STUDY OF POST-TENSIONED STRUCTURAL SYSTEM

by

James R. Goodman and John Haygreen

Prepared for

Boise Cascade Corporation
Lumber Division

Colorado State University Research Foundation
Civil Engineering Section
and
Forestry and Range Management Section

November 1962

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CONTENTS

FIGURES

TABLES

PLATES

STUDY OF POST-TENSIONED STRUCTURAL SYSTEM

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I. INTRODUCTION

In October of 1961 a proposal (CEP61JRG25) was submitted to the Boise Cascade Corporation for a pilot study of the proposed post-tensioned structure system. This proposal was the result of several preliminary efforts to reach agreement on a method of approach to the investigation of the proposed system.

Following a discussion with Mr. Steele Barnett, Product Development Manager of the Boise Cascade Corporation, a contract was written covering the research work to be done. This contract was mutually agreed on in April 1962. The contract amount was $6,445.

During May 1962, preliminary work began on the project. Late in June the wire and other materials were obtained from the sponsor. Equipment and instrumentation construction was begun in June. Testing work began in July and continued into September. Testing was completed during September. Data analysis and the completion of the report continued into October.

II. OBJECTIVES OF RESEARCH

The objectives of the research conducted were outlined in the proposal submitted in October 1961. The basic objective of this research effort was to test the over-all structural soundness of the proposed system. The investigation was directed to determine the strength properties of a single joint; i.e. its ability to carry load imposed in several manners. In addition, the effect of moisture and wood creep on the system were felt to be basic to the structural soundness. A basic study of the properties of the system under the action of moisture change and creep was included.

Additional items were included in the test objectives such as an investigation of the proposed bearing block system, wire size and wood properties perpendicular to the grain.

All of tests were based on the idea of obtaining the properties of the system which would enable a design to be made using the results of the research effort as basic criteria.

III. EQUIPMENT AND INSTRUMENTATION

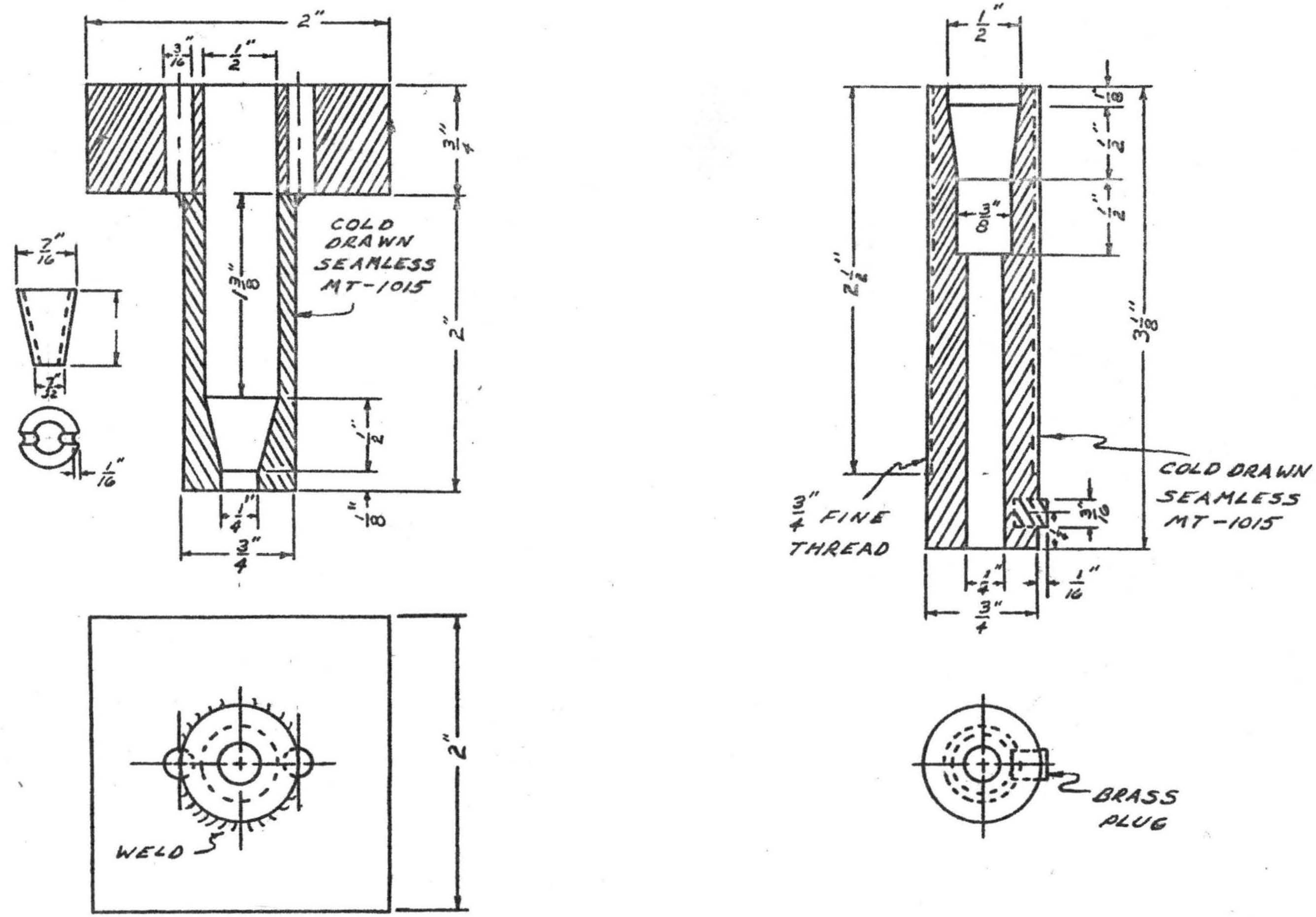
To accomplish the objectives of the testing program it was felt that it was basic to the study to have the ability to measure the force in the tendons at all times. A system was needed which would allow measuring the tensioning force to determine the original joint compression. The transducer developed must also be capable of measuring changes in the tendon force during the various tests. After several preliminary efforts a system was devised which met the requirements imposed.

The tendon force transducer consisted of a strain gauge device which also served as a bearing system for the wire. Basically this system consisted of a hollow steel tube on which the strain gauges were mounted. A tapered hole in the tube served to grip the wedge system which held the wire. A block was welded to the end of the transducer system which in turn was used as a bearing. A drawing of this system is shown in Fig. 1. Two strain gauges were used on each transducer to eliminate bending effects.

This system worked very well both in measuring the original force applied and also to determine the tendon force during testing.

At the other end of the wire a tensioning system was combined with the wire wedge system. A threaded tube was used in order to pull the wires to the proper tension. The wedge recess was placed at the end of the tube. A stop to prevent turning of the system during tensioning was provided. This stop fits into a recess in the bearing plate. A drawing of this system is also shown in Fig. 1.

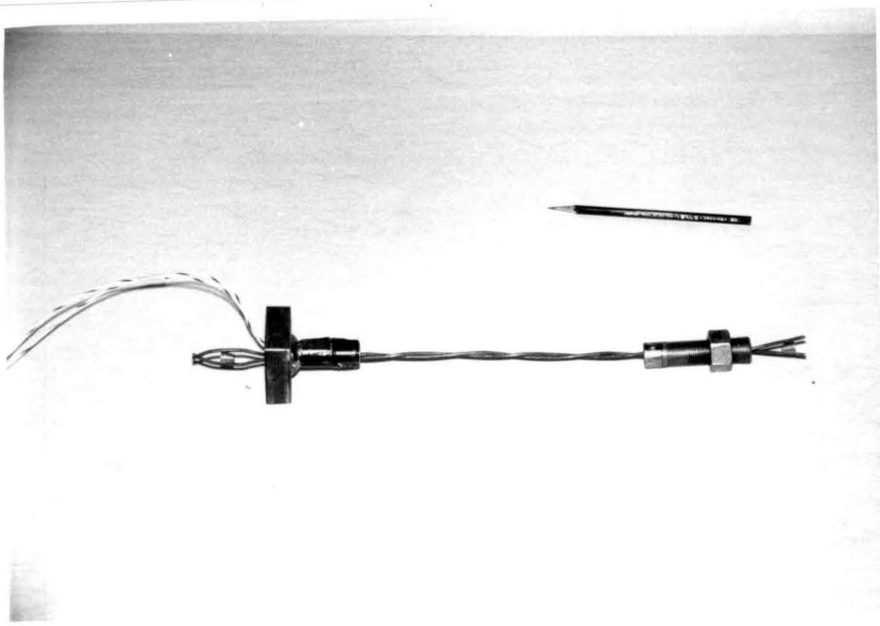
A photograph of the combined system is shown in plate 1. The hardened steel wedge with grooves on the side was used to grip the wire. This system was capable of developing stress in the wire to nearly its yield point before failure occurred. The wedges are shown on the wire protruding from the ends of tensioning and transducer devices.



TRANSDUCER ASSEMBLY

FIG. 1

TENSIONING DEVICE



TENSIONING AND TRANSDUCER DEVICE

PLATE 1

The entire bearing tensioning and transducer system is shown in the next photograph, plate 2. Bearing plates with built-up blocks at each wire were used. The blocks were recessed to hold the tensioning and transducer devices. The use of the bearing plates gave a uniform pressure on the wood throughout the tests.

The entire assembly worked very well and insured that the stress conditions during testing were known. This provided a means of reliable analysis.

COMPLETE TENSIONING SYSTEM

PLATE 2

IV. TYPICAL PANELS

A typical panel used in structural testing consisted of 7 individual boards. The over-all panel length was 48 inches. This allowed the use of 4 wires spaced at 12 inches center to center. Plates were bolted to the panels in such a manner as to expose the center board. The test joints were the two joints on each side of the center board.

The panels were stressed prior to tightening the bolts in the side plates. The side plates were 4-1/2 x 1/4 x 4' - 0" plates and insured that the test joints only were exposed to the test load. A sketch of a typical panel is shown in Fig. 2.

V. STRUCTURAL TESTS

Structural tests were run under several loading conditions so that the joint strength could be obtained. Load was applied in all cases using the 60,000 lb. capacity, Baldwin Hydraulic Testing Machine. During all tests a record of tendon pressure was made during testing so that changes in wire force could be determined. A multiple channel switching and balancing unit and a Baldwin strain indicator were used to obtain the strain readings.

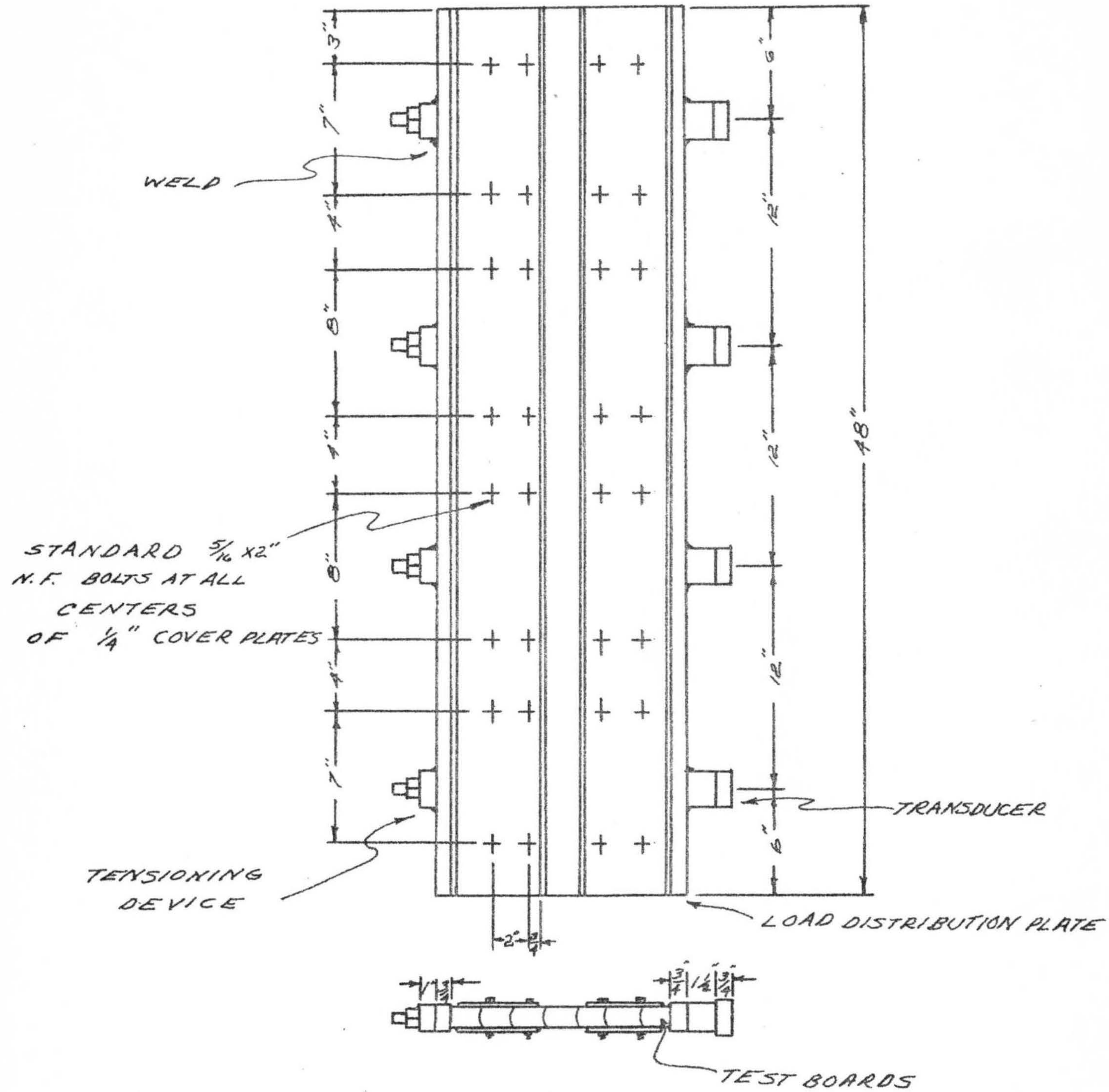
The wood used for the structural tests contained about 8 percent moisture at time of testing. No attempt was made to select special wood pieces for testings except that all pieces used were free of any damage of any sort. All pieces used were flat-sawn sapwood of ponderosa pine.

A. Direct Compression Perpendicular to the Grain

The amount of direct compression which the system will withstand in the perpendicular to the grain direction is a function of the wood strength only. Several compression tests were run on the wood perpendicular to the grain. The results of a typical test are given by a graph (Fig. 3). The average proportional limit of the wood in compression perpendicular to the grain is about 840 psi. The modulus of elasticity is about 45,000-50,000 psi.

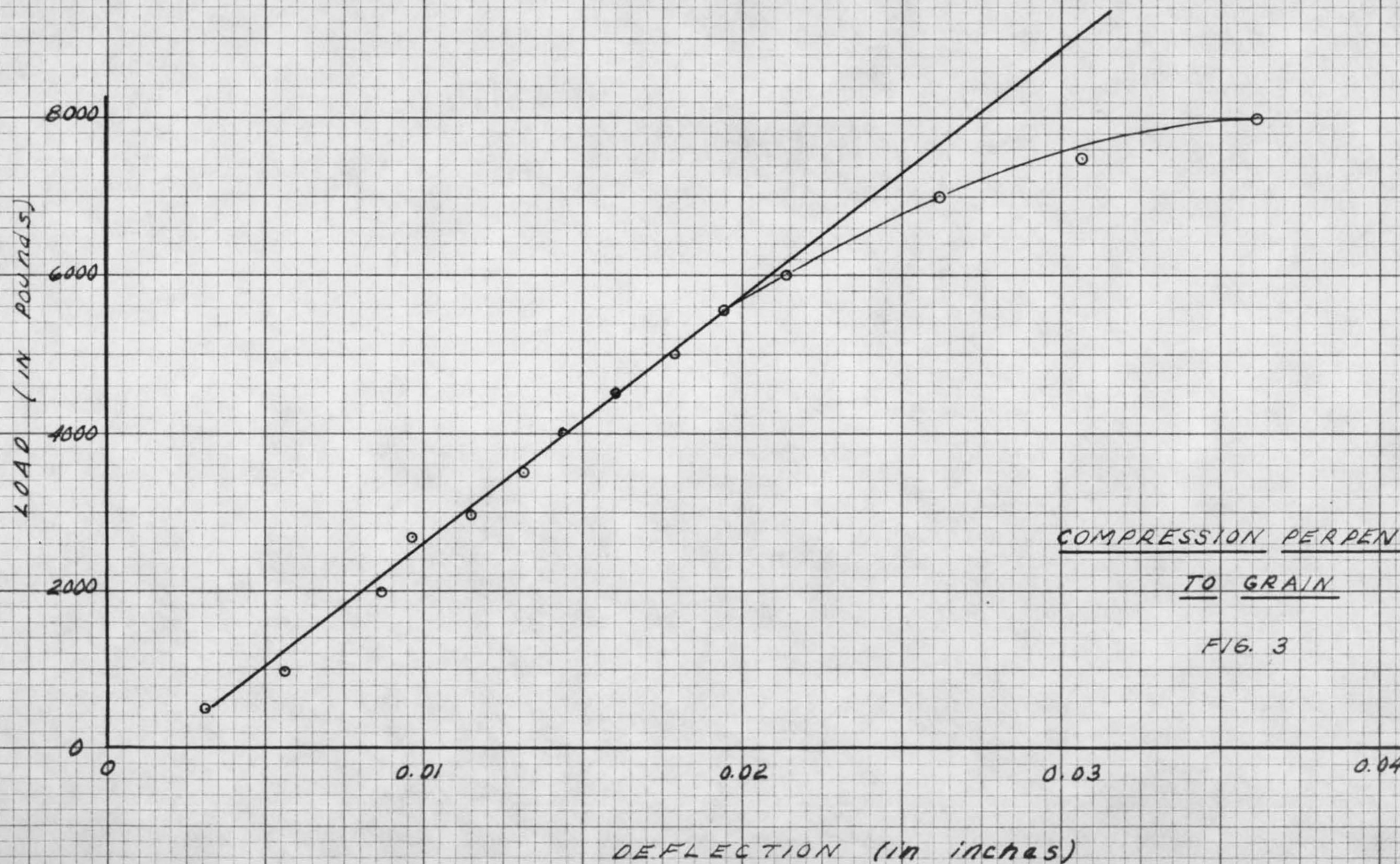
B. Maximum Prestress Load

The maximum amount of prestress force available is limited by three criterion; 1) Crushing of the wood (Direct Compression), 2) Failure of the wire in tension, 3) Failure of wedge system to hold the wire properly.



TYPICAL TEST PANEL

FIG. 2



COMPRESSION PERPENDICULAR TO GRAIN

FIG. 3

The first item is limited to about 840 psi as seen in the description of tests of direct compression. A reasonable allowable stress would be about 40 percent of 840 psi or 340 psi.

The second item is limited for all practical purposes to the yield strength of the wire. The yield strength of the wire used, ASTM A-421, is about 200,000 psi. Using a cross-section area of 0.0177 square inches the force available is about 3540 pounds. The maximum prestress force on the wood is then about 250 psi. Note that this is considerably less than the allowable value of the wood obtained which as given above could be about 340 psi.

The third item is very critical if one is to obtain the full benefit of the wire used. It is possible to make a wedge system such as the one developed for these tests which will grip the wire well enough to allow it to reach its yield strength. This appears to be the useful limit however. Tension tests of the wedge system developed for test use showed that the wedges would hold the wire to nearly its yield point consistently.

Several tension tests were run on the bearing and wedge system supplied by the sponsor. In no case was the maximum load near the yield point of the wire. The wire pulled through the hardened brass wedges. The maximum load reached was about 2500 pounds and most tests failed at a lower load.

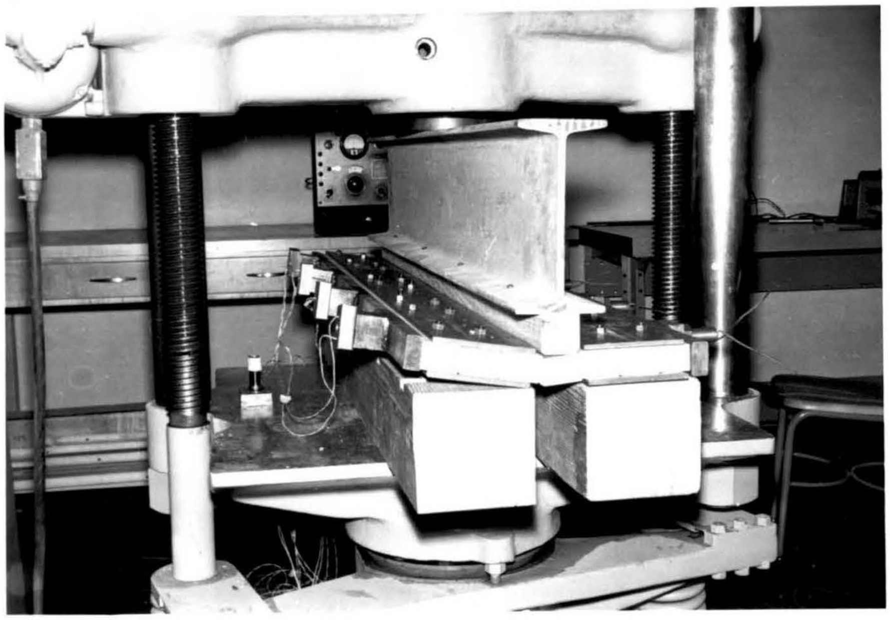
During preliminary work to develop a adequate wedge several brass and steel wedges were tried. It was found that the wedges must be such that the wire cannot bite into the wedge. Case hardened steel wedges were finally used. The grooves were kept to slightly less than the wire diameter to help prevent slippage.

C. Shear Tests

Shear tests of two types were run. In one case the load was applied as a line load along the boards. In the other case the load was applied to the end of the board in order to test the amount of friction developed by the joint.

1. Line Load Shear Test

A sketch of the setup for this test is shown in Fig. 4. A photograph of the test setup during a test is also shown in plate 3.



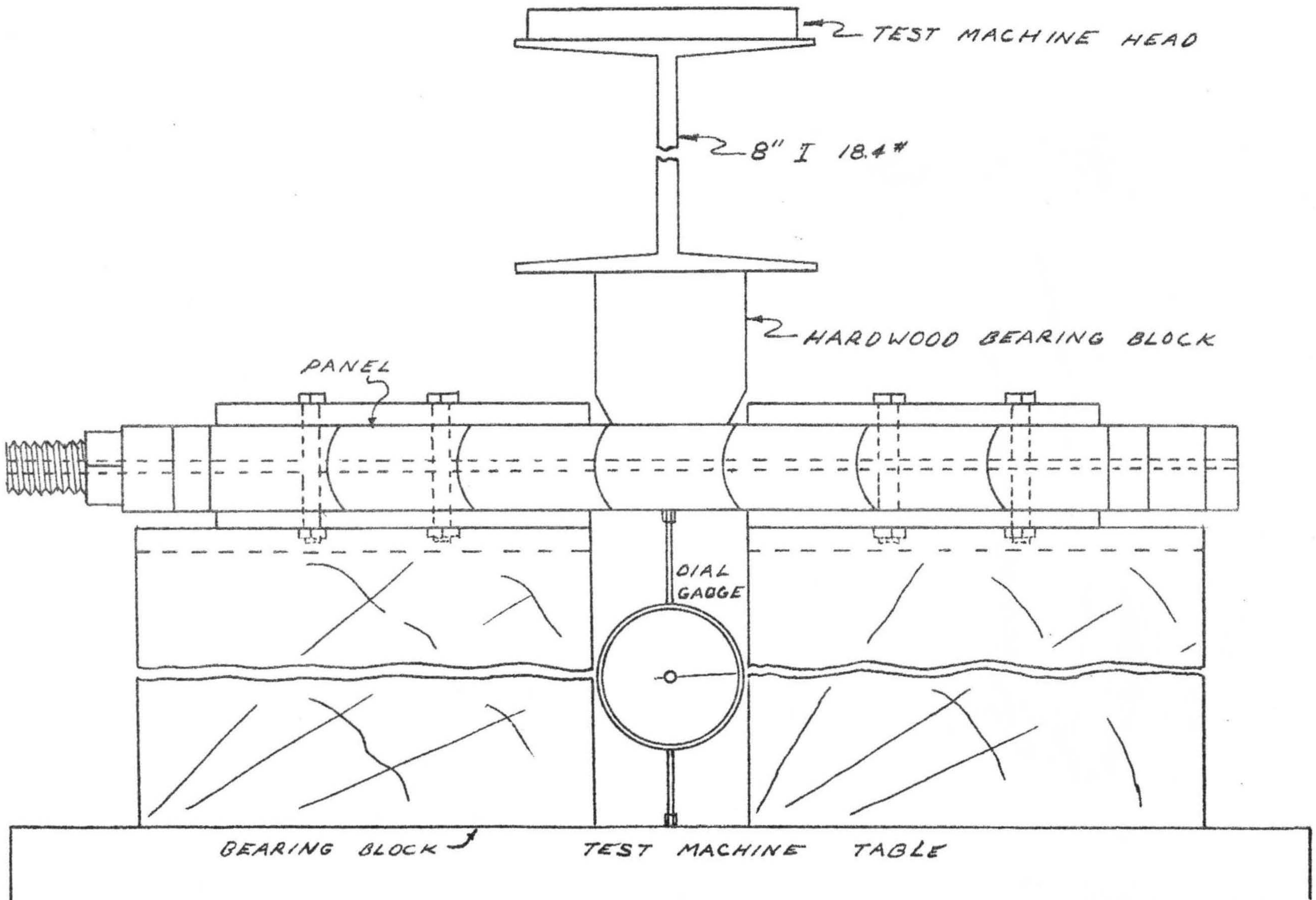
LINE LOAD SHEAR TEST
PLATE 3

The purpose of this test was to determine the amount of force required to cause yield (excessive movement) of a single board under direct force. This value would give an upper limit on the force which could be applied to the system under any circumstances since bending will be considerably more severe in its action on the panel.

The procedure used was to thread the tendons through the wood members and the bearing system after the wire had been wedged into the transducer device. The tensioning device was then wedged to the wire. The nut was then tightened to apply the prestress force. The prestress force was applied in increments in a uniform manner to insure an even pressure on the wood.

After the prestress was applied the test load was placed on the panel in increments. Loading was continued until excessive movement or yielding occurred.

This entire procedure was repeated for each of the several types of tests run and for each of the panels.



LINE LOAD SHEAR TEST

FIG. 4

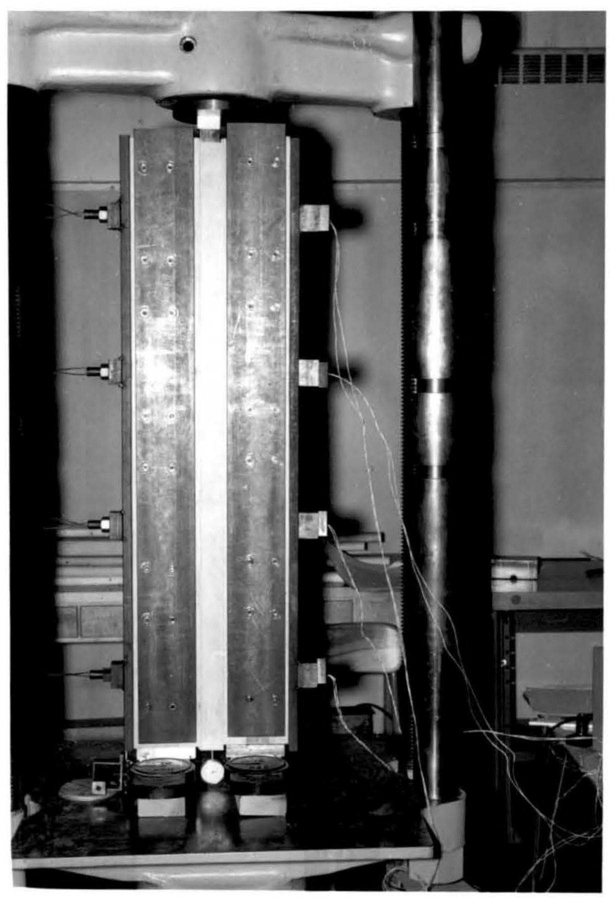
A typical set of data is shown in Table 1.

A graph of the data given is shown in Fig. 5. The useful load capacity was taken where the curve became essentially horizontal. Some rotation occurred in these tests but shear was the predominate effect. The wood was split and failure was induced in each case. Tendon pressures did not increase materially during the test.

Three tests of this type were run at different tendon pressures. A plot of the yield value; i.e. the highest useful shear stress reached versus the tendon pressure applied originally is shown in Fig. 6. The values have been converted to psi of force on the wood for easy comparison with the other tests run.

2. Basic Shear Test (Friction)

A sketch of the setup for this test is shown in Fig. 7. A photograph showing the actual test in progress is shown in plate 4.



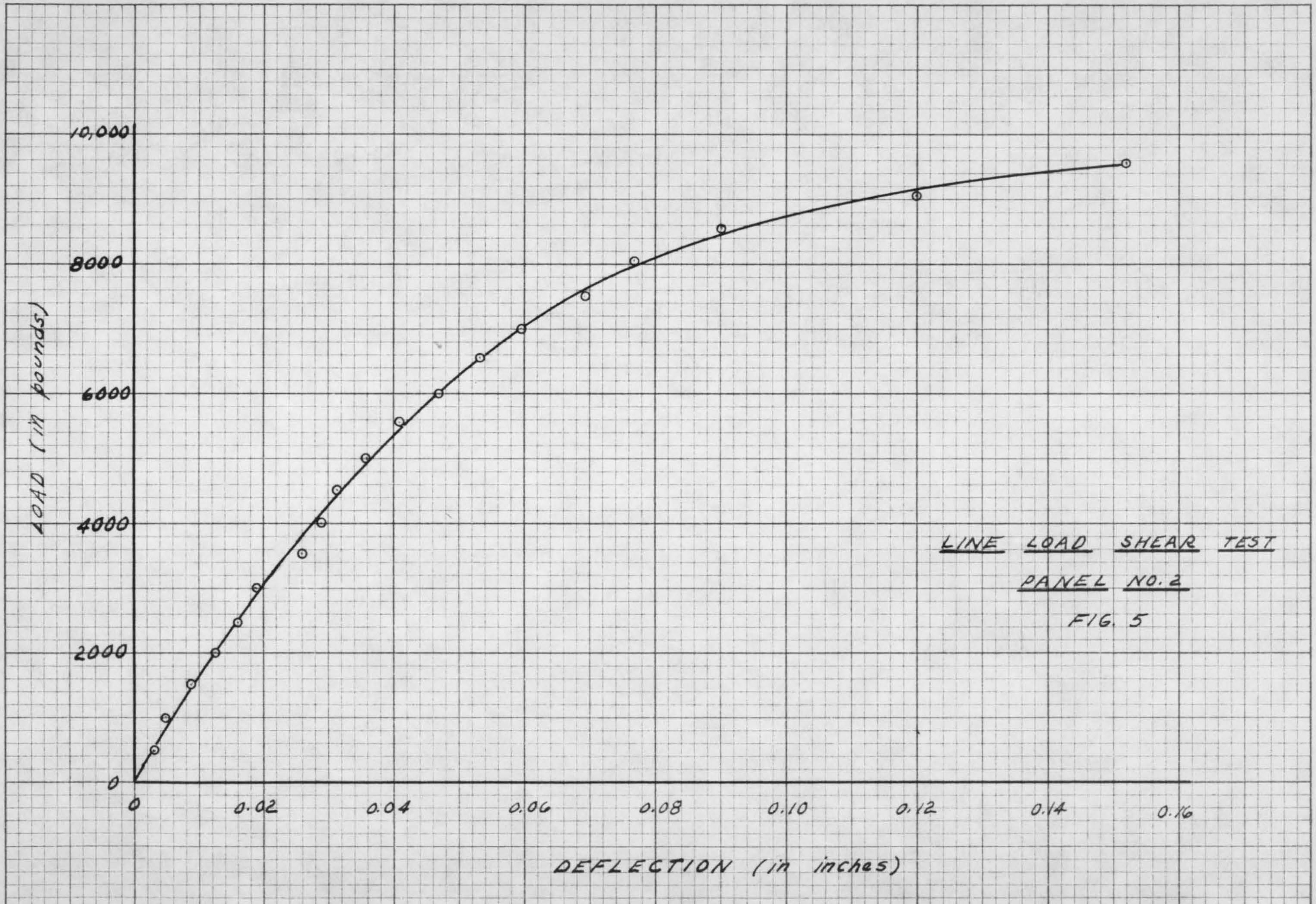
BASIC SHEAR TEST

PLATE 4

TABLE 1

SHEAR TEST NO. 2

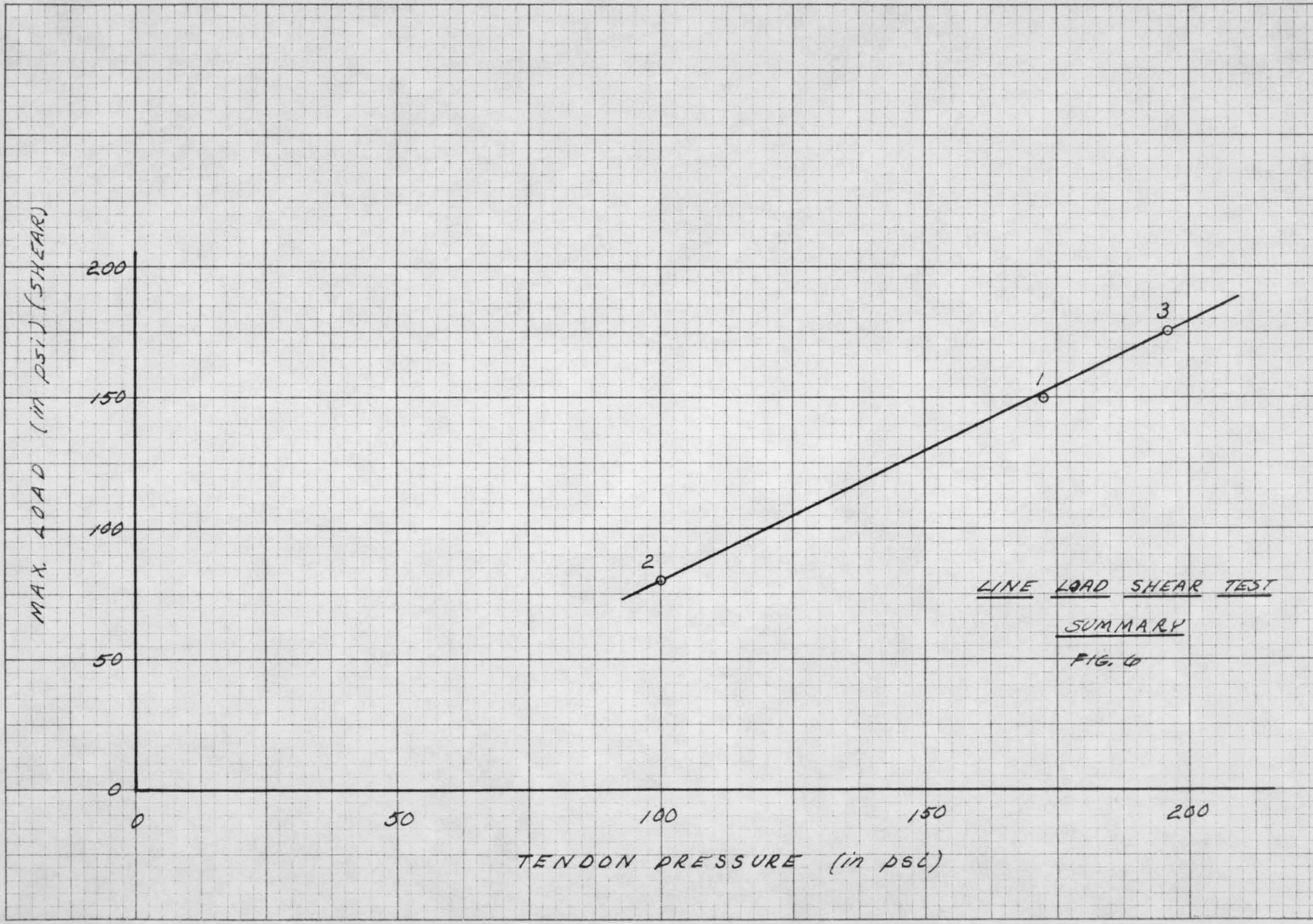
LOAD	STRAIN (in. microinches)				Δ DIAL
(in lbs)	GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4	(in inches)
Zero read	170	855	535	860	
Stressed	350	1035	717	1038	
0	358	1038	720	1050	0
500					0.003
1000	370	1049	738	1056	0.0055
1500					0.009
2000					0.0125
2500					0.016
3000					0.019
3500					0.026
4000	358	1038	728	1050	0.029
4500					0.0315
5000					0.036
5500					0.041
6000	360	1038	730	1050	0.047
6500					0.0535
7000					0.0595
7500					0.0695
8000					0.0765
8500					0.0900
9000	368	1038	750	1050	0.120
9500					0.152
10,000					
10,500	390	1060	773	1070	
11,000					
11,500					



LINE LOAD SHEAR TEST

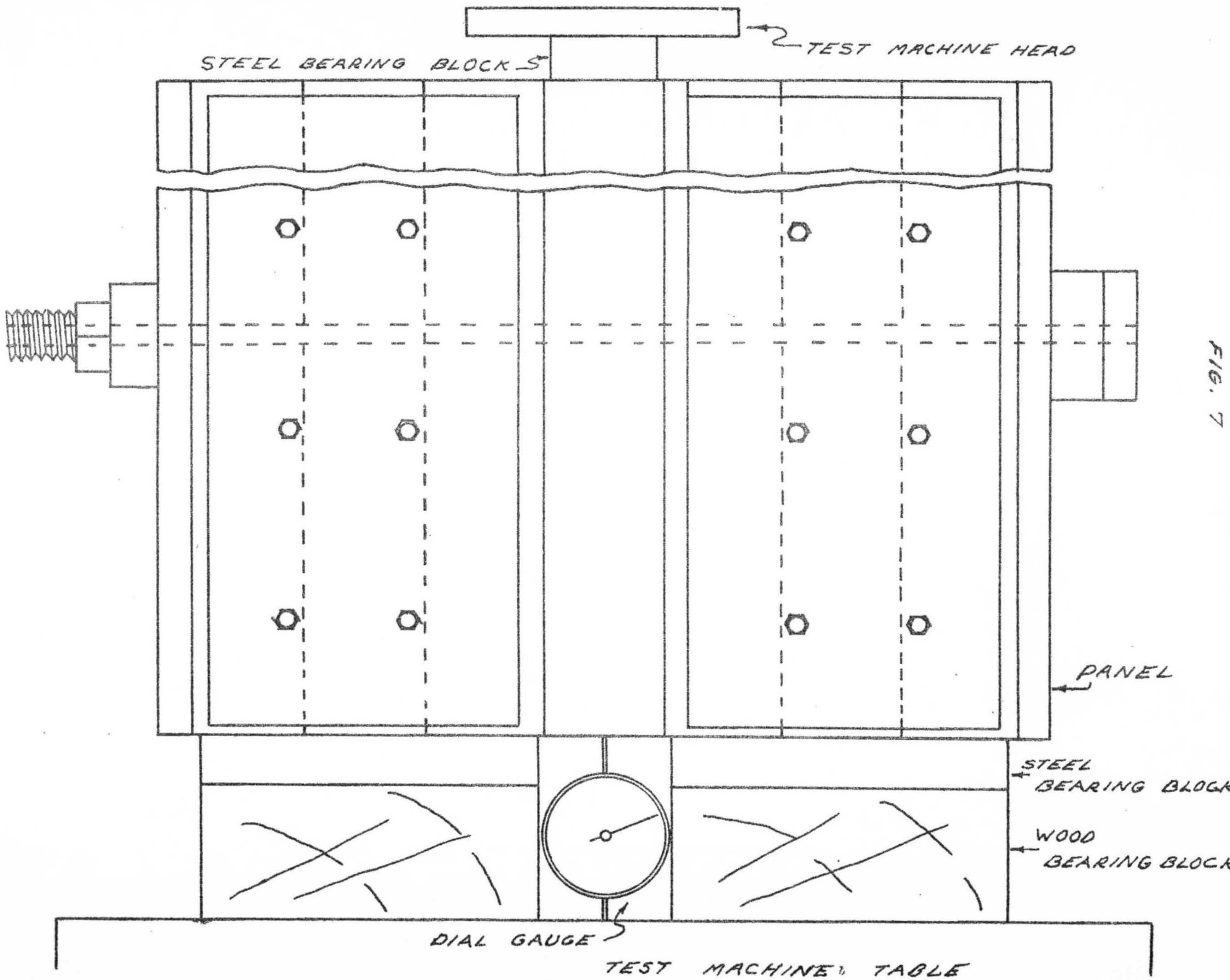
PANEL NO. 2

FIG. 5



LINE LOAD SHEAR TEST SUMMARY

FIG. 6



BASIC SHEAR TEST (FRICTION)

FIG. 7

The purpose of this test was to determine the amount of friction force developed by the system under the application of various tendon pressures.

This data will give the amount of force which can be applied along the longitudinal axis of the boards before a slip occurs.

A table of data from a typical test is shown in Table 2.

A graph of the data is shown in Fig. 8. There is quite a definite point at which slip occurs. The deflection recorded prior to this point is elastic deformation of the bearing system. This was checked by installing a second dial to record movement of the entire panel. The point of slip is taken as the useful limit of the system. It should be noted that a greater ultimate load can be carried when the wires begin to bear on the wood (after slip has occurred). This occurs after considerable movement however and the design limit should be taken at the point of slip. Again in this test tendon pressured did not increase until the wood began to bear on the tendons after slip occurred.

Five tests of this type were run in order to correlate tendon pressure and the amount of friction developed. A graph of these variables is given in Fig. 9.

As shown in the graph there appears to be a level of stress at which the friction developed does not increase substantially. The wood properties are such that the coefficient of friction changes after a certain level of prestress. The coefficient of friction at a stress level of 175 psi is about 0.36. At 110 psi the coefficient of friction is 0.38. These values are obtained by dividing the vertical shearing stress at slip by the prestress load.

3. Bending Tests

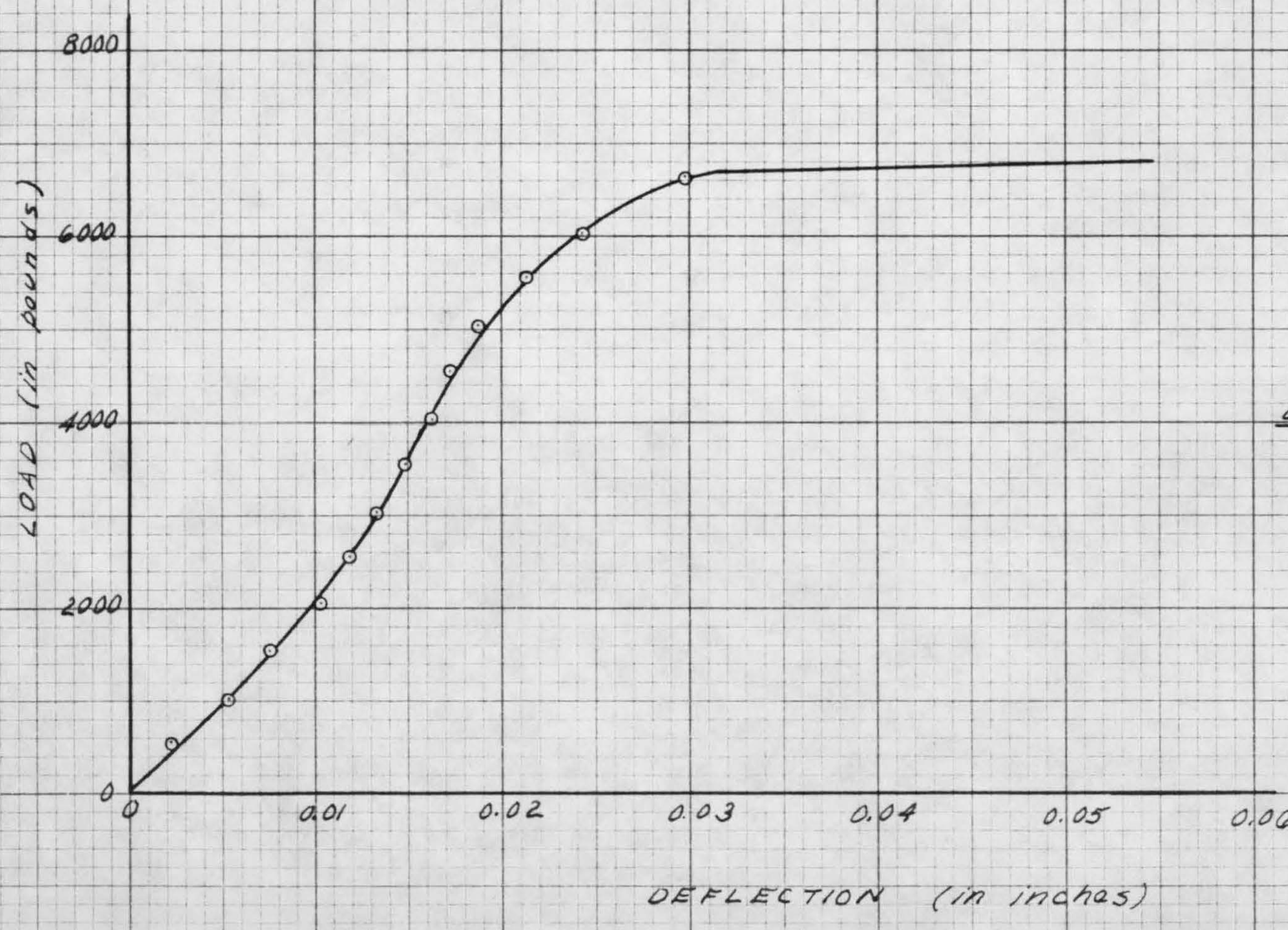
Bending tests were run to determine the resistance of a joint to the action of moment applied about the longitudinal axis of the wood. Bending strength about the transverse axis is determined almost entirely by the wood properties. Standard strength values obtained from published data may be used to determine the bending strength in this direction.

TABLE 2

BASIC SHEAR TEST NO. 5

LOAD (in lbs.)	STRAIN (in microinches) GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4	△ DIAL (in inches)
Zero read	448	1041	709	1162	
Stressed	750	1272	912	1420	
0	750	1272	912	1420	0
500					0.002
1000					0.005
1500					0.0075
2000					0.010
2500					0.0115
3000					0.013
3500					0.0145
4000	748	1272	922	1422	0.016
4500					0.017
5000					0.0185
5500					0.0210
6000					0.0240
6500					0.0295
6600					*

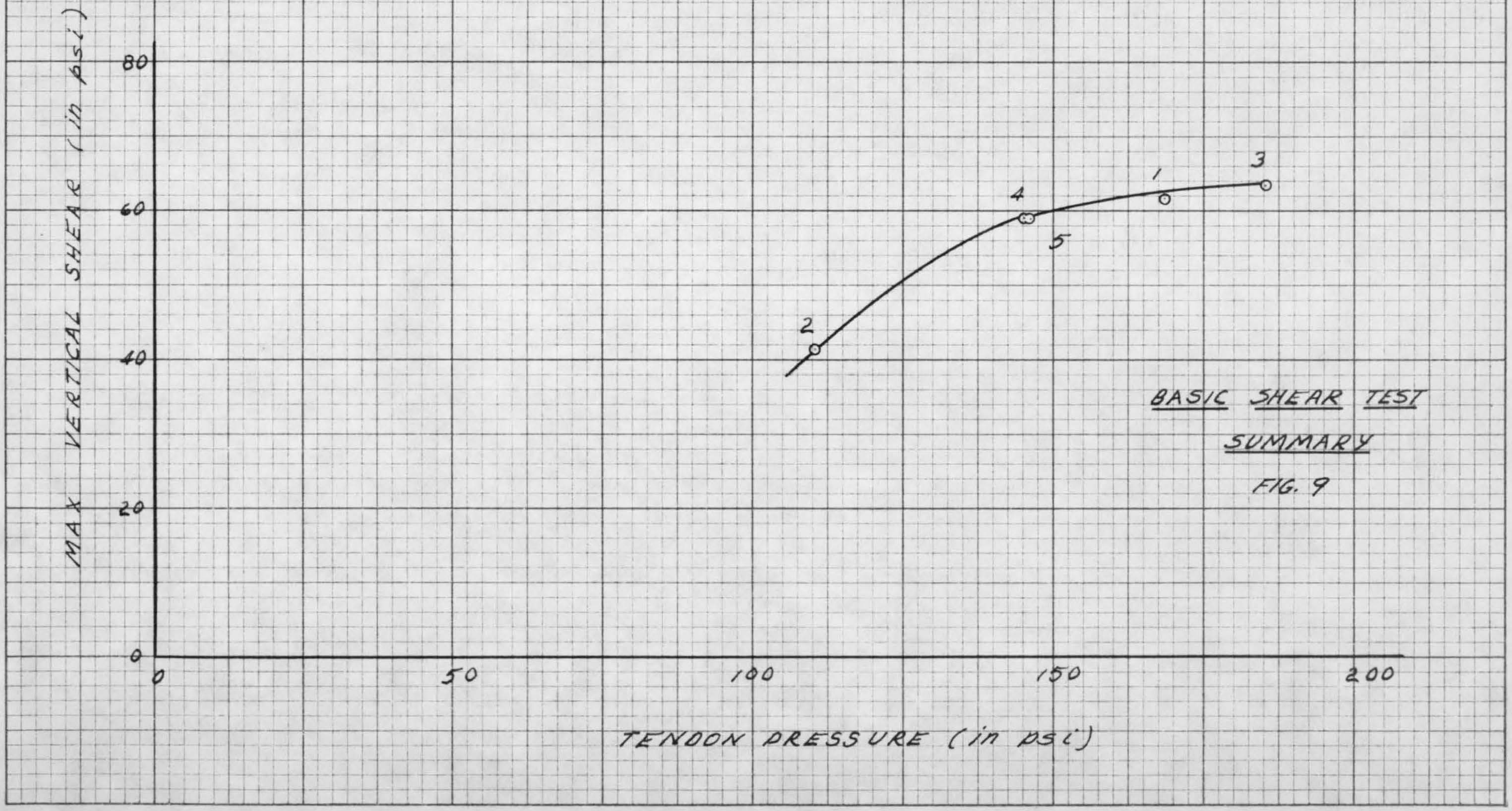
* Dial increasing rapidly without load increase.



BASIC SHEAR TEST

PANEL NO. 5

FIG. 8

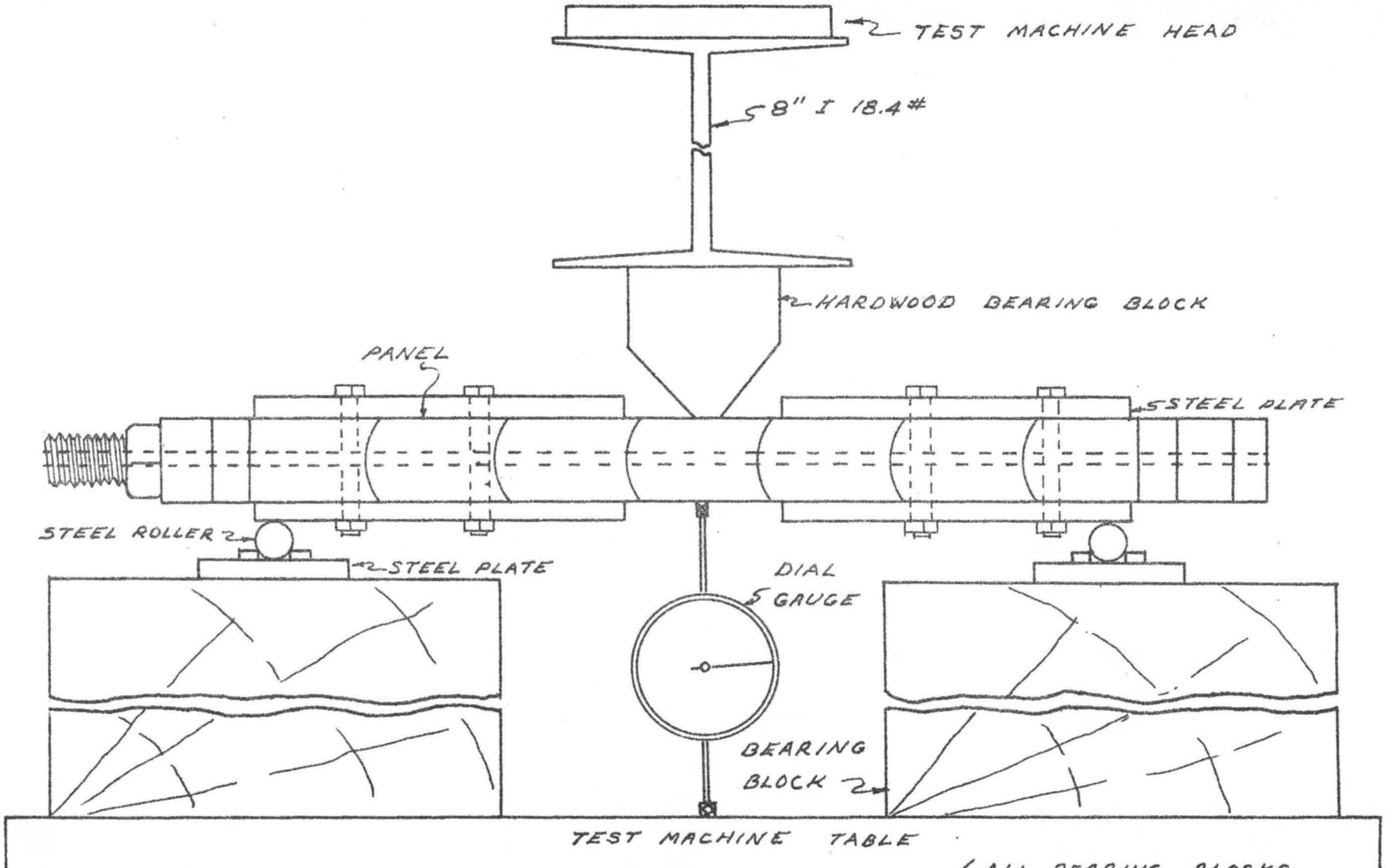


BASIC SHEAR TEST
SUMMARY
FIG. 9

Panels were constructed and a test setup as shown in Fig. 10 was used to obtain the moment resistance for bending about the longitudinal axis. An effective span of 11 inches was used in the tests. Load was applied for the full length of the panel using the heavy beam to distribute the load. The panels were stressed to a given level and then the bending load was applied in increments. A typical set of data of the dial movement versus the load applied is shown in the Table 3. As seen in the table the tendon pressure does not increase during the bending. It is felt that the position of the tendon on the neutral axis accounts for this fact.

A graph of the dial deflection versus load applied is shown in Fig. 11. It can easily be seen that the action of the panel was quite linear up to a certain load level. At this load there was a sudden large deflection brought on by rotation at the joints. This point was used as the useful limit of moment resistance.

A graph of the moment resistance per lineal inch versus the applied prestress is shown in Fig. 12 for the four tests run. The bending resistance versus tendon pressure is taken as a linear function although some scatter exists in the data. It is felt this variation is due primarily to wood properties. At a stress level of 175 psi the maximum bending resistance is about 64 in-lbs per inch of length. Translated into a flat panel of 5 foot span, the maximum carrying capacity of the panel in terms of moment resistance is about 20 psf. From this example it is seen that this action on the panel is quite serious. The bending resistance in this direction will limit the usefulness of the system in many cases. However, if the main bending of the structure is about the transverse axis rather than the longitudinal axis this limitation will not be so severe.



(ALL BEARING BLOCKS, PLATES, & ROLLERS EXTEND FULL LENGTH)

BENDING TEST

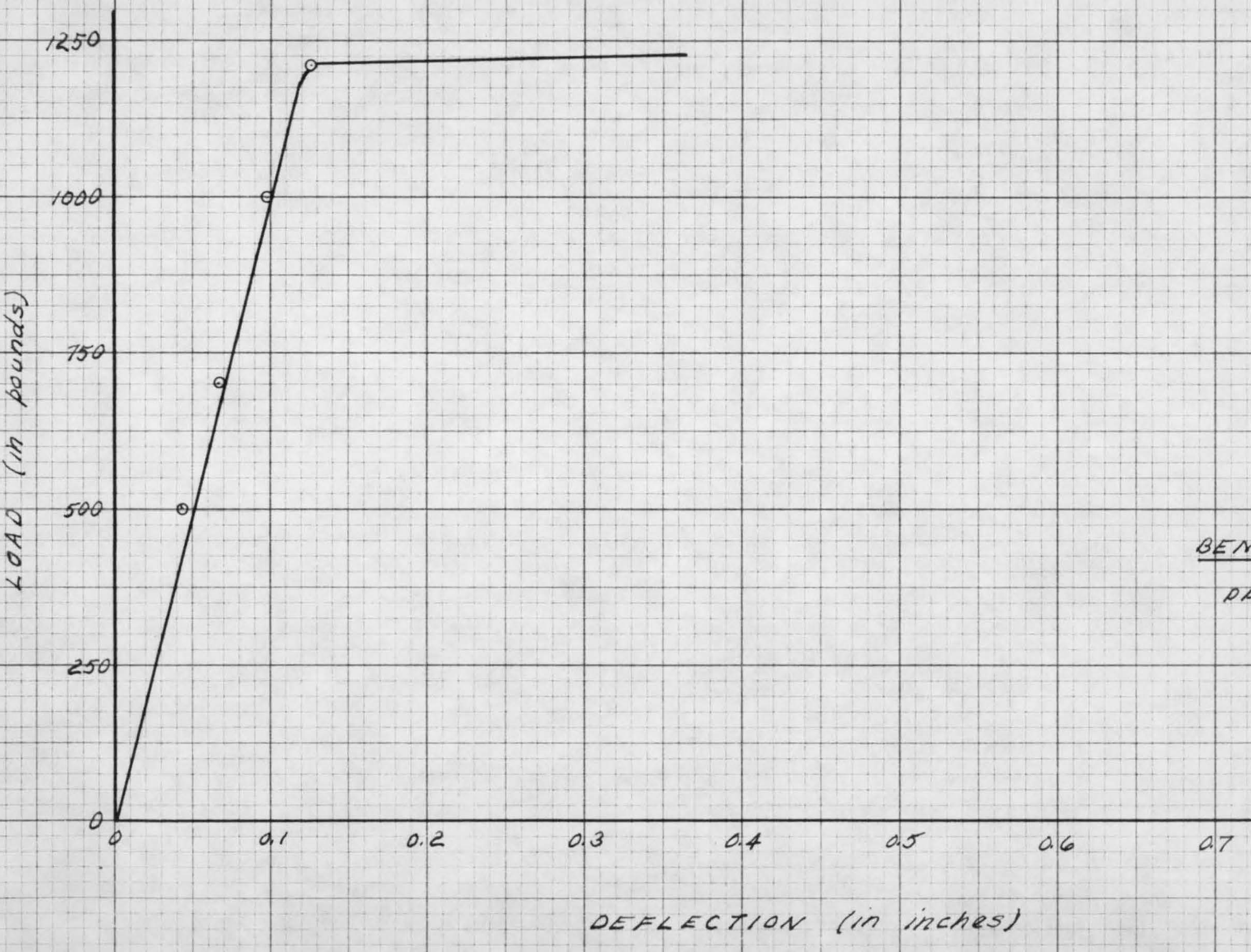
FIG. 10

TABLE 3

BENDING TEST NO. 1

LOAD (lbs)	STRAIN (in microinches) GAUGE NO. 1	GAUGE NO. 2	GAUGE NO. 3	GAUGE NO. 4	Δ DIAL (in. inches)
Zero read	395	1085	840	1160	
Stressed	635	1350	1140	1495	
0	635	1350	1140	1495	0
* 500					0.045
* 700					0.069
1000	625	1335	1120	1485	0.100
*1200					0.128
900	600	1260	1085	1450	0.679

* (Strain readings omitted due to small amount of change)

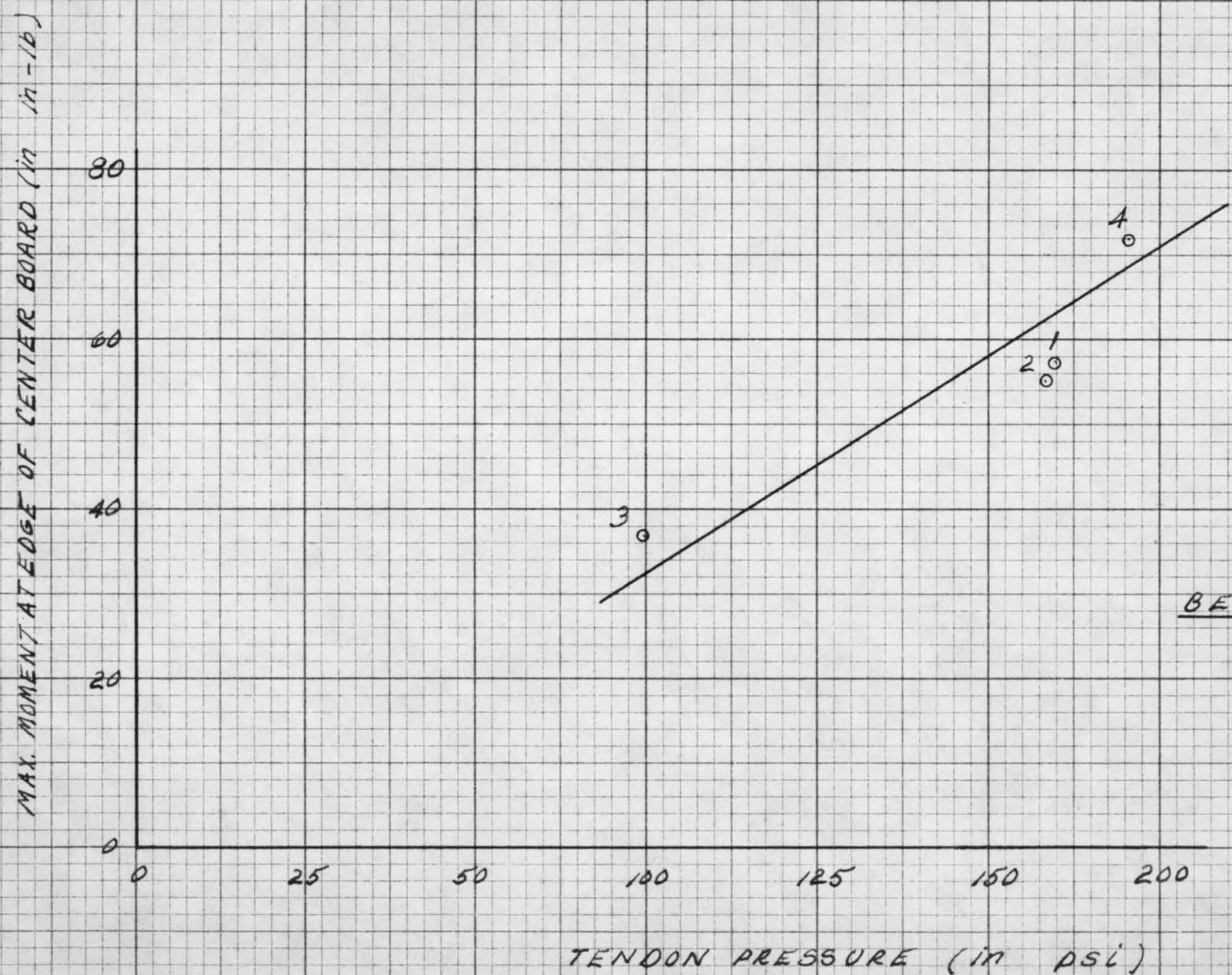


BENDING TEST

PANEL NO. 1

FIG. 11

K+E 10 X 10 TO THE INCH KEUFFEL & ESSER CO. 359-5DG MADE IN U.S.A.



BENDING TEST SUMMARY

FIG. 12

VI. SUMMARY AND CONCLUSIONS CONCERNING BASIC STRENGTH TESTS

To make full use of the capacity of the wood the tests indicate that a higher initial prestress force should be used. A prestress level of about 300-320 psi is recommended. This value of prestress would use only about 40 percent of the proportional limit capacity of the wood (perpendicular to the grain). This should produce a general increase in all strength properties of the system.

In particular the bending value of the system (transverse bending) is quite small. The results of the bending tests indicate that moment resistance is essentially a linear function of wood prestress. A direct increase in bending strength with increased prestress should be obtainable. The ultimate bending of about 64 in-lbs per inch at 175 psi would rise to about 117 in-lbs per inch at 320 psi prestress. This change may be very significant in structural applications.

In basic shear (friction) the increase brought about by increased prestress is more problematical. From the summary graph of the basic shear tests it appears that the increase in friction as a result of increased prestress will not be as great as in the bending test. This should be checked by further testing. The limit of shear is about 64 psi at 175 psi prestress. If in structural applications the shear governs it may be necessary to increase the wood area to overcome this limitation.

Other test results indicate no serious limitations of the system. The individual wood members are well locked together as indicated by the line load shear tests.

It should be indicated that under any bending situation producing stresses parallel to the grain, the usual allowable stress values for tension and compression will give the maximum values of structural system. These values are not affected by the prestressed system proposed.

VII. TESTS TO DETERMINE THE LOSS OF PRESTRESS WITH TIME

All of the structural strength tests were conducted at predetermined wood stress levels. These tests were conducted immediately after the tendons were tightened. In order to rely upon the design stresses obtained the tendon force must be maintained.

There are two possible causes of loss of the tendon force. One cause is creep in the wood. Creep is an inelastic strain at a constant stress. In this system the direction of prestress is perpendicular to grain. Creep is known to be more serious in this wood direction than parallel to grain. Creep in wood is also known to be a function of both moisture content and temperature. The magnitude of creep generally increases as moisture content increases and also as temperature increases.

In this system a second cause of loss in tendon force could be a change in moisture content. As the moisture content decreases, wood shrinks. In this type of prestressed system such shrinkage would result in a decrease in tendon (and wood) stress. Some change in the moisture content of wood in use is inevitable. The moisture content of framing members fluctuates during the year. In most areas of the country this change is in the magnitude of 2-3 percent moisture content. Framing members in different parts of a structure also attain different moisture contents. Fluctuation in moisture content is thus unavoidable and must be considered in the analysis of this system for permanent structures.

A series of three tests was set up to determine the effect of creep and moisture content change on the tendon force. Unfortunately no controlled environment chamber was available for conducting these tests, and therefore there was a fluctuation in moisture content during all three tests. The effect of creep and moisture content, therefore, cannot be entirely separated.

The general test setup is shown in plate 5 and 6. The panel used here was nearly the same as for the structural tests. The side plates differed in that restraining bars were designed to hold the panel flat during the tests. It was found that these bars were not needed. A dial gauge was fixed to one of the bars to give a reading of the change in total panel width during the test. The dial gauge can be seen in the figure. The tendon force was measured with the same transducer system as was used during the structural strength tests. To overcome the problem of zero drift during the long duration of these tests, two readings were taken with the strain indicator each time data was collected. One reading was taken in the usual manner and one with the active and compensating gauges reversed. In this way a new gauge zero was obtained each time data was collected.

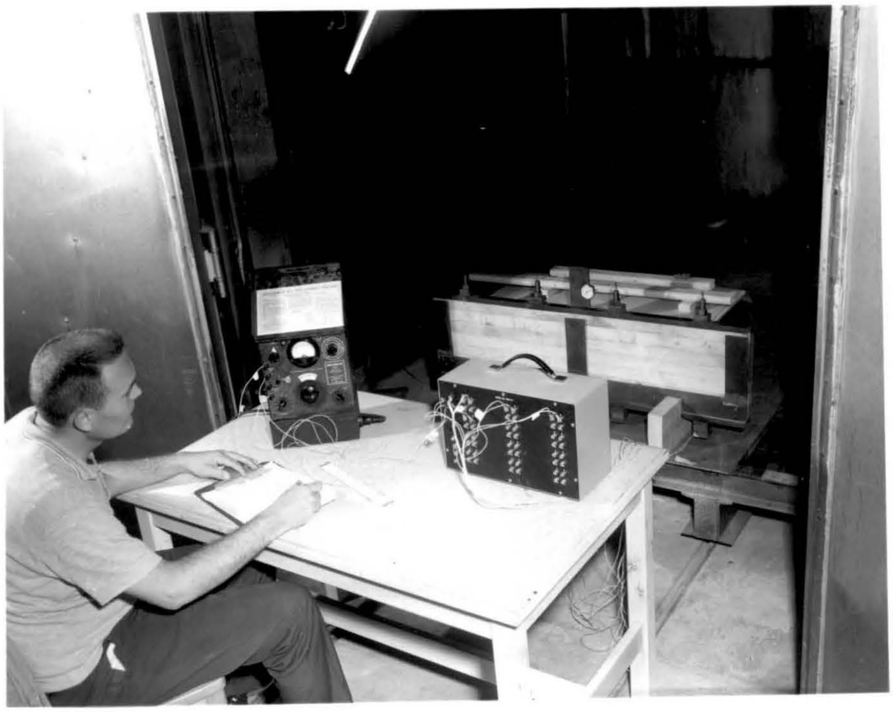
The three tests will be described briefly. Data from these tests may be found in Tables 4, 5, and 6. In Test One the panel was assembled with the wood at approximately 8 percent moisture content. The panel was kept at room conditions for about four days. It was then put in a chamber at approximately 120°F and 90 percent relative humidity. After about four days in this atmosphere the moisture content had increased to 11 percent. After fourteen days in this atmosphere the panel was moved again to room conditions and allowed to dry out. The moisture content dropped to 7.5 percent. See Fig. 13.

Test Two was designed to determine creep without the effect of moisture content change. However, the equilibrium moisture content in the laboratory dropped during the test causing a corresponding drying out of the panel. See Fig. 14. The fluctuation of moisture content during this forty day test was between 7.5 and 6.5 percent moisture content. Normal room temperatures of 65° to 85°F. existed during this test.

For Test Three the boards were dried to 5 percent moisture content before the panel was assembled. After assembly the panel was left in the laboratory and allowed to come to equilibrium conditions of about 6 percent. After 26 days the panel was placed in a kiln and dried at 120°F to 2.5 percent moisture content.

The stress on the wood and the moisture content during these tests is shown in Figs. 13, 14 and 15. In all three tests it can be seen that there is an immediate drop in stress after the panel is assembled. This drop is due to initial creep and perhaps working of the edges of the strips into intimate contact. This initial creep seems to continue for from 1 to 2 days after initial tensioning. It appears that at higher initial moisture contents the initial creep is higher. This relationship is shown in Fig. 16.

After the period of initial creep in Tests One and Two, the stress continued to slowly drop. Most of this drop seems to be associated with



MOISTURE TEST PANEL IN KILN

PLATE 5

CREEP TEST PANEL

PLATE 6

moisture content loss. In Test Three the moisture content is steady after the first three days of drying. In this test it can be seen (Fig. 15) that the stress level also remained essentially constant.

It should be noted that when the moisture content was increased there was little if any increase in stress. Presumably this was due to the fact that creep also increased thus nullifying the swelling effect. However, when moisture content was decreased a very abrupt loss in stress resulted. The decrease in wood stress ranged between 20-40 psi percent change in moisture content. A five percent change in moisture content could thus evidently be enough to cause a loss of almost all the prestress.

The presence of long time creep is difficult to detect in these tests. If it is present it is masked by moisture content changes. In Test Three however, it would appear that very little long time creep occurred. The magnitude of long time creep should be studied further if this system is to be considered for permanent structures.

In these tests the total panel strains from the combination of creep and moisture change amounted to 4000 micro-in/in in Test One, 2000 micro-in/in in Test Two, and 3200 micro-in/in in Test Three. In future work on the creep problem, approximately 4000 micro-in/in could be considered as the maximum amount of creep likely to be encountered.

VIII. CONCLUSIONS FROM CREEP AND MOISTURE TESTS

1. A significant initial loss of prestress is to be expected. The higher the moisture content of the wood at the time of assembly the greater will be this initial stress relaxation.

2. After about two days the rate of creep is negligible. It may proceed at a very slow rate, but in these tests was not serious enough to show up.

3. An increase in moisture content causes little if any increase in stress. Creep is evidently accelerated thus counteracting the swelling effect.

4. A decrease in moisture content causes a loss in wood stress of 20-40 psi per percent moisture content change.

TABLE 4

PANEL NO. I

Elapsed time (Days)(hrs)	Panel width (in) change	Moisture content	(in microinches) Gauge Strains 1	2	3	4	Ave. E
0	0	8%	335	335	335	335	335
0 - 1	-0.0009	8%					322
0 - 2	-0.0025	8%	281	281	284	286	283
0 - 8	-0.0038	8%	251	271	271	269	265
0 -19	-0.0046	8%	246	269	269	266	262
1 - 1	-0.0065	8%	231	259	259	258	252
1 -20	-0.0072	8%	235	256	266	253	253
2 -22	-0.0090	8%	223	250	264	258	249
3 -19	-0.0083	8%	219	245	260	256	245
3 -20	-0.0087	7.5	222	252	259	256	247
3 -22	-0.0112	7.3	309	192	214	197	228
4 -01	-0.0168	7.0	129	179	194	183	171
4 -03	-0.0170	7.3	138	184	195	190	177
4 -06	-0.0162	8.1	141	179	200	192	178
4 -18	-0.0175	8.5	123	151	181	173	157
5 - 2	-0.0175	8.5	109	138	178	163	147
5 -19	-0.0238	8.6	83	111	144	136	118
5 -21	-0.0205	9.5	103	123	170	158	139
6 - 2	-0.0190	9.8	98	134	166	160	140
6 -19	-0.0205	10.2	89	104	161	145	125
7 - 2	-0.0208	10.5	92	117	165	149	130
7 -19	-0.0218	10.2	70	103	162	137	118
8 -21	-0.0207	11.2	93	64	176	141	118
9 -23	-0.0210	11.2	68	Gauge	276	179	180
11 - 2	-0.0265*	11.2	75	fail-	202	156	145
12 - 2	-0.0265*	10.9	75	ure	214	152	147
13 - 4	-0.0265*	10.5	81		245	167	165
13 -22	-0.0265*	11.0	121		320	181	207
14 -21	-0.0265*	10.5	90		275	171	180
18 -19	-0.0265*	11.0					
20 -21		9.2	70		37	57	55
22 - 0	-0.0745	8.4	47		15	42	35
22 -21	-0.0760	8.3	42		17	32	30
25 -18	-0.0910	7.6					
26 -18	-0.0940	7.4					

*Dial frozen

TABLE 5

PANEL NO. 2

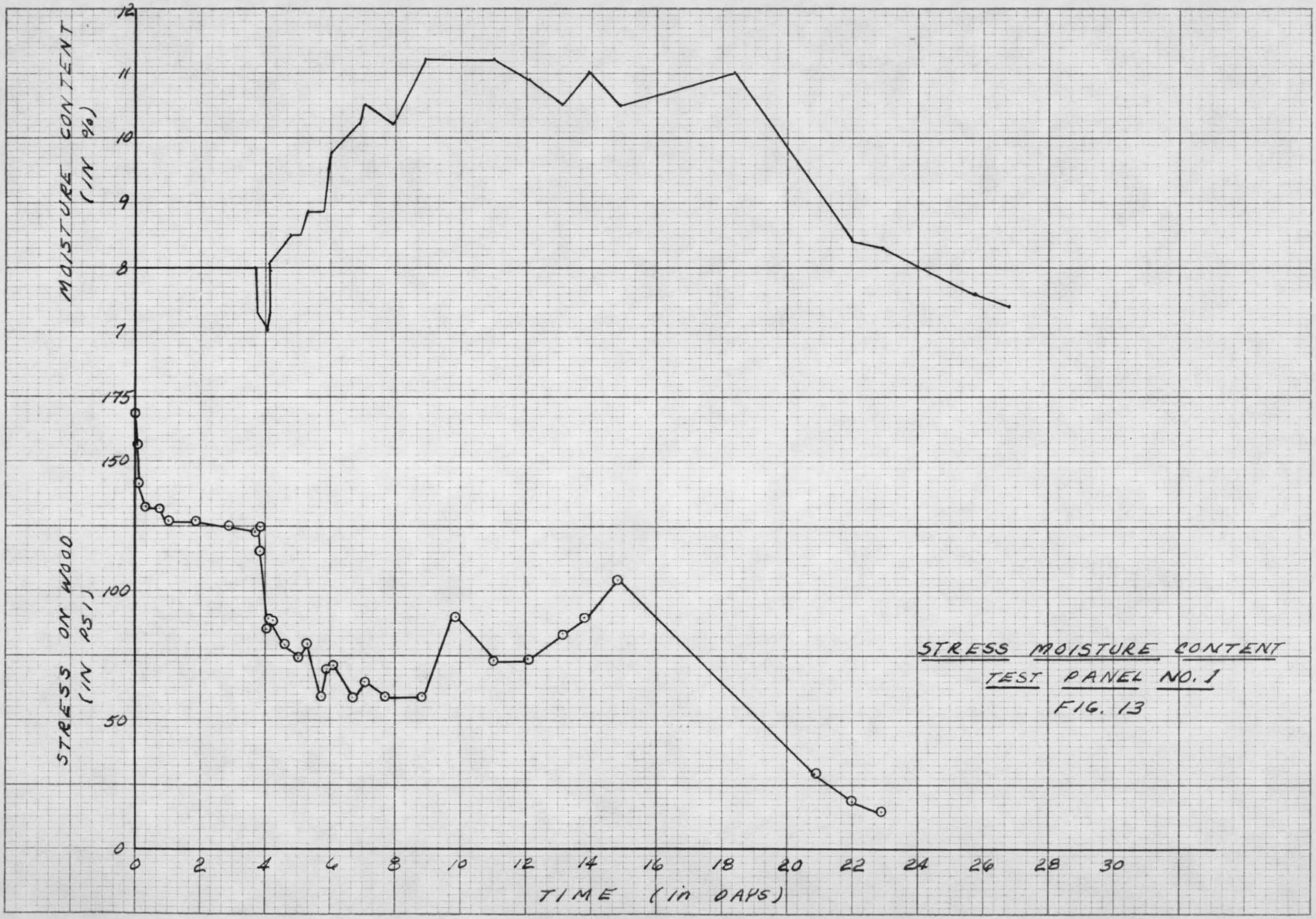
Elapsed time (Days)(hrs)	Panel width (in) change	Moisture content	(in microinches) Gauge Strains 1	2	3	4	Ave. E
0	0	7.34%	326	316	319	302	315
0 - 6	0.0029	7.34%	307	300	295	268	293
0 -16	0.0042	7.34%	295	295	290	266	286
1 - 0	0.0061	7.34%	300	304	323	286	297
1 -18	0.0069	7.34%	291	295	280	251	278
2 -19	0.0071	7.49	299	306	290	265	290
5 -17	0.0101	7.55	272	278	261		270
6 -19	0.01015	7.78	275	293	275	258	275
7 -23	0.0101	7.55	275	293	276	252	274
8 -18	0.0114	7.26	265	287	259	242	263
11 -16	0.0165	7.20	246	273	238	217	244
13 -19	0.0181	6.82	236	273	254	221	246
14 -16	0.0202	6.82	222	261	232	202	230
15 -16	0.0206	6.80	216	252	229	197	223
16 -18	0.0244	6.46	207	248	225	183	216
18 -17	0.0231	6.70	209	253	220	189	218
20 -19	0.0249	6.53	212	251	222	184	217
22 -17	0.0251	6.82	202	242	220	182	212
26 -17	0.0249	6.54	200	248	222	188	216
32 -19	0.0245	6.33	202	251	228	198	220
26 - 1	0.0247	6.96	195	246	217	188	211
40 -18	0.0244	6.80	190	246	217	183	208

TABLE 6

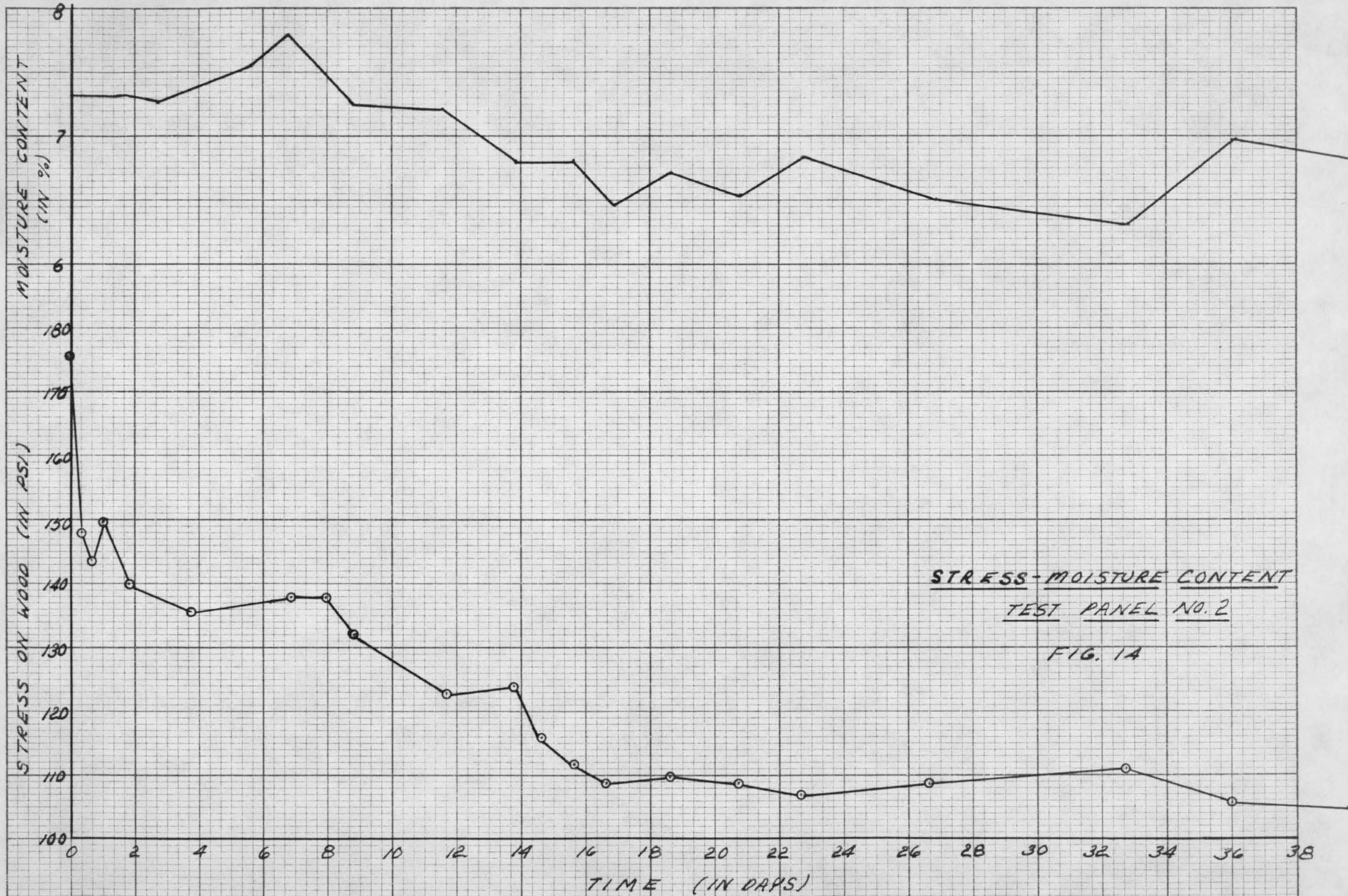
PANEL NO. 3

Elapsed time (Days)(hrs)	Panel width (in) change	Moisture content	(in microinches) Gauge Strains 1	2	3	4	Ave. E
0	0	4.92%					336
0 - 1	0.0025	5.07	300	310	285	302	300
0 - 6	0.0050	4.88	287	295	277	293	287
0 -23	0.0075	5.10	290	290	286	315	295
2 -22	0.0040	5.88	297	280	285	315	295
5 - 0	0.0052	5.71	285	272	285	309	288
6 -22	0.0042	5.92	275	270	285	324	289
10 -23	0.0028	6.12	308	275	290	324	299
17 - 0	0.0016	5.98	322	275	282	301	295
20 - 6	0.0007	6.47	320	272	287	293	293
24 -23	0.0028	6.12	281	250	275	327	284
25 -22	0.0250	3.37	145	143	180	142	155
26 -22	0.0350	2.44	95	75	107	117	98
27 -22	0.0390	2.40	45	60	100	145	100

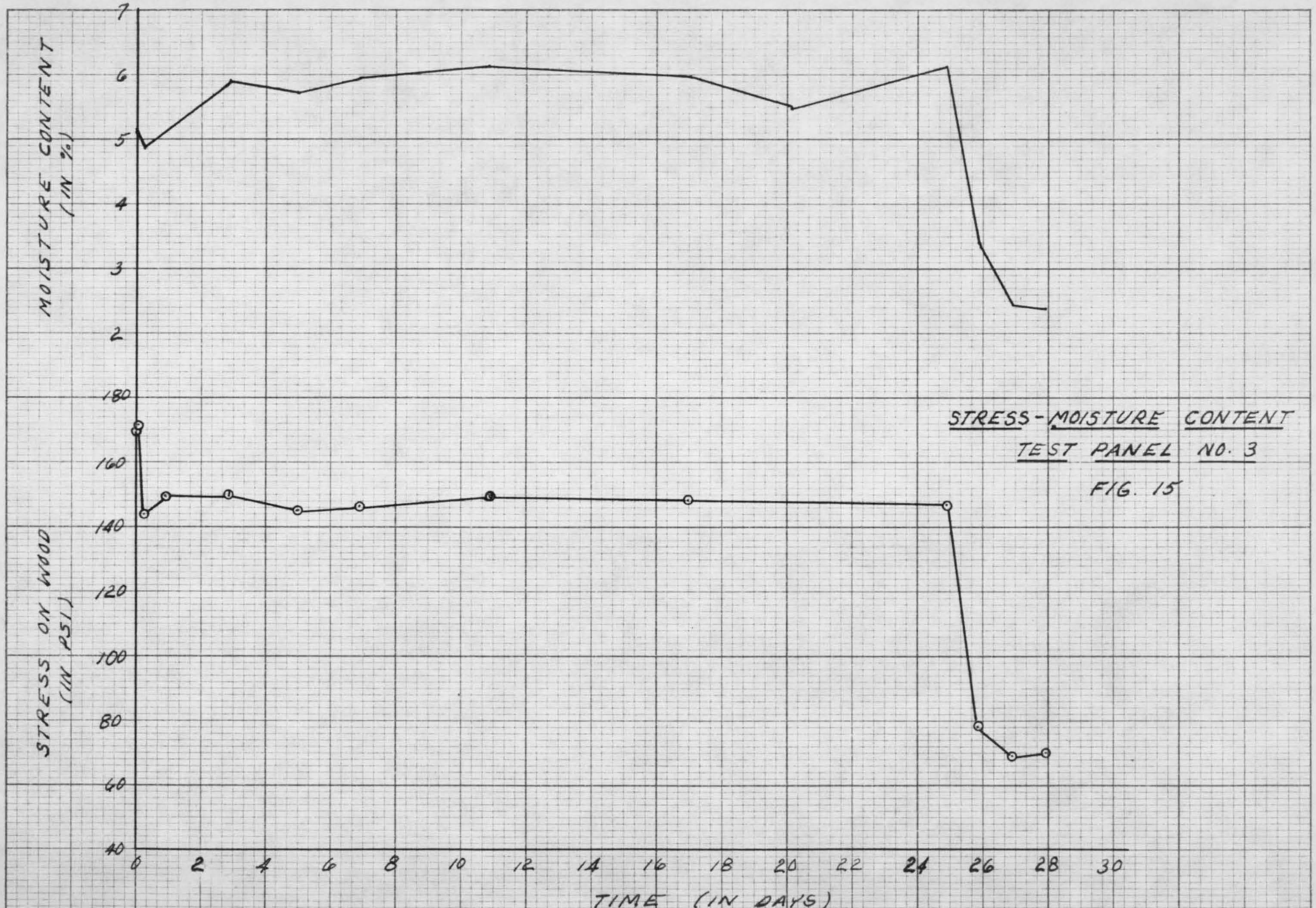
K+E 12 X 20 TO THE INCH 359-21
KEUFFEL & ESSER CO. MADE IN U.S.A.



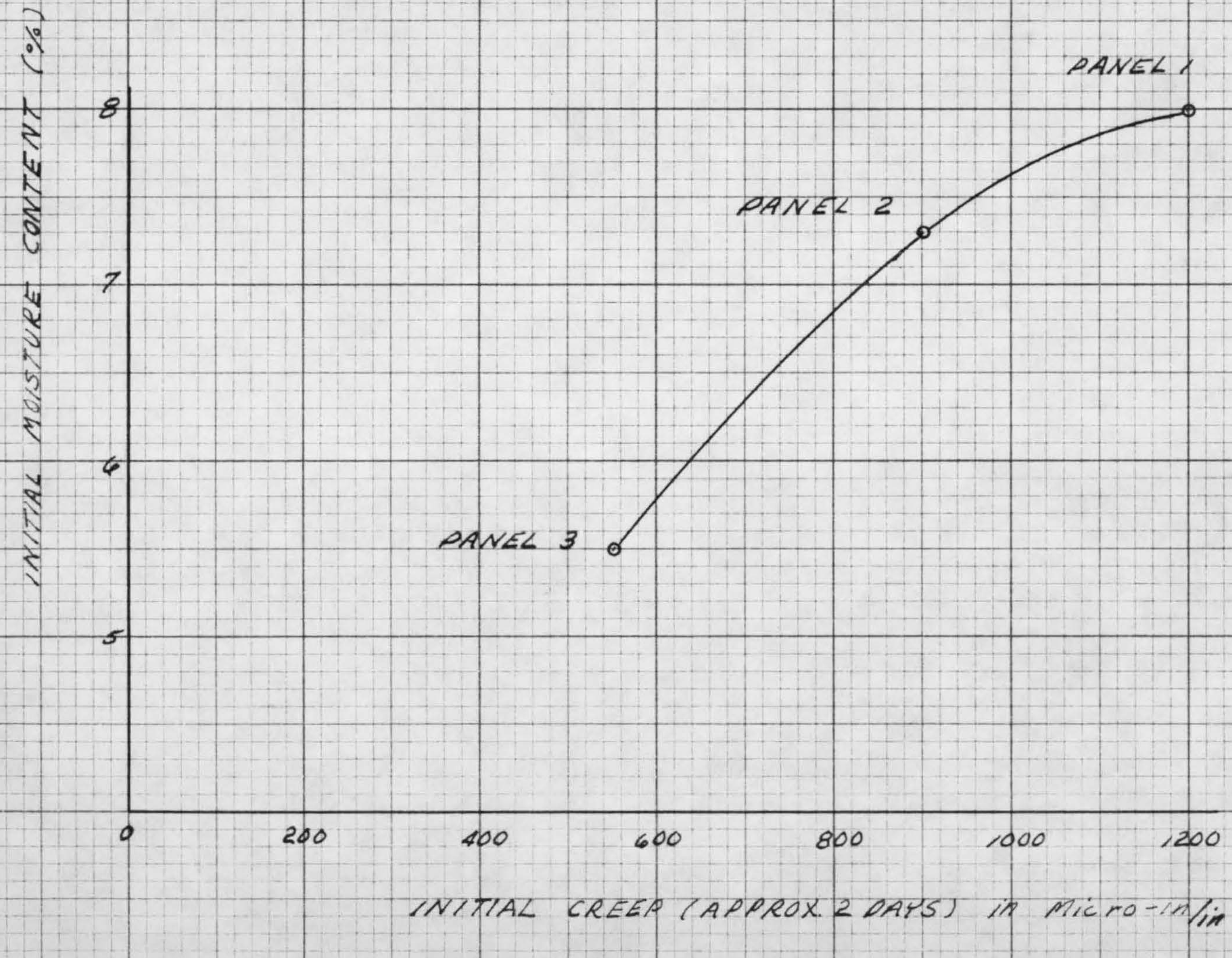
STRESS MOISTURE CONTENT
TEST PANEL NO. 1
FIG. 13



STRESS-MOISTURE CONTENT
TEST PANEL NO. 2
FIG. 14



STRESS-MOISTURE CONTENT
TEST PANEL NO. 3
FIG. 15



EFFECT OF MOISTURE CONTENT ON INITIAL CREEP

FIG. 16

5. Repeated cycling of moisture increase then decrease could be expected to cause continual decrease in stress level. However, in this study no repeated cycling tests were conducted. This effect is only significant when the system is used for a permanent structure.

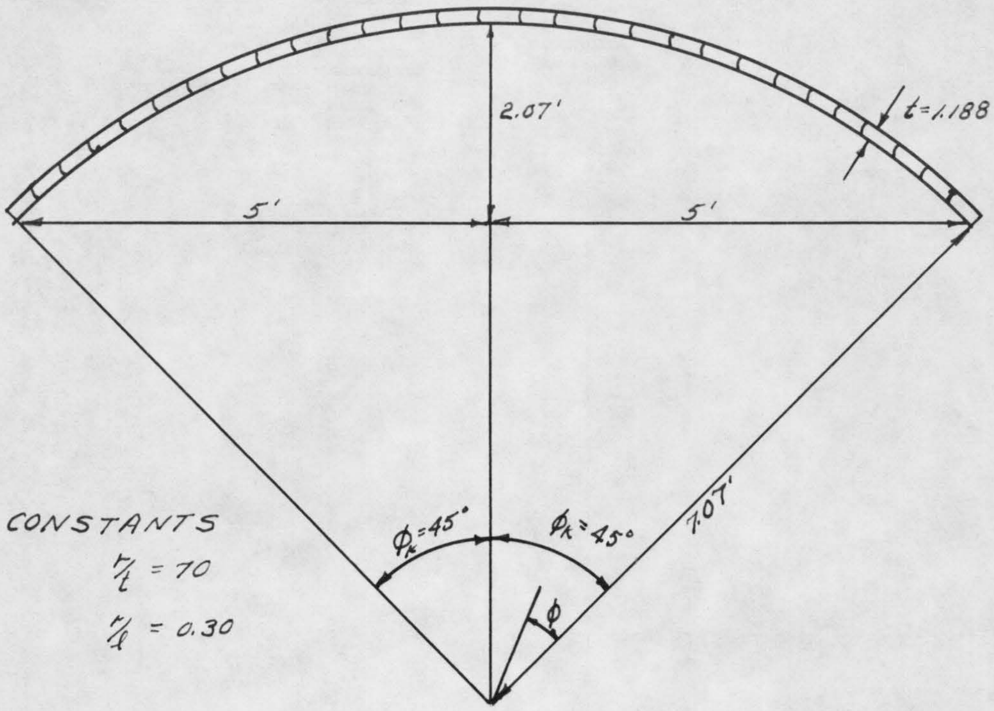
6. To make the system workable a spring loaded tendon system is suggested. This system would reduce the loss of prestress due to moisture content decrease or creep. It would also allow calibration of the amount of initial prestress by direct measure of spring deflection.

IX. SHELL ANALYSIS AND INVESTIGATION

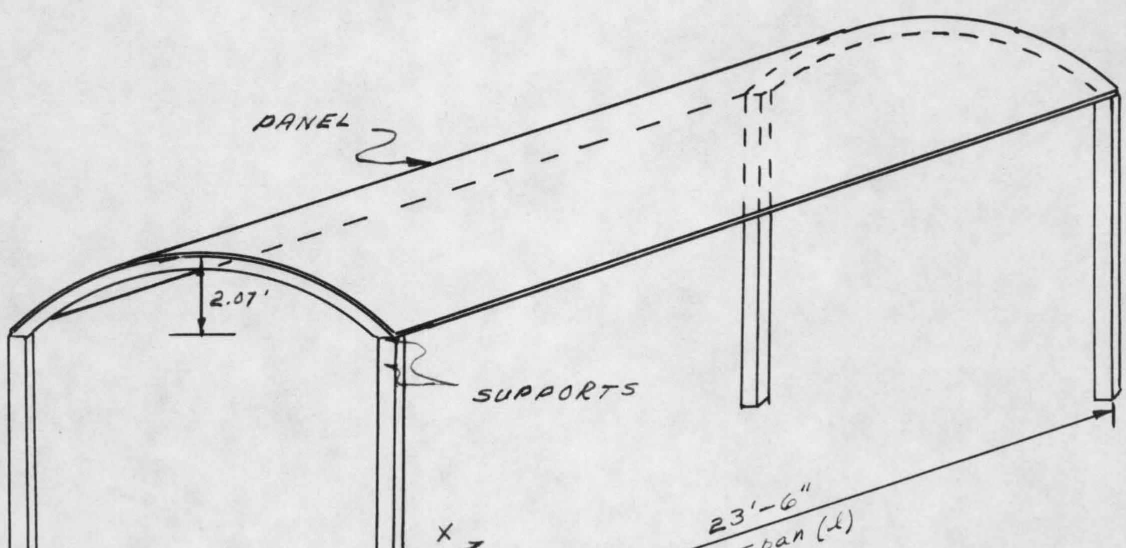
To make an estimate of the capability of the structural system, two barrel shell designs are presented. The shells chosen are representative of the two types used, one is a long span shell and one is a short span shell (referring to the assumptions made in analysis). The shells have been chosen arbitrarily but are representative of the span lengths and chord widths which may prove to be desirable. The shell dimensions and cross-sections are shown in Figs. 17 and 18.

Dimensions of the shells have been chosen so that the design tables of ASCE Manual 31, "Design of Cylindrical Concrete Shell Roofs" may be used. The assumptions used in the analysis are:

1. The shell acts elastically and the individual members are locked together so that complete membrane action is obtained.
2. Prestress effects are considered as internal forces for analysis purposes. Final forces are obtained by combining shell stresses with prestress effects.
3. The design forces may be obtained by usual elastic procedures.
4. The portions of the shells subjected to high longitudinal tensile forces are considered to be made of strips which are continuous in the longitudinal direction.
5. The shells are simply supported and have no edge supports in the longitudinal direction.
6. Design is made for a surface load such as dead load and a live load such as wet concrete.



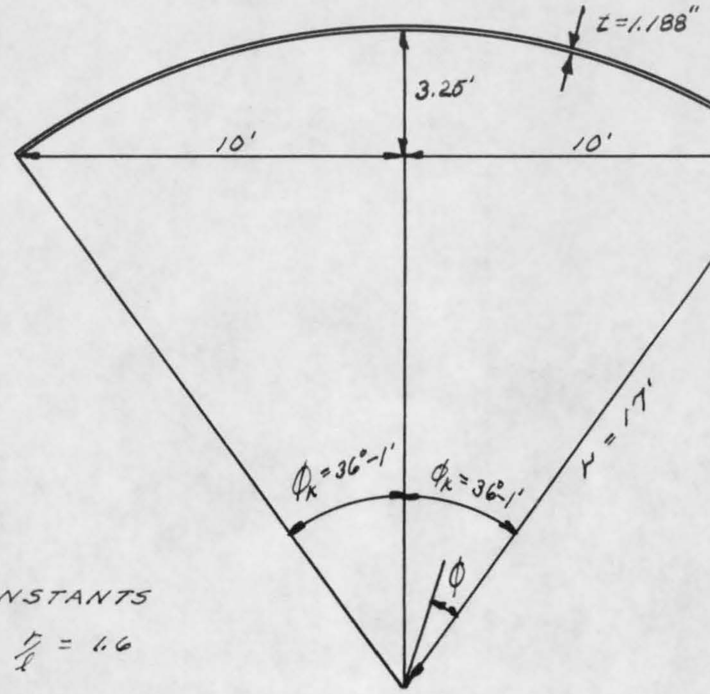
CROSS-SECTION
(Scale 1" = 2')



GENERAL SKETCH
(Scale 1" = 5')

LONG SPAN SHELL

Fig. 17



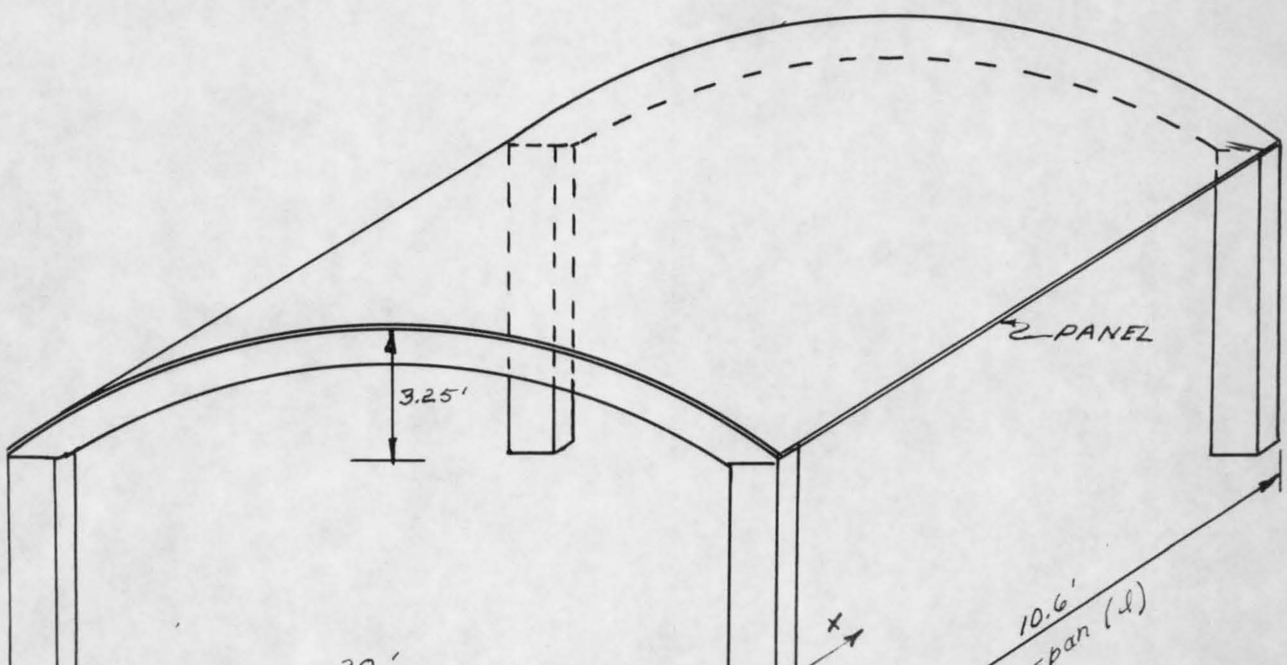
CONSTANTS

$\frac{r}{\ell} = 1.6$

$\frac{rt}{\ell^2} = 0.015$

$\sqrt[4]{r+\ell^2} = 3.71$

CROSS - SECTION
(scale 1" = 5')

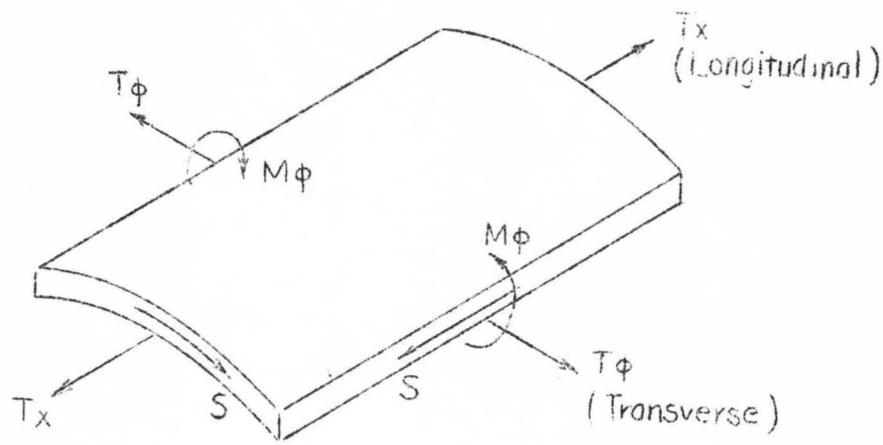


GENERAL SKETCH
(scale 1" = 5')

SHORT SPAN SHELL

Fig 18

To aid in the understanding of the analysis a typical element of the shell is shown in Fig. 19 with the primary stresses and moments shown acting on the element. The directions shown are considered positive.



TYPICAL SHELL ELEMENT
Figure 19

Internal shell forces and moments are calculated only for the locations where the maximum values exist. The shell analysis for the two cases chosen is shown in Tables 7 and 8. The final forces from the individual analyses are shown in the tables.

The critical values for each shell type are given in Table 9.

TABLE 7

INTERNAL FORCE TABLE FOR LONG BARREL SHELL

φ φk-φ	45 0	30 15	20 25	10 35	0 45	φ φk-φ	45 0	30 15	20 25	10 35	0 45
Load	Coefficients					Multiplier	Forces				
$T\phi$ @ x = $\ell/2$ (lbs. per ft.)											
						0	-	-	-	-	-
D.L. + L.L.	-1.000	-0.9659	-.9063	-0.8191	-0.7071	9w	-9.00w	-8.70w	-8.16w	-7.36w	-6.36w
V_L	-1.178	-1.454	-1.326	-0.330	+0.707	4.5w	-5.30w	-6.55w	-5.96w	-1.49w	+3.18w
H_L	+0.3055	+0.8385	+1.279	+1.158	+0.7071	4.5w	+1.37w	+3.78w	+5.75w	+5.21w	+3.18w
S_L	+0.0719	+0.0328	+0.0241	+0.0645	0	13.5w	+0.97w	-0.44w	+0.32w	+0.87w	0
	Final Forces						-11.96w	-11.91w	-8.05w	-2.77w	0
S @ x = 0 (lbs. per ft.)											
D.L. + L.L.	0	-0.1648	-0.2690	-0.3652	-0.4502	30w	0	-4.94w	-8.06w	-10.95w	-13.5w
V_L	0	+0.432	-0.788	-2.214	0	15w	0	+6.47w	-11.82w	-33.20w	0
H_L	0	-1.061	-0.5051	+0.6328	0	15w	0	-15.92w	-7.58w	+9.49w	0
S_L	0	-0.0851	-0.1013	-0.0128	+0.3000	45w	0	-3.83w	-4.56w	-0.58w	+13.5w
	Final Forces						0	-18.32w	-32.02w	-35.24w	0

TABLE 7

INTERNAL FORCE TABLE FOR LONG BARREL SHELL (Continued)

Tx @ x = $\ell/2$ (lbs. per ft.)											
D.L. + L.L.	-0.2026	-0.1957	-0.1837	-0.1660	-0.1433	100w	-20.3w	-19.6w	-18.4w	-16.6w	-14.3w
V_L	+1.400	-1.040	-3.110	-0.946	+11.24	50w	+70.0w	-52.0w	-155.0w	-47.3w	+562.0w
H_L	-1.970	-0.0472	+1.941	+1.513	-5.257	50w	-98.5w	-2.4w	+97.0w	+75.7w	-263.0w
S_L	-0.1142	-0.0776	+0.0382	+0.3209	+0.8743	150w	-17.2w	-11.7w	+5.7w	+48.1w	+131.0w
	Final Force						-66.0w	-85.7w	-70.7w	+59.9w	+415.7w
φ φk-φ	45 0	30 15	20 25	10 35	0 45	φ φk-φ	45 0	30 15	20 25	10 35	0 45
Load	Coefficients					Multiplier	Forces				
Mφ @ x = $\ell/2$ (ft.-lbs/ft.)											
V_L	-0.2565	-0.2296	-0.1743	-0.0899	0	31.8w	-8.16w	-7.30w	-5.55w	-2.86w	0
H_L	+0.1591	+0.1550	+0.1338	+0.0821	0	31.8w	+5.06w	+4.94w	+4.25w	+2.61w	0
S_L	-0.0059	-0.0040	-0.0016	-0.0001	0	95.5w	-0.56w	-0.38w	-0.15	-0.10w	0
	Final Force						-3.66w	-2.74w	-1.45w	-0.35	

TABLE 8

INTERNAL FORCE TABLE FOR SHORT BARREL SHELL

$\frac{S}{\sqrt[4]{r't\,\ell^2}}$	3.2	1.6	0.8	0.4	0.2	0		3.2	1.6	1.8	0.4	0.2	0
S	11.85	5.84	2.97	1.488	0.74	0		11.85	5.84	2.97	1.488	0.74	0
ϕ	40°	19.65°	10.02°	5.01°	2.49°	0°		40°	19.65°	10.02°	5.01°	2.49°	0°
$\phi k-\phi$	-4°	16.35°	26°	31°	33.5°	36°		-4°	16.35°	26°	31°	33.5°	36°
			Coefficients				Multiplier			Forces			
				$T\phi$ @ $x = \ell/2$ (lbs. per ft.)									
D.L. + L.L.		-.9588	-.8982	-.8572	-.8333	-.8085	21.65w		-20.75w	-19.45w	-18.55w	-1800w	-17.5w
T_L	+.055	-.255	-.236	+.416	+.803	+1.000	17.5w	+.96w	-4.55w	-4.13w	+7.29w	+14.05w	+17.5w
S_L	+.0146	-.0657	+.0112	+.1242	+.1242	0	5.05w	+.07w	-.33w	+.06w	.63w	+ .63w	0
Σ								1.03w	-25.63w	-23.52w	-10.63w	-3.32w	0
Forces due to line loads @ Far Edge									+1.70w	+1.00w	+.60w	+.30w	0
Final Forces									-23.93w	-22.52w	-10.03w	-3.02w	0
				S @ x = 0 (lbs. per ft.)									
D.L. + L.L.		-.1749	-.2789	-.3277	-.3511	-.3740	13.5w		-2.36w	-3.76	-4.42w	-4.73w	-5.05w
T_L	.115	+.505	-.952	-1.845	-1.529	0	17.5w	-2.01w	+8.00w	-16.65w	-32.00w	-26.50w	0
S_L	-.016	+.059	-.260	-.160	+.214	-1.000	5.05w	-.09w	+.30w	-1.31w	-.81w	+1.08w	+5.05w
Σ								-2.10w	+5.94w	-21.72w	-37.23w	-30.15w	0
Forces due to line loads @ Far Edge									+2.20w	+1.60w	+1.00w	+.50w	0
Final Forces									+8.14w	-20.12w	-36.23w	-29.65w	0

TABLE 8

INTERNAL FORCE TABLE FOR SHORT BARREL SHELL (Continued)

						Tx @ $\ell/2$ (lbs. per ft.)							
D.L. + L.L.		-.1943	-.1821	-.1735	-.1688	-.1638	8.4w		-1.63w	-1.53w	-1.46w	-1.42w	-1.37w
T_L	+.011	-.243	-2.753	.371	+3.678	+10.843	17.5w	+.19w	-4.25w	-48.20w	-6.5w	+64.35w	+189.8w
S_L	+.030	-.187	-.284	+1.023	+2.498	+4.811	5.05w	+.15w	-.94w	-1.43w	+5.17w	+12.61w	+24.25w
Σ								+.34w	-6.82w	-51.16w	-2.79w	+75.54w	+212.7w
Forces due to line loads @ Far Edge									-8.00w	-8.00w	-4.00w	-2.00w	0
Final Forces									-14.82w	-59.16w	-6.79w	+73.54w	+212.7w
						MΦ @ x = $\ell/2$ (ft-lbs/ft)							
T_L	+.0795	-.4648	-.3476	+.0440	+.1287	0	1.735w	+.13w	-.79w	-.60w	+.08w	+.22w	0
S_L	.0149	-.0980	-.0145	+.0435	+.0352	0	.501w	+.01w	--.05w	-.01w	+.02w	+.02w	0
Σ								+.14w	-.84w	-.61w	+.10w	+.24w	0
Forces due to line loads @ Far Edge									+.19w	+.12w	+.08w	+.06w	0
Final Forces									-.65w	-.49w	-.18w	+.30w	0

TABLE 9

MAXIMUM FORCES IN SHELL EXAMPLES

Long Barrel Shell		Short Barrel Shell	
$T\Phi$	-11.96w* (lbs per ft.)	$T\Phi$	-23.93w (lbs. per ft.)
S	-35.24w (lbs per ft.)	S	-36.23w (lbs. per ft.)
Tx	415.7w (lbs per ft.)	Tx	212.7w (lbs. per ft.)
$M\Phi$	-3.66w (ft-lbs per ft.)	$M\Phi$	-.65w (ft-lbs. per ft.)

* w refers to a uniform dead and live load on shell surface.

The corresponding values of resistance of the structural system to the forces and moments above may be evaluated by examining the structural test ~~tests~~ results. An initial prestress value of 175 psi will be used for the design evaluation.

A. Maximum Value of $T\Phi$

The maximum value of $T\Phi$ may be obtained from the tests made on the wood in compression perpendicular to the grain direction. Using a lower limt value of the proportional limit of 500 psi and an initial prestress of 175 psi, the usable compressive stress is 325 psi. The maximum $T\Phi$ force in lbs per ft. is then (325 psi) (14.2 sq. in. per ft.) gives 4610 lbs. per ft.

B. Maximum Value of S

Referring to the basic shear (friction) tests the value of the longitudinal shear at 175 psi prestress may be obtained as about 65 psi. The maximum shear in lbs. per ft. is obtained as (65 psi) (14.2 sq. in. per ft.) which gives 923 say 925 lbs. per ft.

C. Allowable Value of Tx

The value of Tx is dependent on the allowable tension of the wood parallel to the grain. A conservative value of tension allowable would be about 1300 psi. The allowable force Tx is then (1300 psi) (14.2 sq. in. per ft.) which equals 18450 lbs. per ft.

D. Maximum Value of $M\Phi$

As obtained from the bending tests the maximum moment is about 65 in-lbs per in. or 65 ft-lbs per ft. at 175 psi prestress.

The allowable dead load plus live load which may be placed on the shell for each of the values above may be obtained by dividing the shell forces and moments shown in Table 9 into the permissible values given above. Table 10 gives the results of this operation.

TABLE 10

DEAD LOAD AND LIVE LOAD SHELL CAPACITY

(All values in lbs. per sq. ft.)

Long Barrel Shell		Short Barrel Shell	
$T\Phi$	385*	$T\Phi$	192*
S	26.2+	S	25.6+
Tx	44.5*	Tx	86.6*
$M\Phi$	17.8+	$M\Phi$	100+

* These values are a function of wood properties only and may be taken as essentially working values for clear material

\+ These values are a function of initial prestress and are maximum values at 175 psi prestress taken from structural panel tests.

X. CONCLUSIONS FROM SHELL ANALYSIS

The results of the shell analysis are summarized in Table 10. Conclusions will be drawn from these results.

The limit of the long shell is the transverse moment capacity ($M\Phi$) which gives a maximum load capacity of 17.8 psf. The shear capacity is also low giving a value of 26.2 psf. These limitations appear to present a serious difficulty. The moment capacity would need to be more than doubled to make the shell usuable as a long barrel structure designed to carry 4 inches of wet concrete weighting about 50 psf.

Turning to the short barrel shell, the system holds better promise. The shell acts more nearly like an arch. Although the transverse compression

($T\phi$) is increased it presents no serious complication. Values of longitudinal tension (Tx) and transverse moment ($M\phi$) are much better. The only limitation on the capacity of the system which prevents it from carrying 4 inches of concrete (50 psf) is the shear. Shear capacity is only 25.6 psf maximum.

XI. RECOMMENDATIONS

From the results obtained from the shell designs made, it appears that short barrel shells are the most promising use for this system. The problems of moment and high longitudinal tension are minimized. By making a series of designs of shells of different chord and span lengths the most efficient shell configuration could be obtained.

Since shear presents the limitation to the system, further work must be done to improve the shear resistance. The following recommendations are made concerning the improvement of this property.

1. The tendon pressure should be increased. A tendon pressure of about 300 psi is recommended. This increase will also serve to give higher values of transverse moment strength.
2. Since the increase in shear which can be obtained by increasing tendon pressure may not be sufficient it is felt that the thickness of material used should be increased to 1-5/8 inches (2 inch nominal). This would result in about a 38 percent increase in shear strength due to the increase in area. The thickness increase would also have a beneficial effect on all values of structural strength.

In the letter of October 10, 1962 suggestions were made for further tests regarding strength properties and the control of prestress. Since the shell analysis indicates that thicker material is desirable it is proposed that the next series of tests be made with 1-5/8 inch stock. By extrapolating the results of the tests which have been made it appears that the use of 1-5/8 inch material and higher prestress should produce a structure capable of carrying the loads of thin concrete shells during forming.