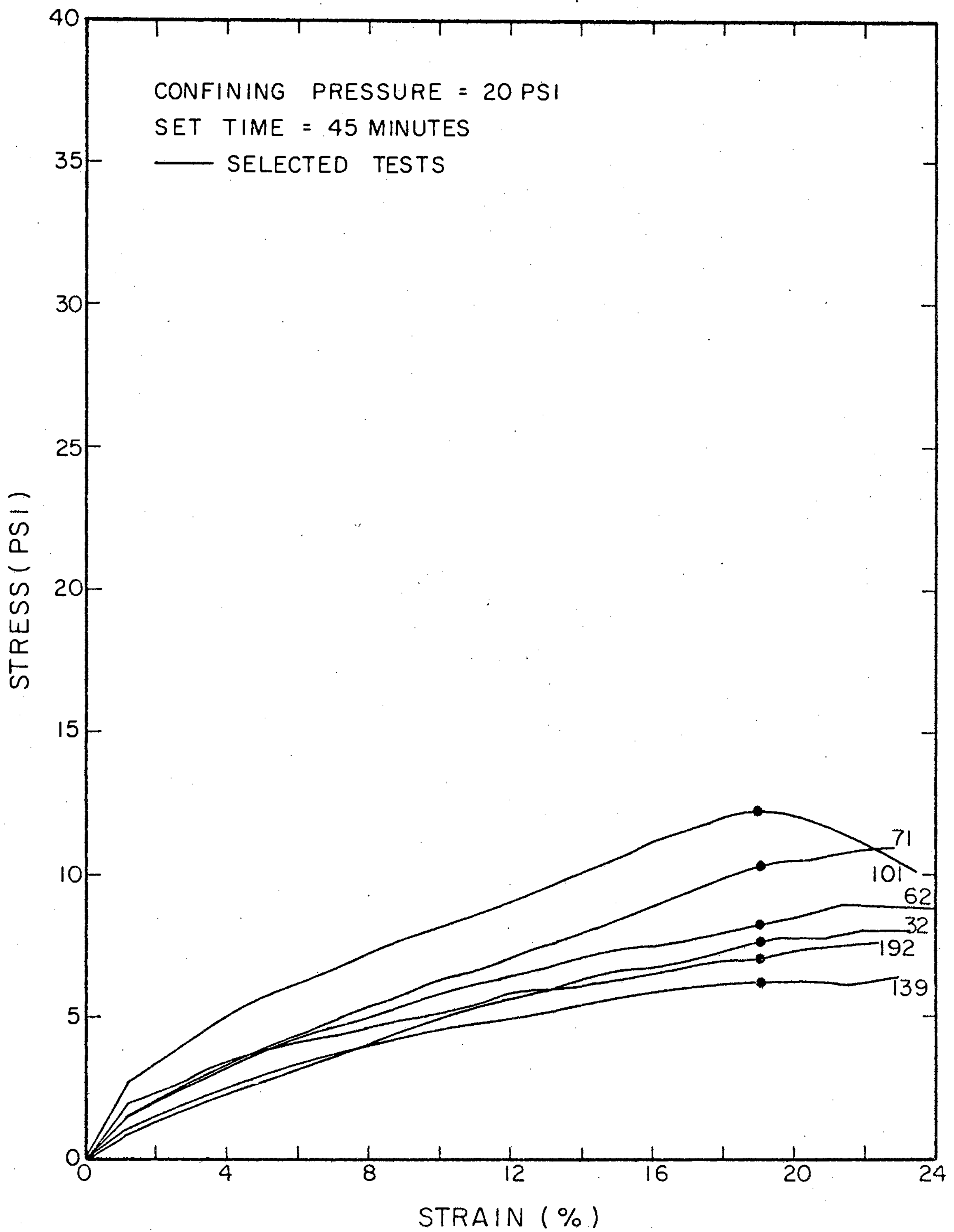

Figure B.3. Stress Versus Strain (CP = 20 PSI, ST = 45 MIN)

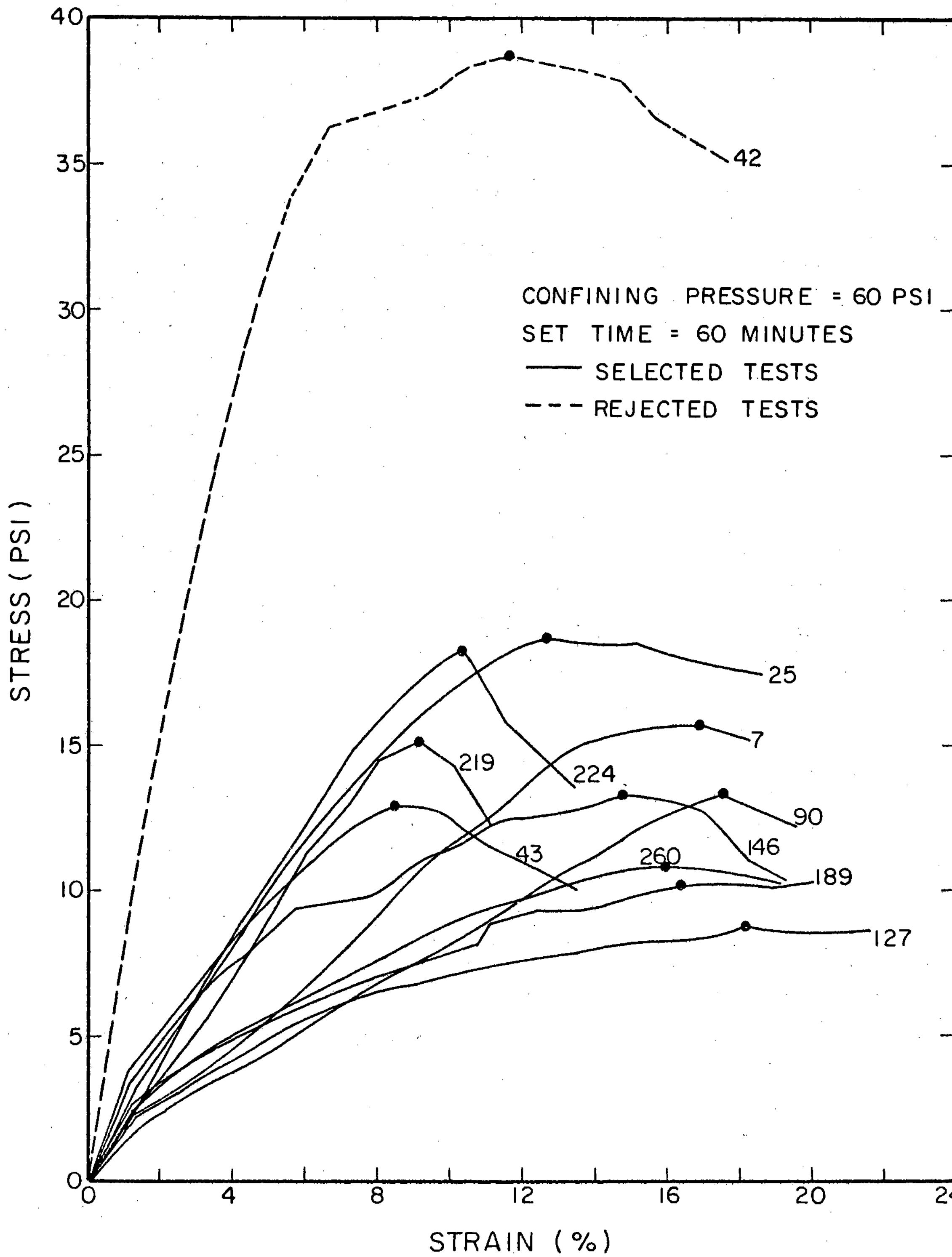

Figure B.4. Stress Versus Strain (CP = 60 PSI, ST = 60 MIN)

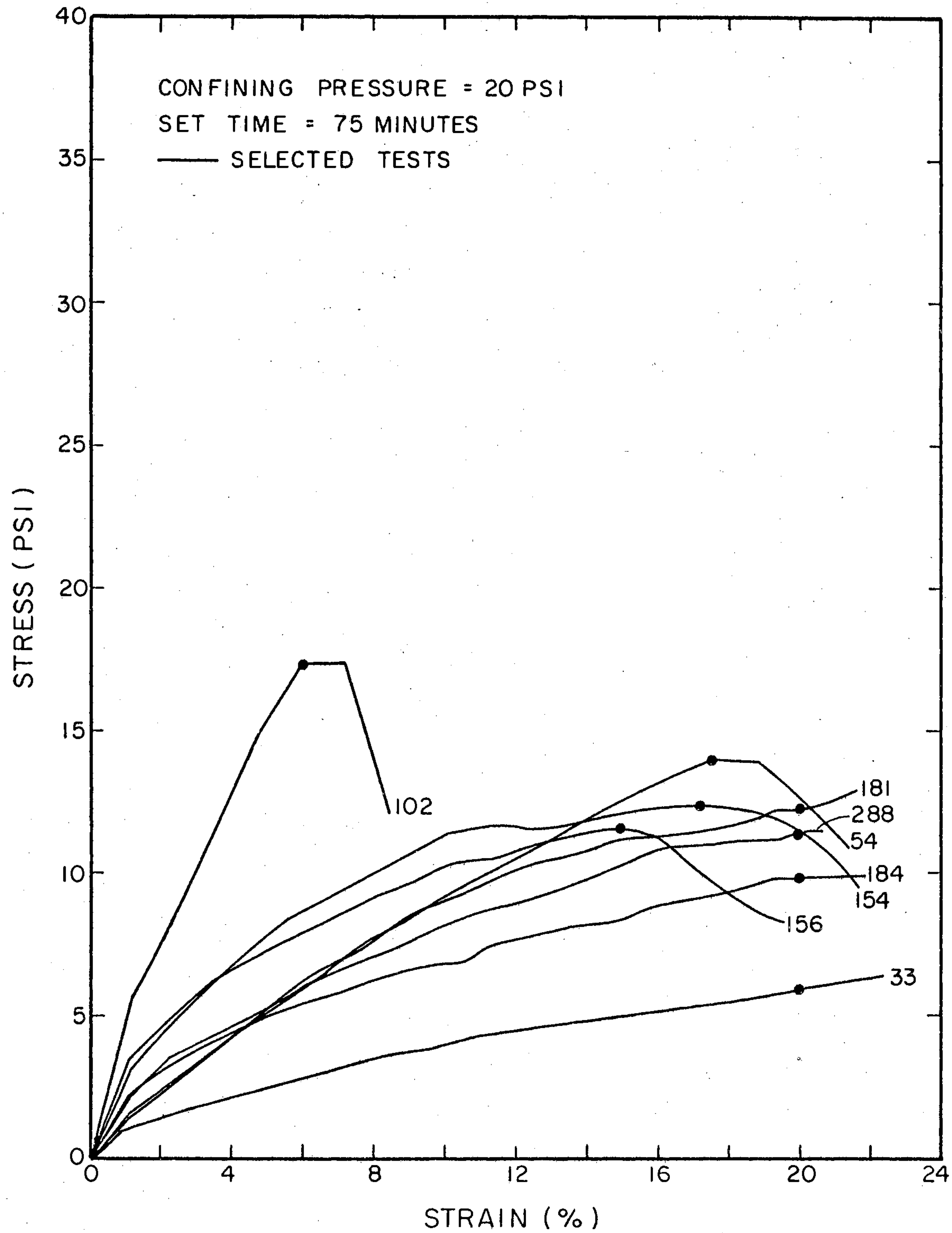

Figure B.5. Stress Versus Strain (CP = 20 PSI, ST = 75 MIN)

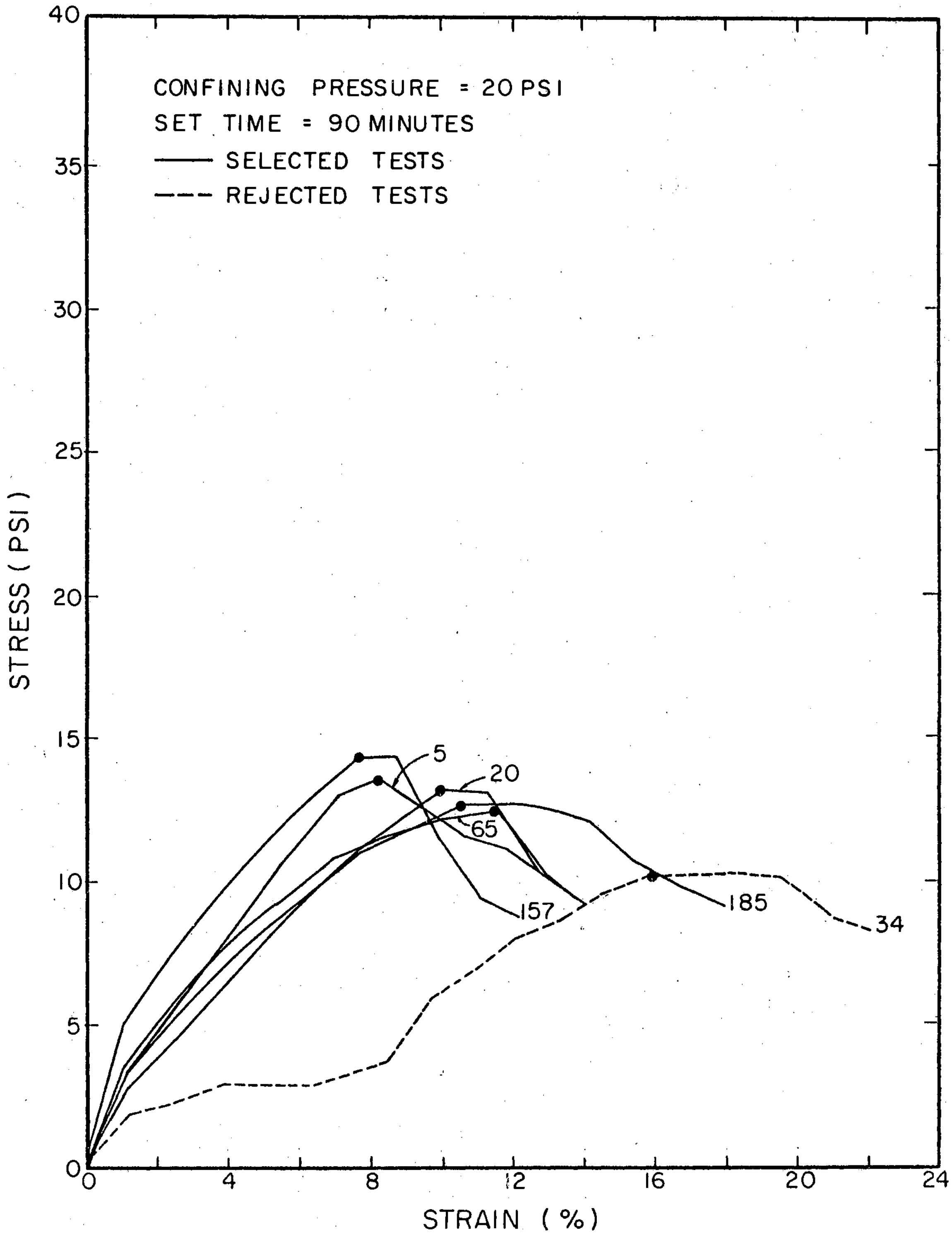

Figure B.6. Stress Versus Strain (CP = 20 PSI, ST = 90 MIN)

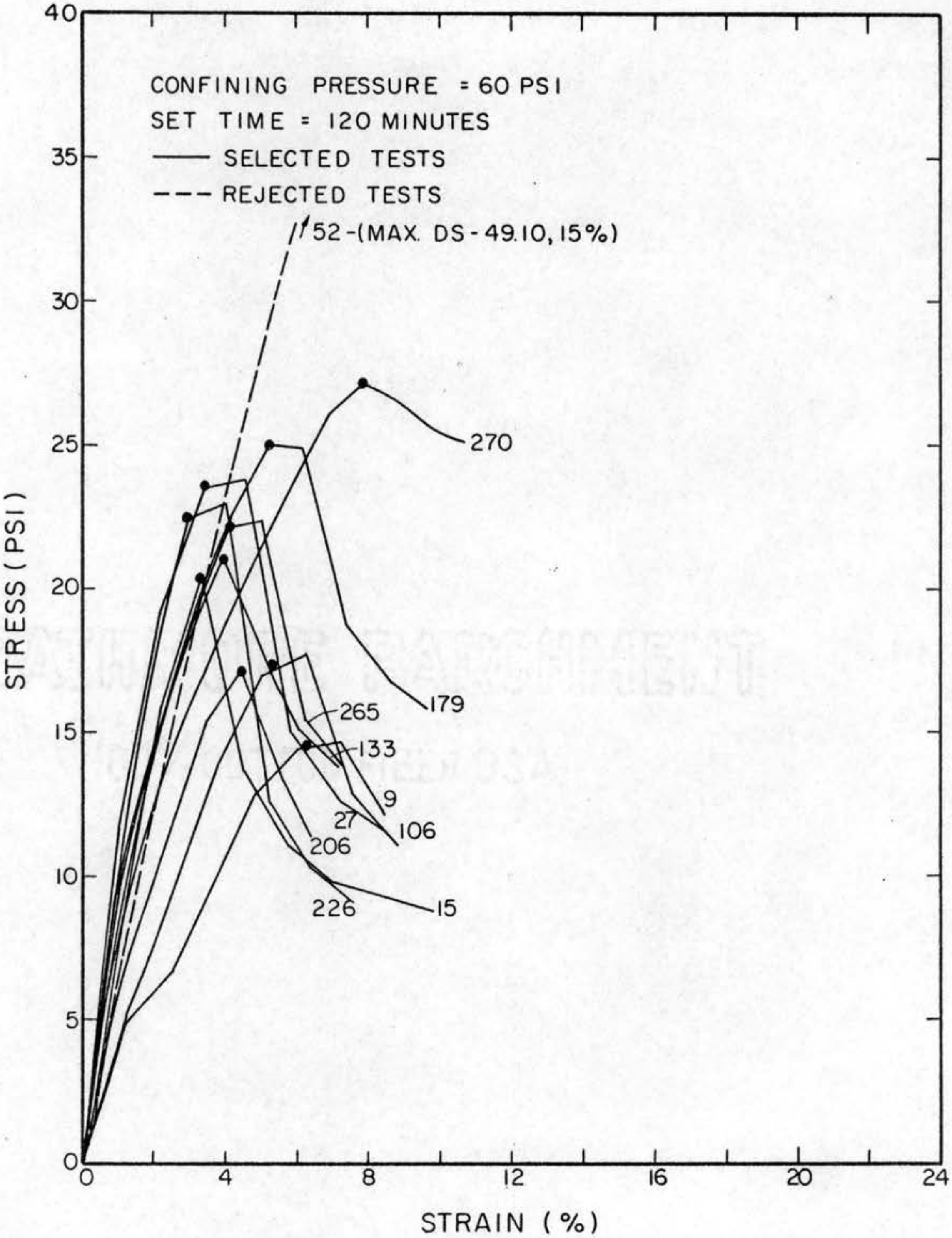

Figure B.7. Stress Versus Strain (CP = 60 PSI, ST = 120 MIN)

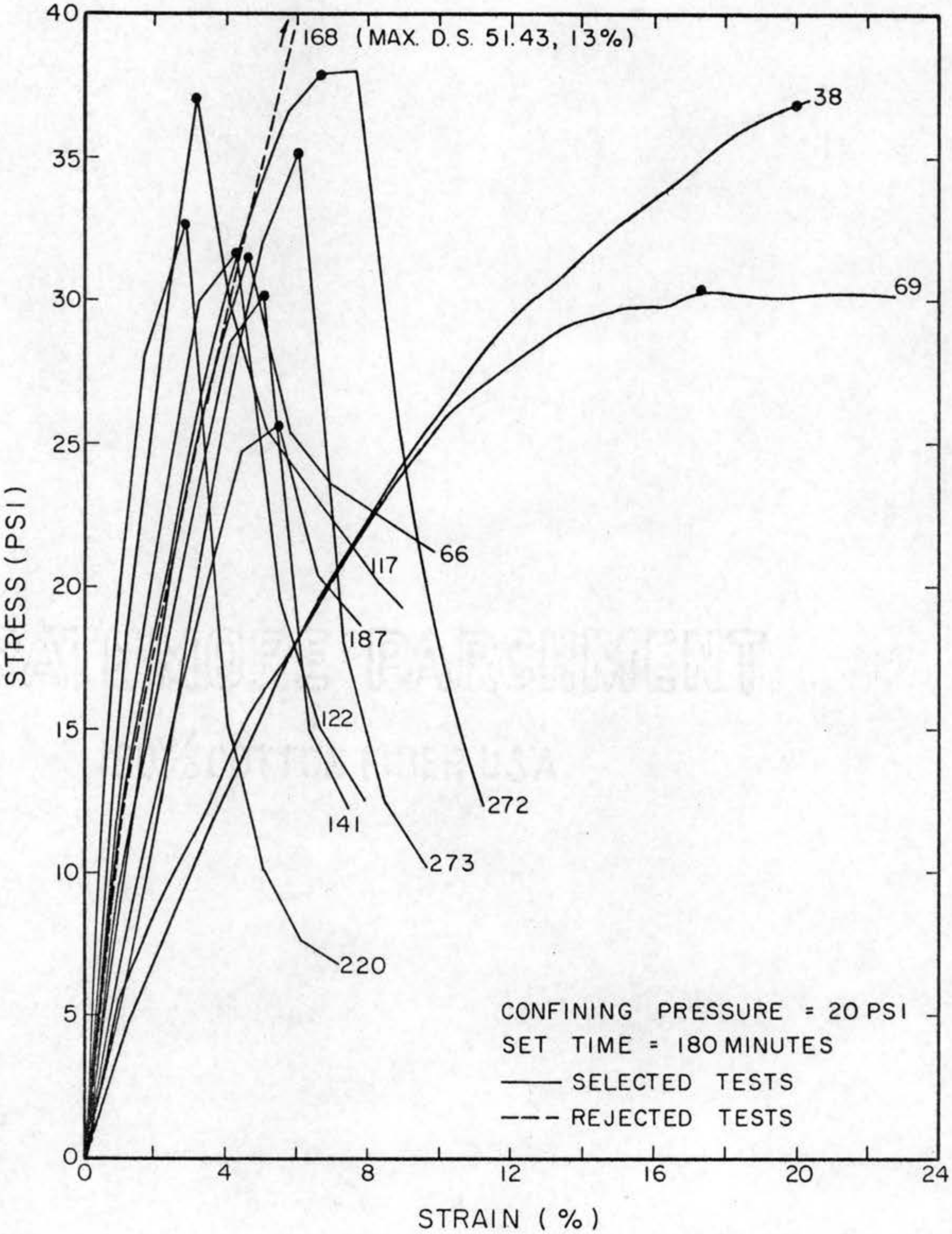

Figure B.8. Stress Versus Strain (CP = 20 PSI, ST = 180 MIN)

APPENDIX C

FLOW CHART SUMMARY OF COMPUTER PROGRAMS USED IN THE CALCULATIONS

PROGRAM I (Figure C.1) —Computation of deviator stress and associated strain for each individual test (see Appendix A).

PROGRAM II (Figure C.2) —Computer plot of the stress-strain relationship for each test with all tests of the same CP-ST combination on one plot (see Appendix B).

PROGRAM III (Figure C.3) —Computation of the average, standard deviation, and coefficient of variation of the deviator stress values in each combination. Check on the two standard deviation limit and ommission of tests. After which the average, standard deviation, and coefficient of variation are recomputed and an 80 per cent C. I. set for the combination.

PROGRAM IV (Figure C.4) —Computation of a 90 per cent confidence interval band on the linear regression line using all values at a particular confining pressure except those with a set time of 180 minutes.

PROGRAM V (Figure C.5) —Computation of lateral pressures of concrete by the new method using the results of the triaxial tests. Also the evaluation of pressures using CERA and ACI formulas.

Figure C.6 —Description of flow chart symbols.

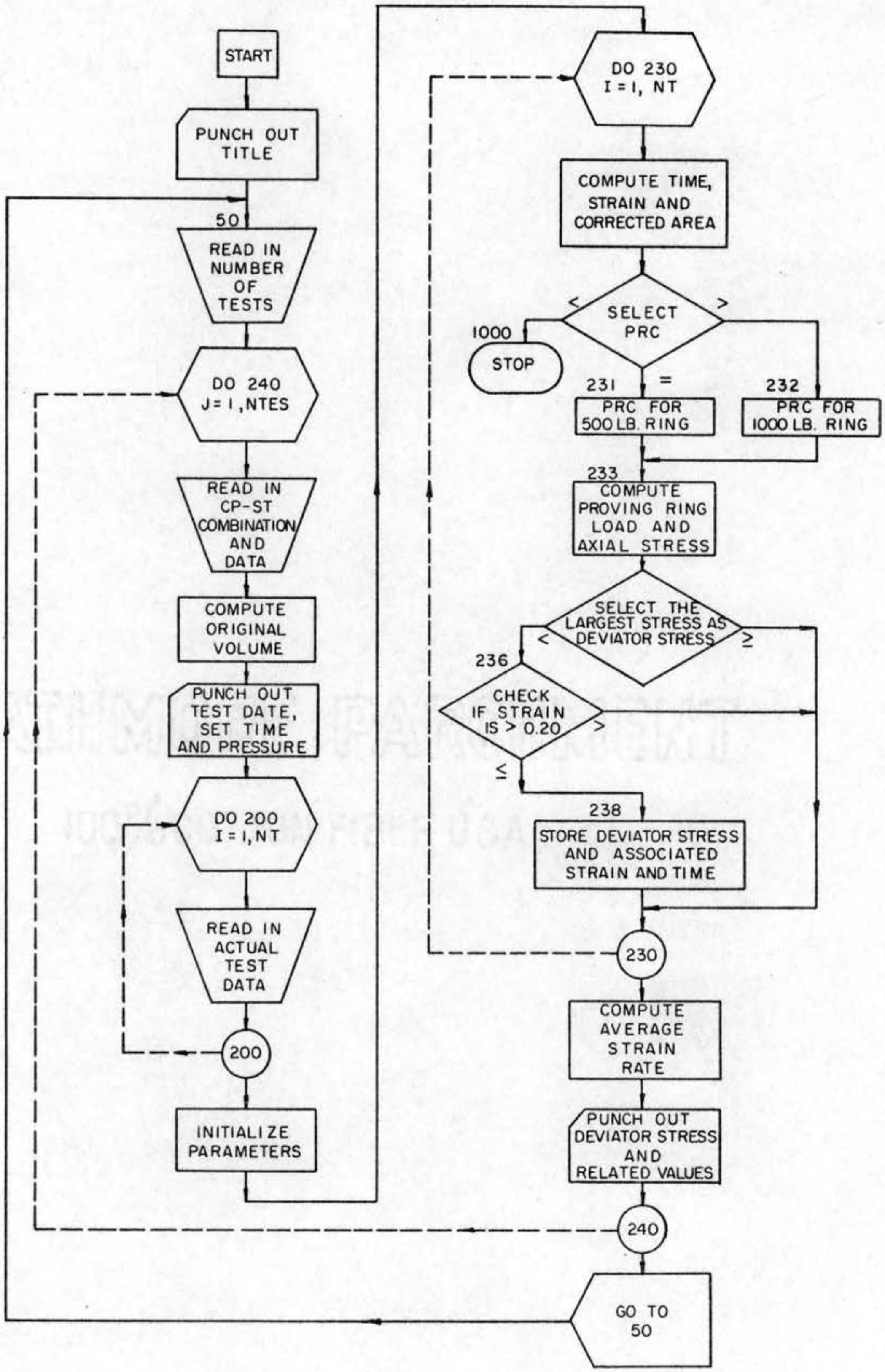

Figure C.1. Flow Chart for Program I

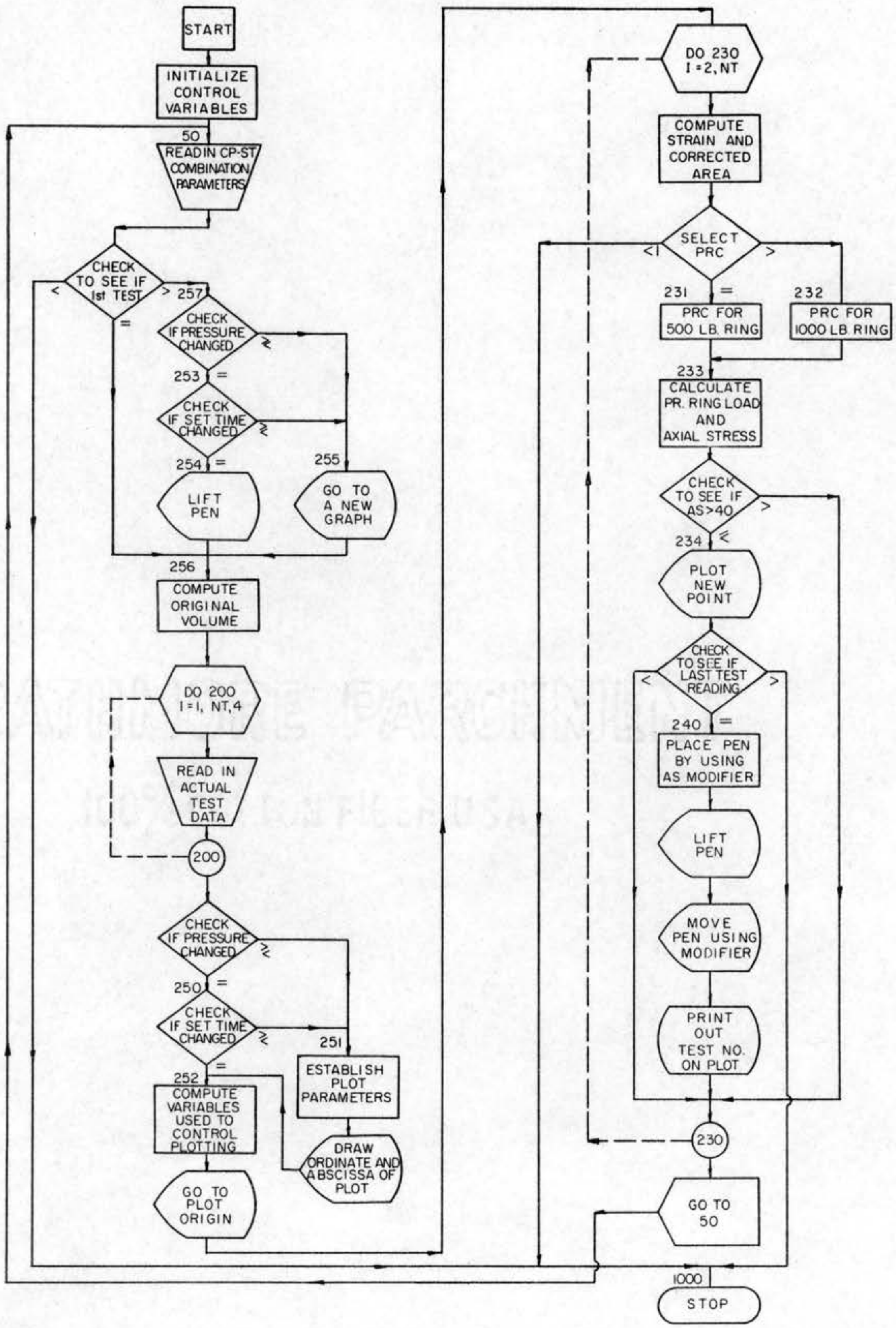

Figure C.2. Flow Chart for Program II

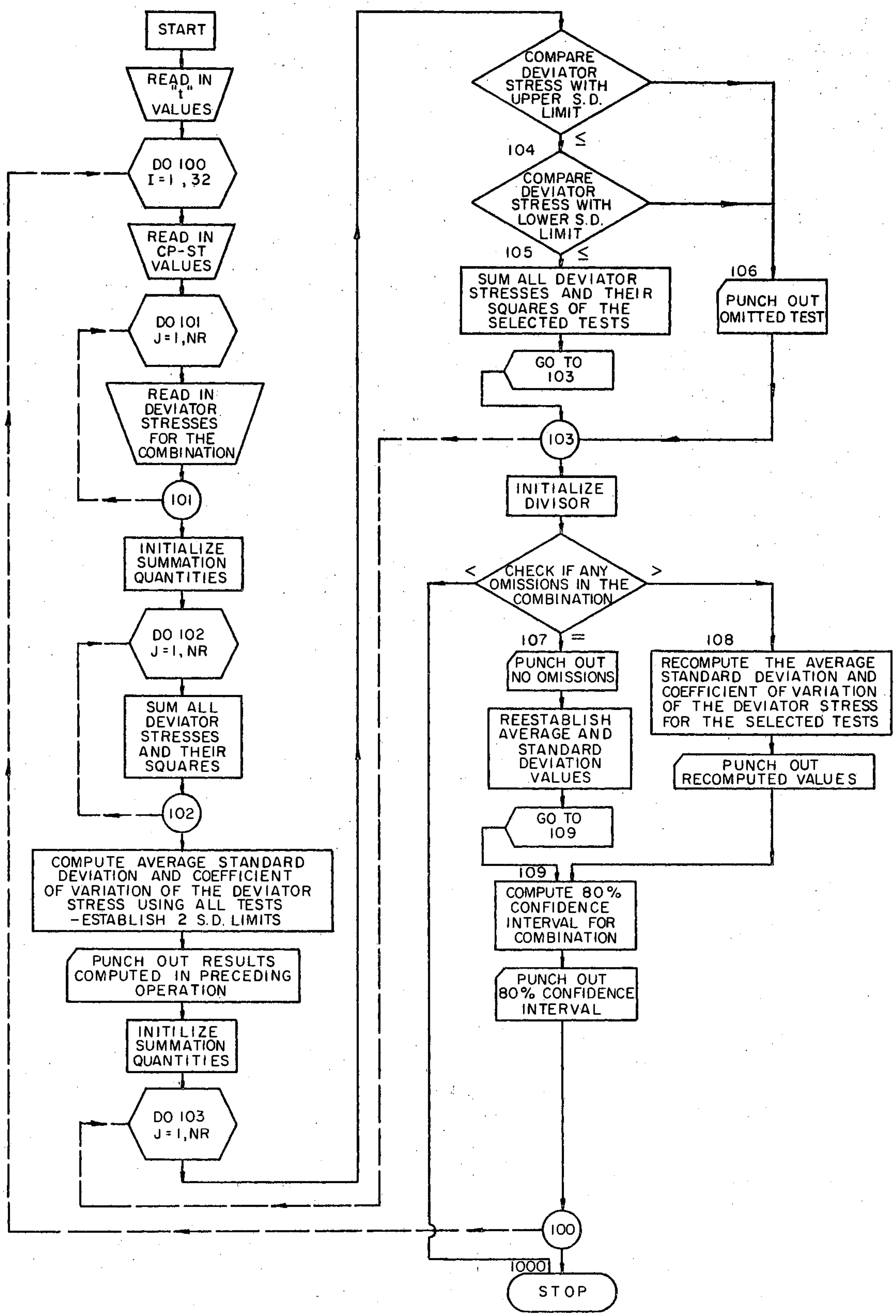

Figure C.3. Flow Chart for Program III

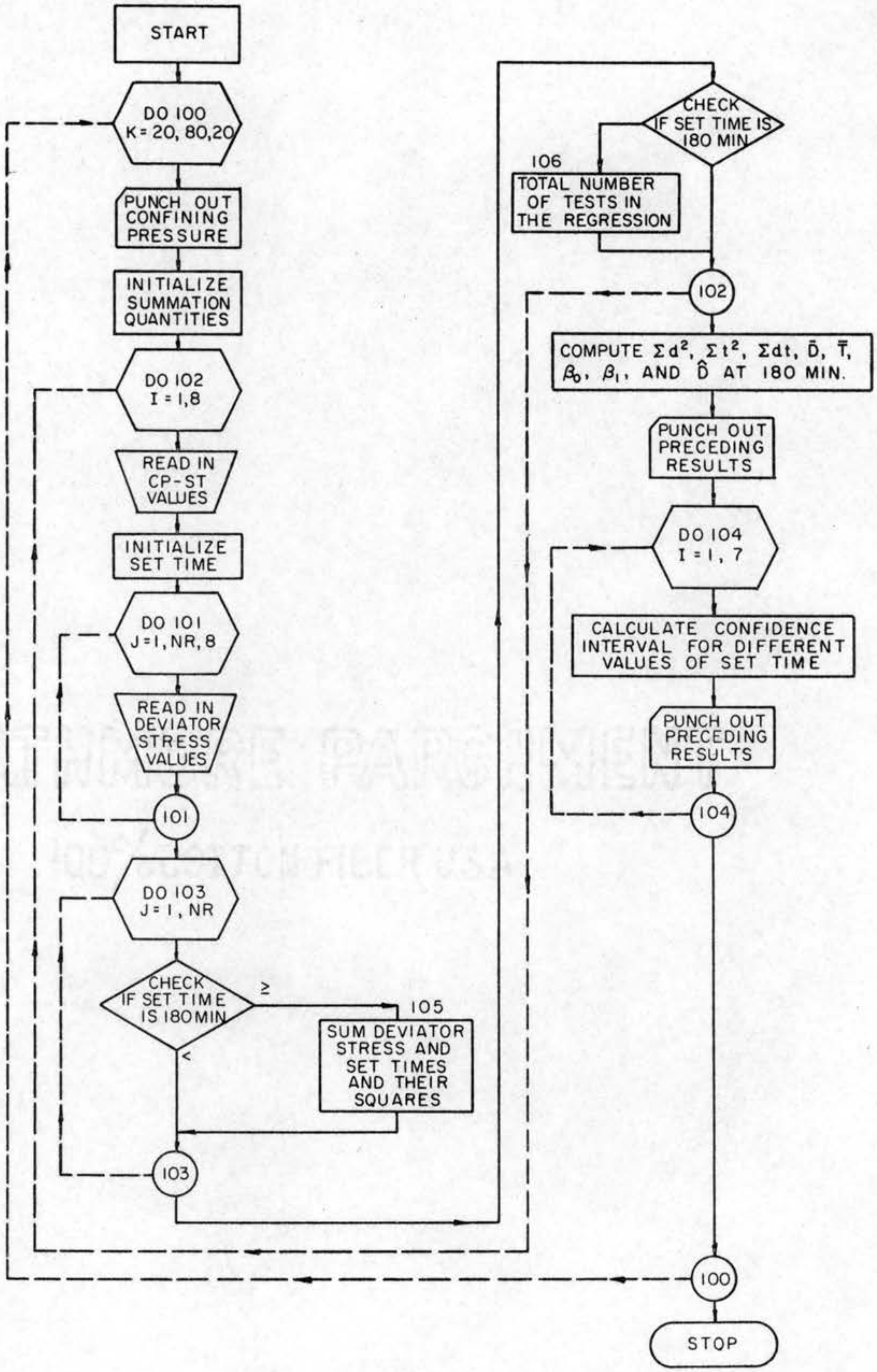

Figure C.4. Flow Chart for Program IV

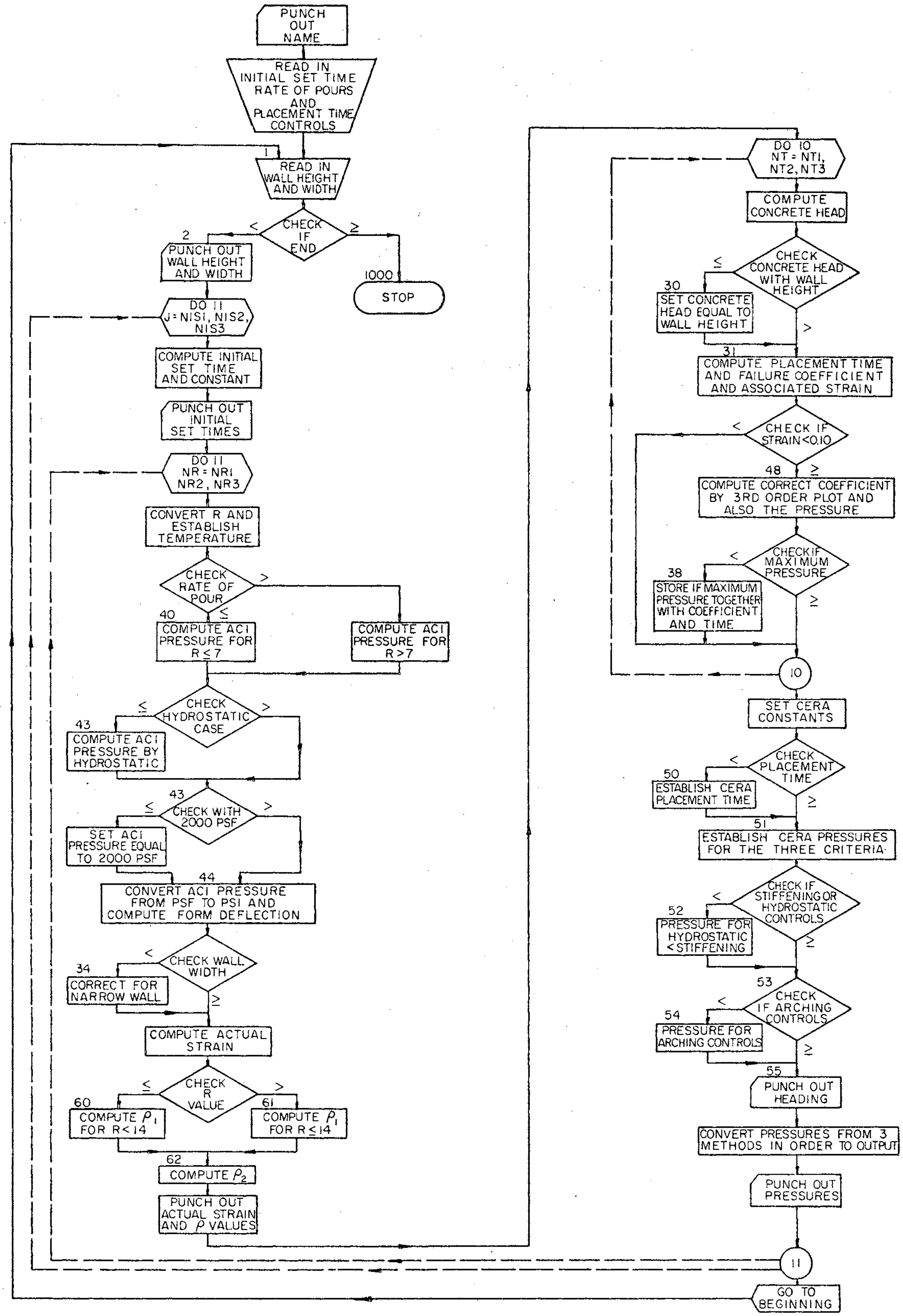

Figure C.5. Flow Chart for Program V

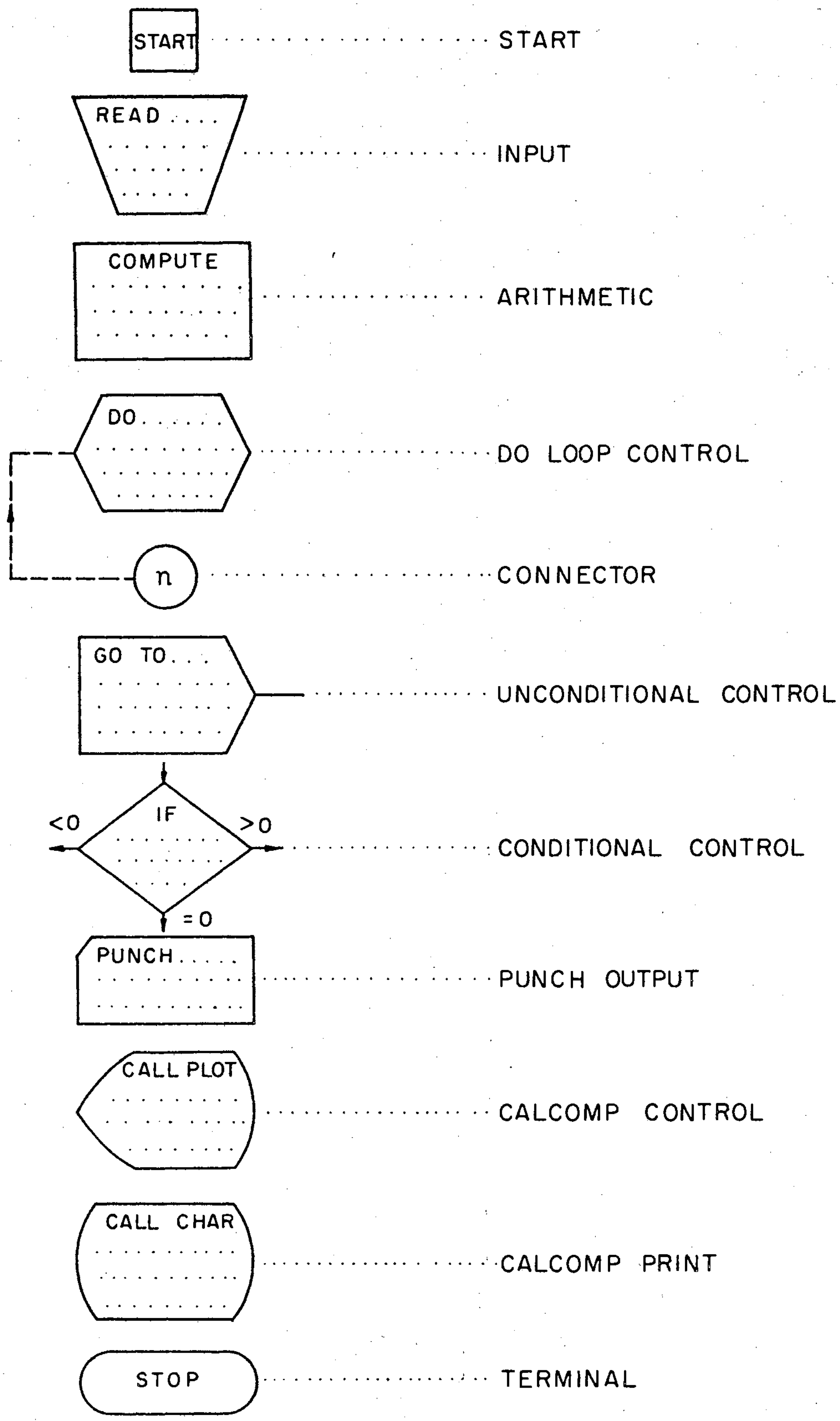

Figure C.6. Description of Flow Chart Symbols

APPENDIX D

CORRECTIONS FOR DEVIATOR STRESS RESULTS

In calculating the deviator stress values in Chapter II three assumptions were made. These were

<u>Assumption 1</u> — As the sample deformed, the cross-sectional area remained uniform throughout the height.

<u>Assumption 2</u> — The concrete sample did not slump after the compaction mold was removed until the time of testing.

<u>Assumption 3</u> — The membrane influence did not contribute to the total strength of the specimen.

If one would prefer not to make these assumptions a method to correct for each of them follows.

<u>Assumption 1</u> — Although a uniform area assumption is common in soil testing (ASTM(23)) the actual area is slightly larger in the center than at the ends. Because of this, the area which is used at the critical section through the center underestimates the actual area. This area in turn overestimates the actual deviator stress. This is particularly true if large deformations are encountered. Another possibility would be if the concrete sample took the form of two frustums of a cone adjacent to each other with a

common base (see Figure D.1).

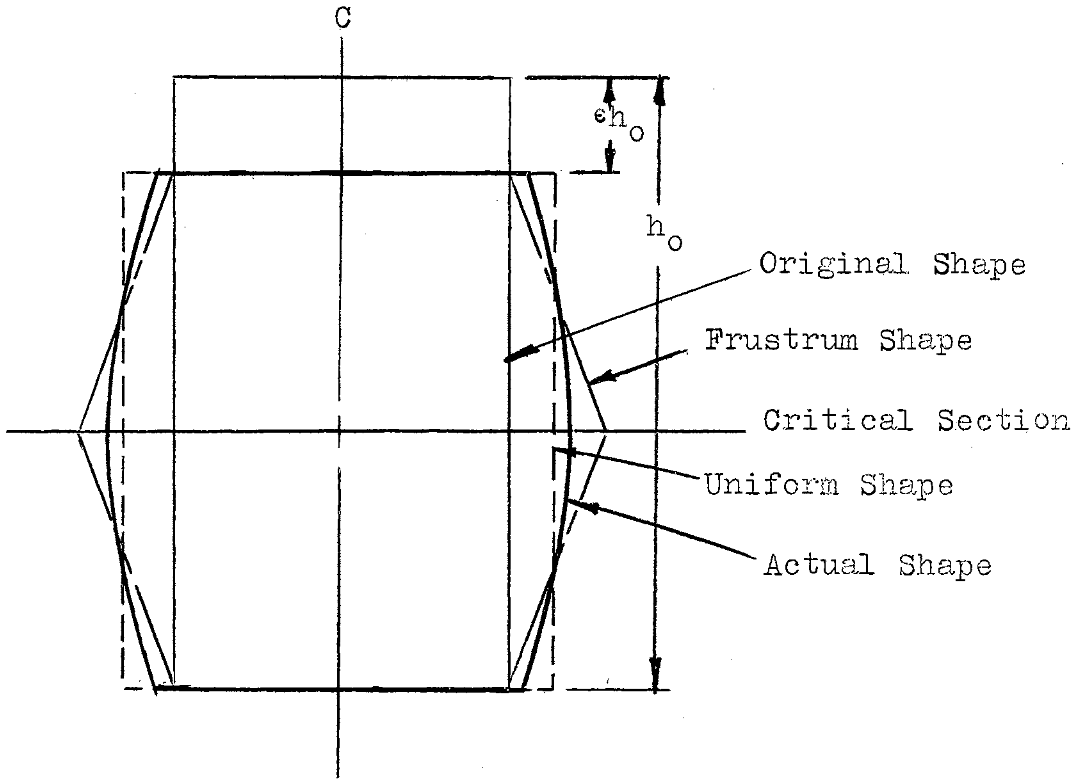

Figure D.1. Deformation Patterns in the Concrete Sample During Triaxial Testing

With a uniform area (A_u) throughout and constant volume the area for any axial strain (ϵ) in terms of the original area (A_o) is

$$A_u = \frac{A_o}{1 - \epsilon} \ . \tag{D-1}$$

With two frustums and constant volume the area at the center for any axial strain (ϵ) in terms of the original area (A_o) is

$$A_{2F} = \frac{A_o}{2(1 - \epsilon)} [5 + \epsilon - \sqrt{9 - 6\epsilon - 3\epsilon^2}]. \tag{D-2}$$

It was assumed in determining this correction that the uniform area was more representative thus the corrected area (A_c) was calculated with the following formula,

$$A_c = \frac{2A_u + A_{2F}}{3} \, . \tag{D-3}$$

Corrections for the deviator stresses (CDSA) were then calculated with

$$CDSA = \frac{A_c - A_u}{A_c} \times 100 \, . \tag{D-4}$$

Formulas D-1, D-2, D-3 and D-4 were evaluated for strains of 5, 10, 15 and 20 per cent and the results are shown in the following table;

Strain	A_u	A_{2F}	A_c	CDSA
%	x A_o	x A_o	x A_o	%
0	1.00	1.00	1.00	0
5	1.05	1.10	1.07	2
10	1.11	1.22	1.15	3.5
15	1.18	1.35	1.24	4.5
20	1.25	1.50	1.33	6.0

<u>Assumption 2</u> — The slump which occurred in the concrete sample after removing the compaction mold and prior to commencement of testing was assumed negligible. However, a value was indirectly obtained for the actual slump (S) at each set time. Assuming that this slump maintained a uniform

area, the following formula for the area resulted,

$$A_S = \frac{h_o A_o}{h_o - S} . \qquad \text{(D-5)}$$

Corrections for the deviator stresses (CDSS) were then calculated with

$$\text{CDSS} = \frac{S}{h_o} \times 100 . \qquad \text{(D-6)}$$

With these two formulas and the observed slump values A_S and CDSS were calculated for each set time. The results are summarized in the following table;

Set Time	Slump	A_S	CDSS
min.	in.	x A_o	%
20	0.46	1.08	8
30	0.43	1.07	7
45	0.38	1.06	6
60	0.32	1.05	5
75	0.30	1.05	5
90	0.22	1.04	4
120	0.18	1.03	3
180	0.06	1.01	1

<u>Assumption 3</u> — The effect the membrane has on the deviator stress is uncertain. Most investigators ignore its effect. Gilbert and Henkel (28) performed a number of tests to evaluate the strength contributed by the membrane. They reported that the strength was dependent only on the

thickness of the membrane and the strain at failure and not on the confining pressure and strength of the specimen. Using their results the corrections for each set time would be

Set Time	Average Deviator Stress	Strength Reduction	CDSM
min.	PSI	PSI	%
20	9.0	1.0	11
30	10.0	1.0	10
45	11.0	0.9	8
60	13.0	0.8	6
75	14.0	0.7	5
90	15.0	0.6	4
120	18.0	0.5	3
180	30.0	0.4	1

VITA

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