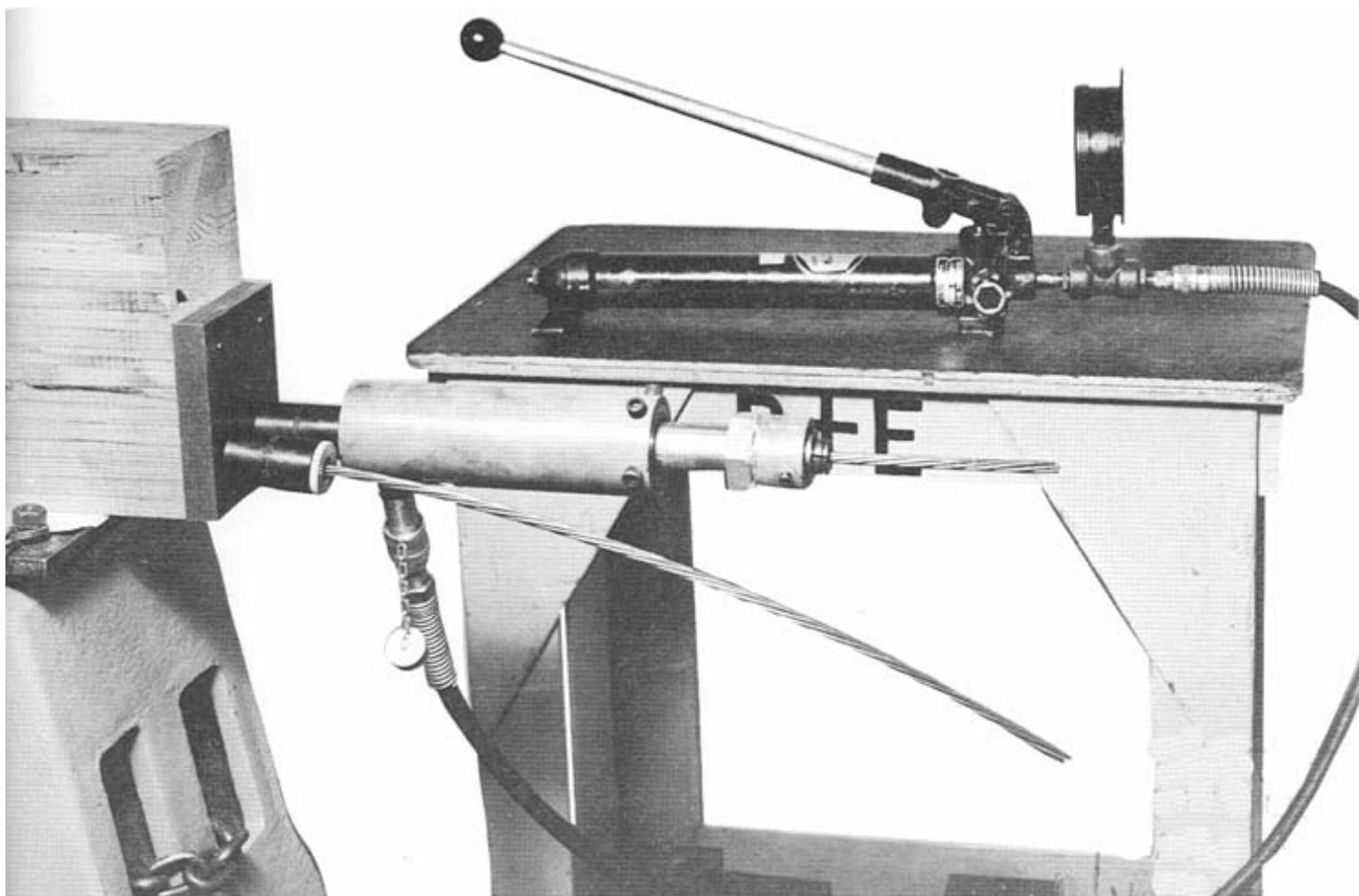


PRESTRESSED LAMINATED WOOD BEAMS



ABSTRACT

This paper presents a comparative evaluation of the flexural strength of 25 pairs of prestressed and matching unprestressed control beams, an evaluation of beams with clear straight-grained tension laminations, and a method of matching pairs of beams of structural size. Results show that an immediate 31 percent increase in strength and approximately 50 percent decrease in variability of strength can be obtained by prestressing, that a clear outer tension lamination probably has more influence upon flexural strength than is recognized by present design methods, and that structural size beams can be successfully matched.

PRESTRESSED LAMINATED

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INTRODUCTION

New techniques are continually being developed to further the utilization of wood. When trees suitable for the production of clear structural lumber became scarce, research was directed toward "engineered wood" that utilizes wood containing knots and other naturally occurring characteristics that influence its strength. In recent years, ideas have evolved of adding some other material to reinforce the wood and allow it to develop its full strength. One such idea is that of prestressing wood members subjected to bending. By prestressing, stresses are induced into the member to counteract all or a desired part of stresses anticipated from external loads. One method of producing these desired stresses is to place highly stressed, high-strength steel strands longitudinally in the member in the correct location.

In 1962, an initial study of prestressed wood members was made at the University of Wisconsin in cooperation with the U.S. Forest Products Laboratory (2).² The results of this investigation suggested that increased flexural strength and decreased variability of strength could be achieved by prestressing.

It was the purpose of the research study discussed herein to continue to study the effects of prestressing upon the flexural behavior of a laminated wood member. Specifically, this research investigates the changes in flexural strength and variability of strength that might be expected by prestressing, and the effective prestress level that will bring about these changes.

During the planning, the question was raised as to how the strength of wood beams, laminated from a low structural grade of material, would be changed if a

¹Maintained at Madison, Wis., in cooperation with the University of Wisconsin.

²Underlined numbers in parentheses refer to Literature Cited at the end of this report.

clear straight-grained board was used as the outer tension lamination. In partial answer to this question, a few beams with a clear, straight-grained outer tension lamination were also evaluated.

A few studies have been made to form a reinforced wood beam by adding metal reinforcement³ (3,4,5). The application of the results to structural-sized members has been attempted in only one of these studies (3). The prestressing of wood floor slabs has been attempted in New Zealand (6). The success of these attempts to reinforce wood in structural use is not known.

Theory of Prestressing

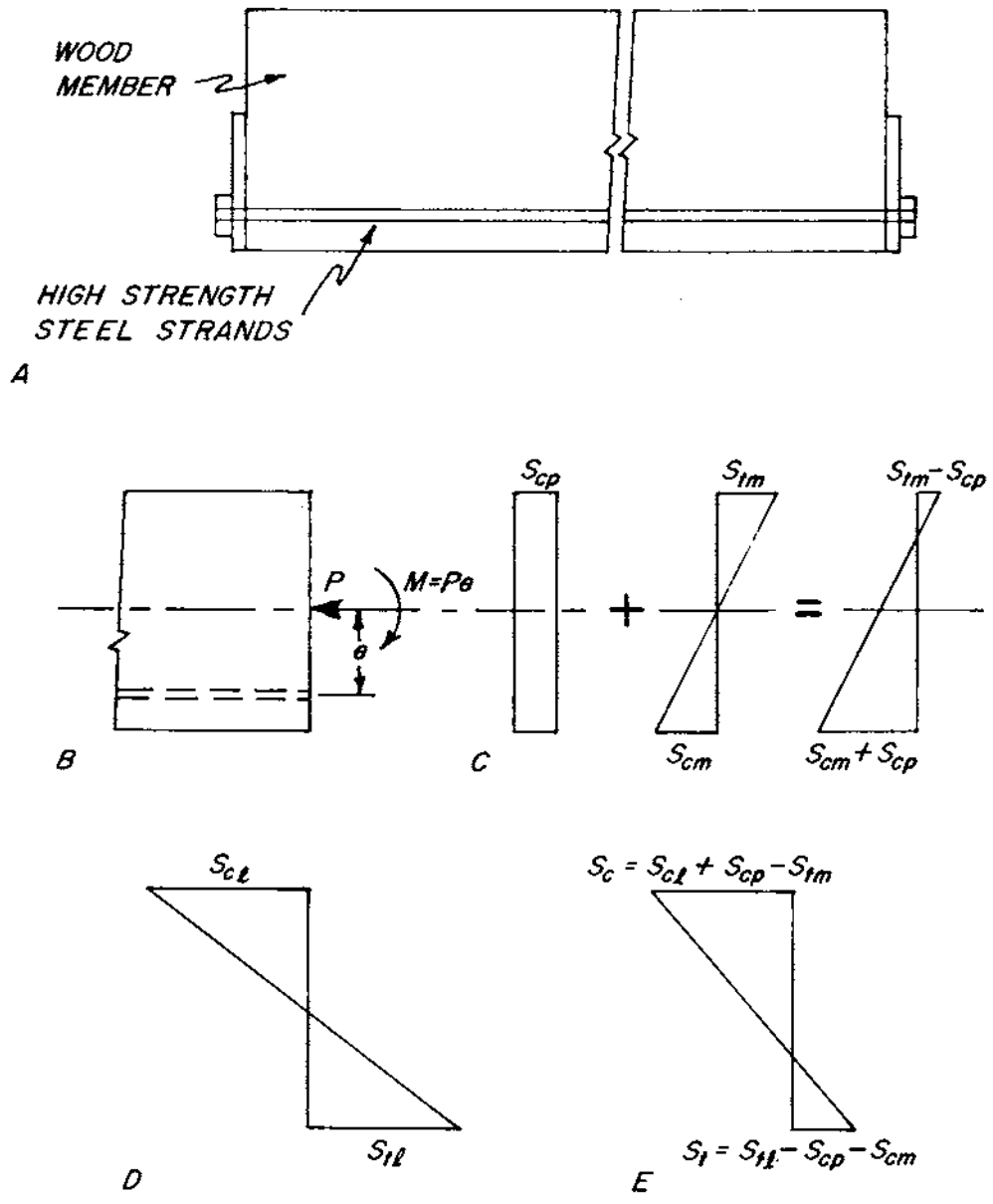
The theory of prestressing is not new. It has been successfully applied to concrete members for many years. The design principles of prestressing can be found in a number of good textbooks. Therefore, only a brief explanation will be given here.

A wood beam is prestressed by placing highly stressed, high-strength steel strands in the beam in regions where tensile stresses are anticipated. These steel strands induce compressive stresses that counteract part of the tensile stresses produced by external loads. By counteracting part of the tensile stresses with the stresses induced by prestressing, the flexural strength of a wood beam is less dependent upon tensile strength of the wood. Therefore, wood with lower tensile strength may be utilized in the fabrication of prestressed beams without decreasing the reliability of the flexural strength of the beams.

There are two general categories of prestressing, post-tensioning and pretensioning. The basic difference between the two is the method of transferring stress from the steel to the prestressed member. In post-tensioning prestressing, the stress in the steel is transferred to the member through end bearing plates. The steel may or may not be bonded to the member after transfer of stress. In pretension prestressing, the steel is highly stressed, then bonded to the member. The stress is transferred to the member through the bond. Post-tensioning prestressing with unbonded steel strands was used in this research on prestressed laminated wood beams.

Briefly, post-tension prestressing is depicted in figure 1A. The steel strands are inserted through the member, highly stressed, and anchored to the end bearing plates, thus inducing the desired stresses into the member. The action of the force in the steel can be divided into components of an axial force and a moment acting at the centroidal axis of the member (fig. 1B). This arrangement of force and moment will produce stresses as shown in figure 1C where S_{cp} is the compressive stress produced by the axial force and S_{tm} and S_{cm} are the tensile and compressive stresses produced by the prestressing moment. The magnitude of the stresses induced by the axial component is P/A and the magnitude of those induced by the moment component is P_e/Z , where P is the prestressing force, A is the cross-sectional area of the mem-

³Carr, William R. An Investigation of Steel Reinforced Wood Laminated Beams. A Master of Science Thesis, Montana State University. 1962.



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Figure 1.--Elastic stress distribution produced in a beam by a prestressing force and by a moment due to some external load.

ber being prestressed, Z is its section modulus, and e is the distance from the centroidal axis of the member to the prestressing force. When an external moment load is placed on the beam, stresses as shown in figure 1D will be produced. The final combination of internal induced stresses and external load stresses is shown in figure 1E. From these figures it is apparent that, with the correct amount and location of force, stresses can be induced into a member that will counteract tensile stresses produced by a design load.

The success of prestressing wood members depends upon whether or not the prestressing force can be retained. Some losses are encountered due to the seating of the end anchorage assemblies. These losses are fairly well understood from the results of research on prestressed concrete. The largest loss occurs from the elastic shortening of the wood during transfer of stress from the steel strands to the wood member and the plastic shortening of the wood and steel after transfer. The elastic loss is eliminated by post-tensioning. The creep loss in the steel is very small and its effect can be eliminated through correct design of the prestressed member. Creep in wood is known to exist, but unfortunately not enough data are available to precisely define its effect. Limited data at the Laboratory suggest that the loss due to creep of wood may not be excessive. More research is needed before the effects of creep can be correctly predicted.

The effect of creep was nearly eliminated in the present study by prestressing the beams immediately before testing. The time from prestress to fail-

ure was about 30 minutes.

Theoretical Effects of Prestressing

Prestressing a wood beam made from a low structural grade of lumber should increase its flexural strength and decrease its variability of flexural strength. This hypothesis is partially validated by the limited data from the initial study on prestressed wood beams (2). It can be further substantiated by an explanation of the stress distribution on a transverse cross section of a wood beam at failure and the mechanical properties of the wood that govern the distribution of stress.

To maintain equilibrium, an external bending moment on a beam is resisted by tensile and compressive stresses within the beam. The summations of the tensile and compressive stresses can be considered as tensile and compressive forces, which must be equal in magnitude and must act together to form a couple, thus resisting the external moment.

An ideal beam would have balanced mechanical properties so that at failure the ultimate tensile and compressive forces that make up the internal couple would be reached simultaneously. For this to occur, the stress distribution within the beam must be such that all the existing compressive and tensile strength of the beam is used. Such a stress distribution is approached in a clear straight-grained wood beam as it is loaded to failure.

The simplified stress distribution developed within a clear straight-grained wood beam as failure is approached, due to an external moment loading, has been

found to be as shown in figure 2A, where S_{cu} is the compressive strength and S_{tu} is the tensile strength (1). Failures that accompany such a stress distribution are a plastic-type compression failure and an elastic-type tension failure. As the beam is loaded, the plastic-type compression failure occurs first, followed by the final elastic-type tension failure. For such a stress distribution and sequence of failures to occur, the tensile strength must be greater than the compressive strength, a condition that exists in clear straight-grained wood. With these types and sequence of failures, stresses equaling the tensile and compressive strengths are developed, thus utilizing all of the flexural strength of the beam. Prestressing a beam that would fail in the compression-tension sequence would produce little added flexural strength if the prestressing was designed to increase only the tension component of the internal couple, as was done for this research study. The tensile strength of such a wood beam is sufficient to produce a tensile force that will balance the maximum compressive force that can be developed within the beam. Therefore, the added component of tensile force produced by prestressing the beam would be wasted. As implied in the original hypothesis, prestressing is probably not beneficial on clear straight-grained beams but probably is beneficial on lower quality beams. The following discussion may explain why.

As the quality of wood decreases due to strength-reducing features, both the tensile and compressive strengths decrease. The relative reduction in strength is not fully understood, but it is generally agreed that, for some combina-

tion of strength-reducing features, the compressive strength becomes greater than the tensile strength. When this occurs, the compression-tension sequence of failure, as discussed for a clear straight-grained wood beam, will not develop. As a beam is loaded in flexure, the stress distributions in the beam are triangular and the stress on the outer tension face is equal to the stress on the outer compression face (fig. 2B). The stresses in the outer faces become unequal, and the plastic-type compression failure starts only after the stresses equal or exceed the compressive strength. Therefore, if the tensile strength is less than the compressive strength, the final tension failure will occur before the compressive strength is reached; thus part of the existing compressive force in the beam will not be used. By prestressing such a beam, stresses are induced in the tension side that should reinforce the tension component of the internal stress couple. This forces the plastic-type compression failure and allows full utilization of the existing compressive force component. This should produce an increase in flexural strength when compared to an unprestressed beam with equal mechanical properties.

It is difficult to predict the exact compressive and tensile strengths of wood with strength-reducing features. This makes it difficult to determine the prestress level that is needed to produce the desired effects in a prestressed wood beam. As previously discussed, the prestressing in the present research study was designed to increase only the internal tensile force component, thus insuring complete utilization of the flexural

strength of a beam by forcing the plastic-type compression failure to occur before the final tension failure. In this report the effective prestress level will be defined as the prestress level that, when induced in the tension side of the beam will cause a wood beam made from a low structural grade of material to exhibit

the compression-tension sequence of failure as is developed in a clear straight-grained wood beam. A stress level exceeding the effective prestress level would reinforce the tensile stress component of the internal couple an excessive amount and would be partially wasted.

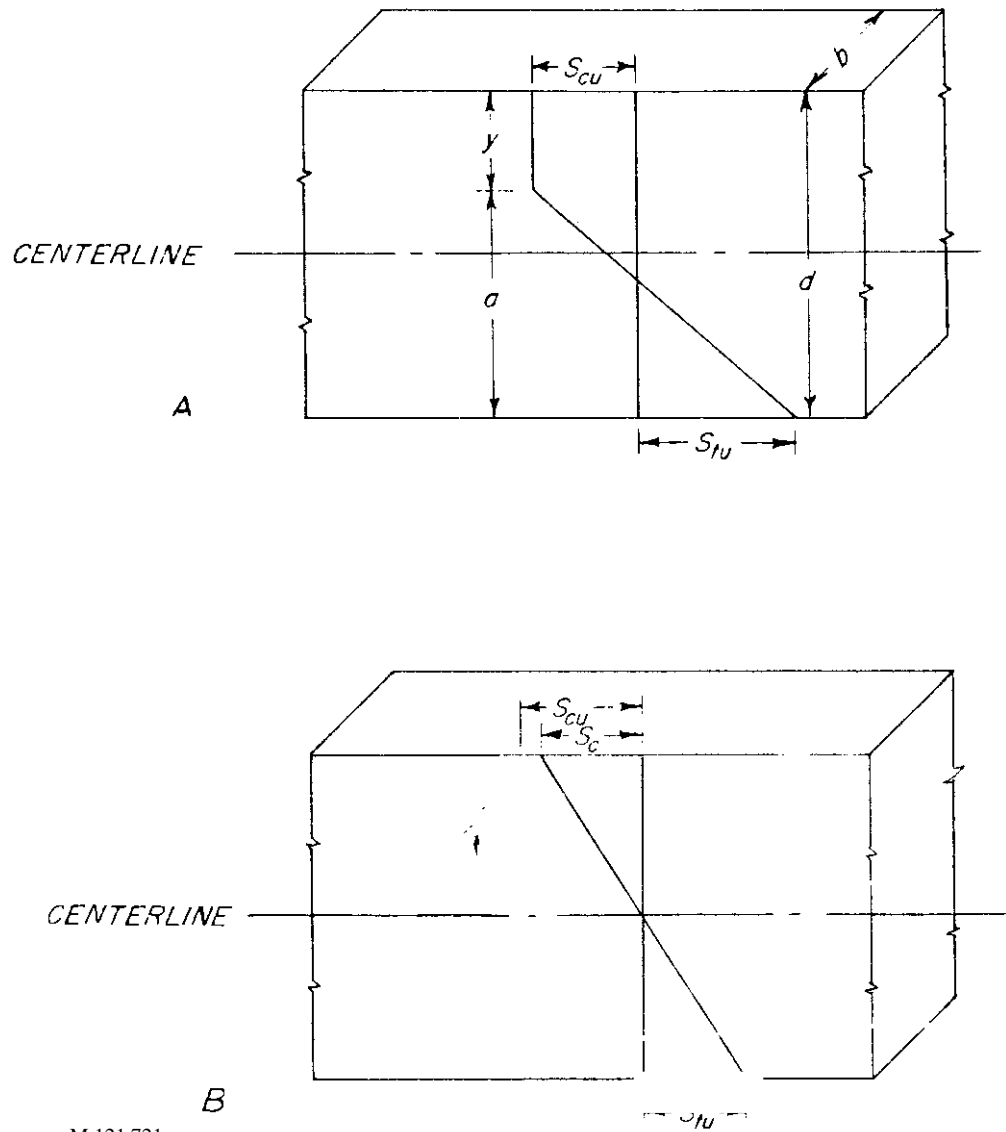


Figure 2.--Simplified longitudinal stress distributions in a wood beam when tensile strength is: A, greater than compressive strength; B, equal or less than compressive strength.

The increase in strength that is believed possible by prestressing should be accompanied by a decrease in variability of flexural strength. Wood, even when free from strength-reducing features, has a considerable variability in flexural strength properties (9). Data from 175 beams having a cross section of 5-1/2 by 12 inches indicate that, as the quality of wood beams decreases, the variability of their flexural strength increases (8). Some of these beams had strengths as low as 40 percent of clear controls. The increase in variability of strength apparently accompanies the relative reduction in tensile and compressive strengths and the type of failure of lower quality beams. Although not documented, it is believed that the tensile strength becomes quite variable when the strength-reducing characteristics

increase in number and severity sufficiently to cause the tensile strength to become less than the compressive strength. The variability of the compressive strength also increases, but to a much lesser extent. Assuming these ideas are true, the reason why prestressing will decrease variability of flexural strength is easily understood. As previously discussed, the sequence of failure of a wood beam is governed by the relative magnitude of the compressive and tensile strengths. If the flexural strength of a wood beam that is normally dependent upon its tensile strength can be forced, by prestressing, to be dependent upon its compressive strength and if the variability in compressive strength is less than that of the tensile strength, then the variability of the flexural strength should be decreased.

SCOPE OF PAPER

This paper gives the results of flexure tests on 68 laminated Douglas-fir beams, 5 by 10-11/16 inches by 18 feet. Each beam contained nineteen 9/16-inch-thick laminations, except where otherwise indicated. The beams were fabricated in matched pairs, and the pairs were divided into groups as follows:

Group 1.--Twenty-five pairs. Each pair consisted of a prestressed beam and a matched unprestressed control beam.

Group 2.--One pair. It consisted of a control beam and a matched beam to which a clear straight-grained 1-1/2-inch-thick lamination was added to the tension face.

Group 3.--Two pairs. Each pair con-

sisted of a control beam and a matched beam to which a clear straight-grained 9/16-inch-thick lamination was added to the tension face.

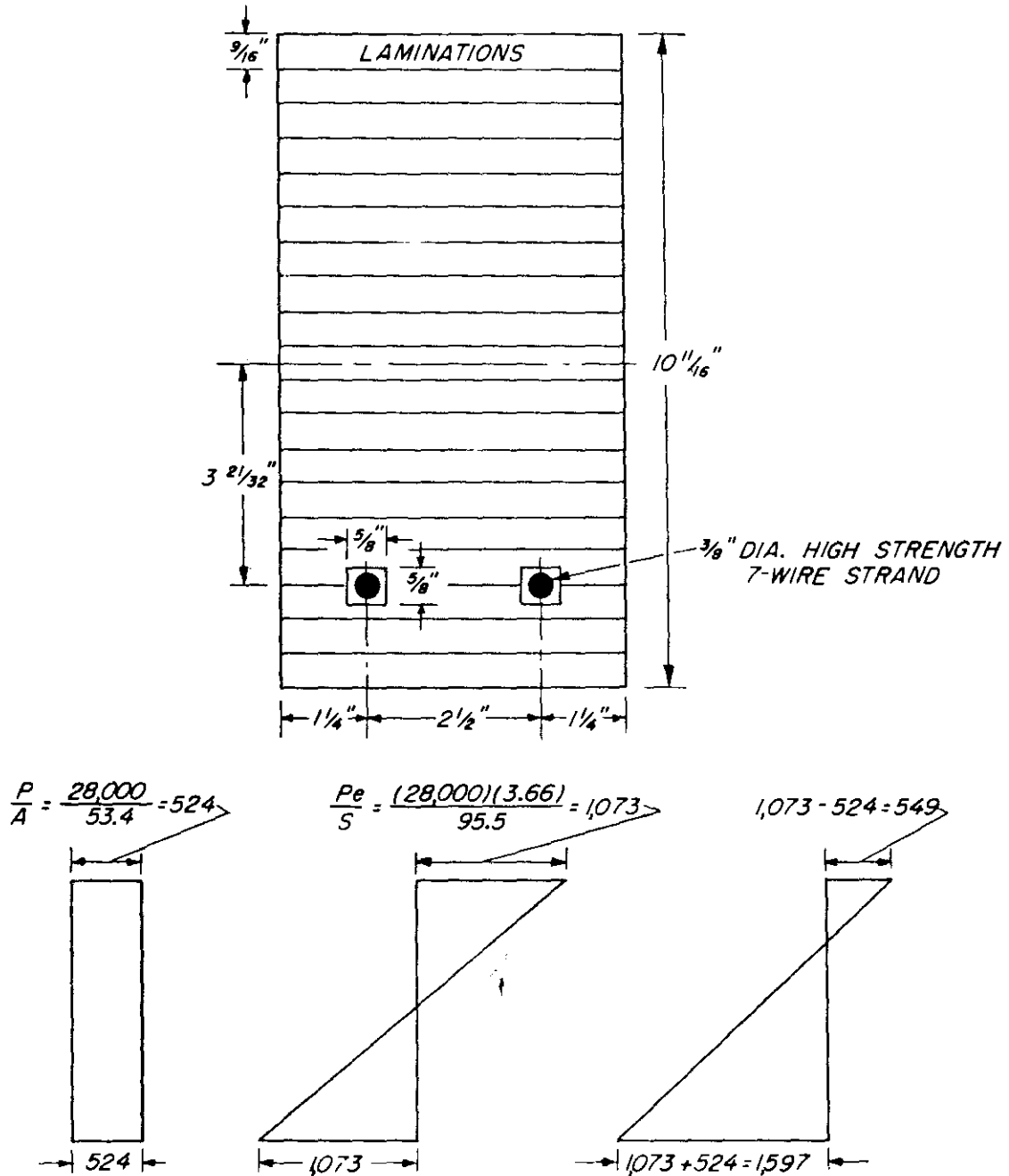
Group 4.- Three pairs. Each pair consisted of matched beams with each beam having a clear straight-grained outer tension lamination.

Group 5.--Three pairs. Each pair consisted of matched beams with no modifications.

The 25 prestressed pairs (Group 1) were further divided into 5 groups which represented prestressing levels of 1,600, 1,800, 2,000, 2,400, and 2,800 p.s.i. (pounds per square inch) of induced compressive stress in the outer tension face.

The complete design data of the pre-stressed beams are given in table 1. A detailed design for the 1,600 p.s.i. stress level is shown in figure 3. The strands were located 1-1/4 inches from each

edge of the beam for all designs where two strands were used at the same vertical location. Where only one strand was used, it was placed at the midwidth.



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Figure 3.--Detailed design of beams prestressed at a 1,600 p.s.i. stress level

Table 1.--Design data of prestressed beams

Group	Prestress stresses ¹		Steel strands			Force per strand ²	Maximum end bearing stress ³
	Compression in bottom	Tension in top	Diameter	Number	Location ⁴		
	<u>P.s.i.</u>	<u>P.s.i.</u>	<u>In.</u>			<u>Lb.</u>	<u>P.s.i.</u>
1-1,600	1,600	550	3/8	2	3	14,000	2,200
1-1,800	1,800	480	7/16	2	4	17,600	2,080
1-2,000	2,000	540	3/8	2	3	13,000	2,310
				1	6	13,000	
1-2,400	2,400	420	7/16	2	4	17,700	2,500
				1	7	17,700	
1-2,800	2,800	140	7/16	2	4	17,800	2,810
				2	8	17,800	

¹ Bottom and top refer to tension and compression faces, respectively, of beam.

² To the nearest 100 pounds.

³ Assuming triangular distribution of stress under end bearing plates.

⁴ Given in numbers of glue lines from bottom of beam.

MATERIAL AND FABRICATION

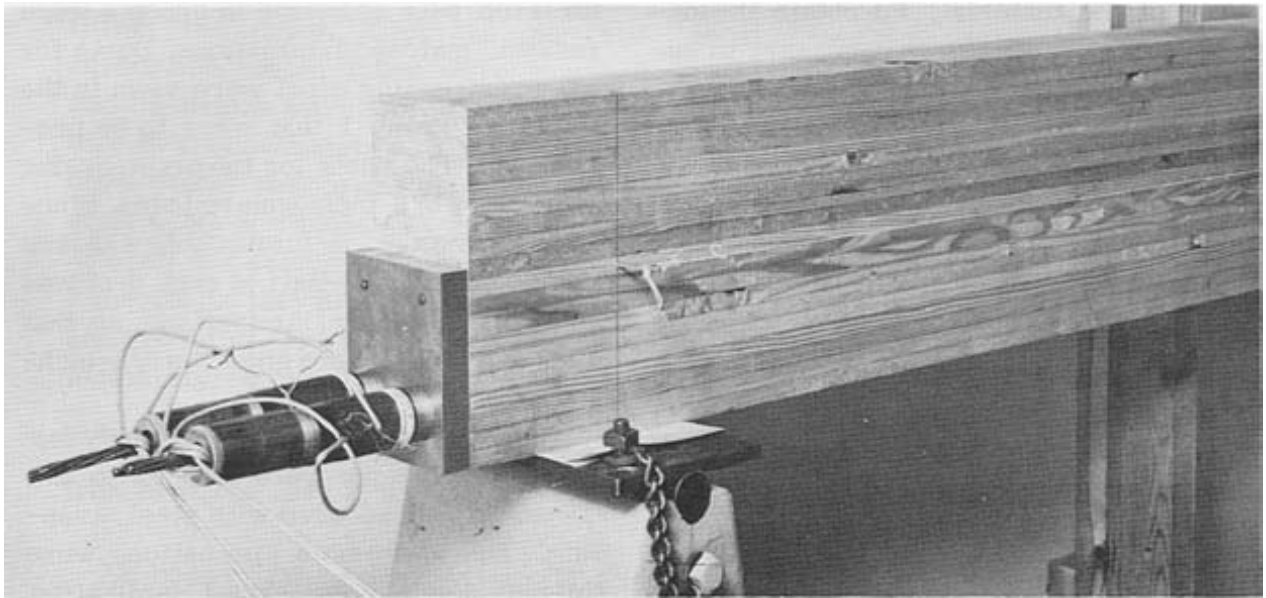
L-3 Grade Douglas-Fir

The laminations for the beams were selected from 700 pieces of 2- by 6-inch by 18-foot L-3 grade coast-region Douglas-fir lumber, as explained in paragraph 154, West Coast Lumbermen's Standard Grading Rules (8). It was purchased from a commercial laminator in Oregon. The lumber had been kiln dried to an average moisture content of

11 percent, with a maximum moisture content of 15 percent. It was stress relieved during kiln drying. No attempt was made to control the moisture content of the lumber after it was received at the Laboratory.

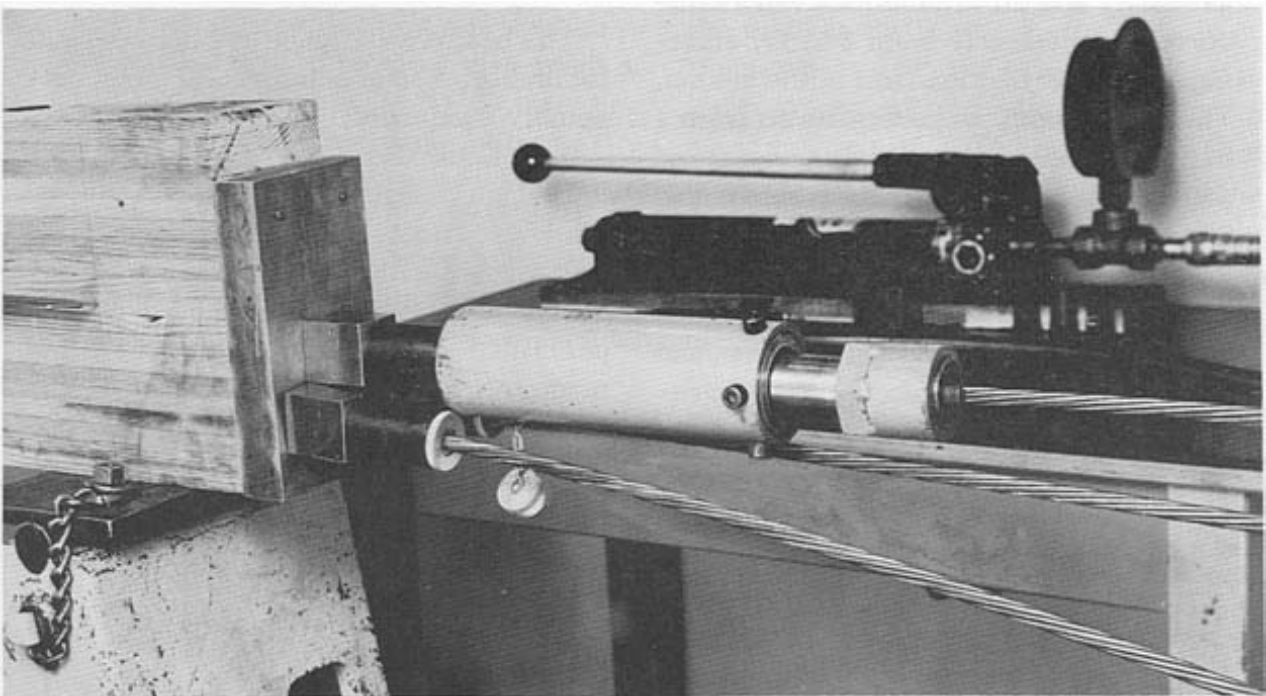
Clear Laminations

The clear laminations were C and Better coast-region Douglas-fir that were purchased from two local suppliers.



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Figure 4.--A view of a beam reinforced with steel cable, showing bearing plates, electrical strain loadcells, and end anchorages.



M 123 224

Figure 5.--Hollow-core 15-ton hydraulic jack and hand pumped to apply the initial prestressing force.

The origin of the material is not known. Average moisture content was 12 percent as checked with an electric moisture meter.

Prestressing Steel and End Anchorages

The prestressing steel was super-tension, stress-relieved seven-wire strands. Properties of the steel are given in table 2. This is the same steel that is used to prestress concrete. The strands were furnished to the Laboratory by the American Steel and Wire Company, Cleveland, Ohio.

The strands were anchored against end bearing plates with strand chucks that are commercially available. These chucks (figs. 4 and 5) consist of a cylinder with a conical interior into which the strands were anchored with tapered jaws. The end bearing plates were flat rolled steel.

Matching of the Beams

To obtain comparable results, the beams were fabricated in matched pairs. The matching was accomplished by re-sawing 2- by 6-inch lumber, producing laminations with finished thicknesses of 9/16-inch. The resawn halves of each 2- by 6-inch piece were placed in corresponding laminations of the matched pairs of beams. This, it was felt, would produce two beams with comparable strength properties. Six pairs of the beams, groups 4 and 5, were designed to check the suitability of this method of matching beam pairs.

Selection and Preparation of Laminations

To eliminate any human judgment in the layup of the beams, all laminations were selected at random after one sort-

Table 2.--Properties of seven-wire super-tension stress-relieved strands¹

Nominal diameter	Nominal steel area	Minimum breaking strength	Minimum 2 percent offset yield strength	Minimum elongation in 24 inches	Typical modulus of elasticity
<u>In.</u>	<u>Sq. in.</u>	<u>Lbs.</u>	<u>Lbs.</u>	<u>Pct.</u>	<u>P. s. i.</u>
3/8	0.080	20,000	17,000	3.5	27,500,000
7/16	.109	27,000	23,000	3.5	27,500,000

¹Data furnished by the American Steel and Wire Co., Cleveland, Ohio.

ing of the material. To obtain the desired matching, it was necessary to use boards with uniform properties throughout their thickness, so that after resawing the two halves would be as near alike as possible. The 700 pieces were sorted into 2 groups, 1 with uniform characteristics and 1 with nonuniform characteristics. From the uniform grouping, pieces were selected at random for the five outer tension laminations and the four outer compression laminations. From the nonuniform grouping, pieces were chosen at random for the remaining 10 middle laminations.

After selection of laminations, they were resawn and skip-dressed to a constant thickness of $23/32$ inch and were sawn to a constant width of $5-1/2$ inches. The beams were then laid up in dry assembly and all laminations were numbered consecutively from 1 to 19, starting from the tension face of the beams. The beam pairs were numbered from 1 to 34. One beam of each pair was marked with a C following the pair number, and the matched beam was marked with a P. Following this, the beams were randomly selected for the five groups. The beams marked with a C were control beams, and the beams marked with a P were prestressed beams of group 1 and the modified beams of groups 2 and 3. In groups 4 and 5, where the beams in each pair were alike, the P had no significant meaning.

After grouping of the beams, a clear straight-grained lamination was added to the outer tension face of one beam in each pair of groups 2 and 3 and lamination 1 was replaced by a clear straight-grained lamination on both beams of each pair of group 4. The clear laminations

for each pair of beams in group 4 were resawn, matched laminations. Ducts for the prestressing steel were sawn in the beams of group 1 that were to be prestressed. The ducts for the prestressing steel were $5/8$ -inch-square holes. Since the thickness of the laminations was less than $5/8$ -inch, one-half of each duct was sawn in adjoining laminations so that when the beams were glued the $5/8$ -inch-square duct was formed.

When the laminations were ready for gluing, the beams were laid up in dry assembly and secured with steel strapping. The assembled laminations were shipped to a commercial laminating plant for final fabrication.

Gluing of the Beams

The beams were glued by a commercial laminator, using commercial plant assembly equipment.

The laminations were surfaced to a $9/16$ -inch thickness just prior to gluing. They were double spread on a mechanical spreader with phenol-resorcinol adhesive at a minimum rate of 60 pounds per 1,000 square feet of glue line. Immediately after the glue was spread, the laminations for each beam were laid up in closed assembly until eight complete beams were ready for clamping, a period of approximately 25 minutes. A gluing pressure of 140 pounds per square inch was applied and the beams were cured for a minimum of 5 hours at a room temperature of approximately 72° F. During the gluing, $1/2$ -inch-diameter copper tubes coated with grease were placed in the ducts for the prestressing steel to prevent squeezed-out glue from

filling the ducts. After the glue had cured the copper tubes were removed and the beams were stored in a temperature of approximately 72° F. for several days before they were planed to a constant width of 5 inches.

When the beams were returned to the Laboratory, they were stored out of doors and covered with tarpaulins. Subzero temperatures were encountered during

most of this time. Before the beams were tested, they were placed in a controlled atmosphere of 75° F. and 64 percent relative humidity for a minimum of 48 hours. From previous studies it was found that a constant temperature throughout the beam would be obtained after this period of time following removal from cold storage.

METHOD OF EVALUATION

All beams were evaluated on a 17-foot span under two-point loading in a 200,000-pound-capacity mechanical testing machine (fig. 6). Load points were spaced 4 feet apart. Roller nests were used under one loading head to insure that vertical loading was applied to the beam. Rate of vertical movement of the loading head was 0.279 inch per minute. Loading was momentarily stopped at each 1,000-pound increment of load to allow time for data readings, but after 8,000 pounds were on the beam, the loading was continuous to failure. After each beam was failed, a 1-inch-long cross-sectional slice was cut out at approximately 24 inches from one end. This slice was used to determine the average moisture content and specific gravity of each beam.

Data taken during a test were midspan deflections over a 36-inch span between load points, midspan deflections between supports, strains on the top and bottom faces of the beams, and the change in stress of prestressing strands. Deflections between load point were recorded to 5,000-pound load, but all other data readings were taken to failure. Deflec-

tions between load points were measured to the nearest 0.001 inch with a dial micrometer attached to a light yoke and to a nail located at the midheight and midlength of the beam. The light yoke was supported on nails 18 inches each side of the midlength of the beam (fig. 6). Deflections between supports were measured to the nearest 0.01 inch with a taut wire and scale. The taut wire was stretched between nails located at middepth of the beam above the supports, and the scale was located at midlength of the beam. A telescope was used to read the location of the wire on the scale. Longitudinal strain was measured with electrical resistance wire strain gages located on the top and bottom faces of the beams.

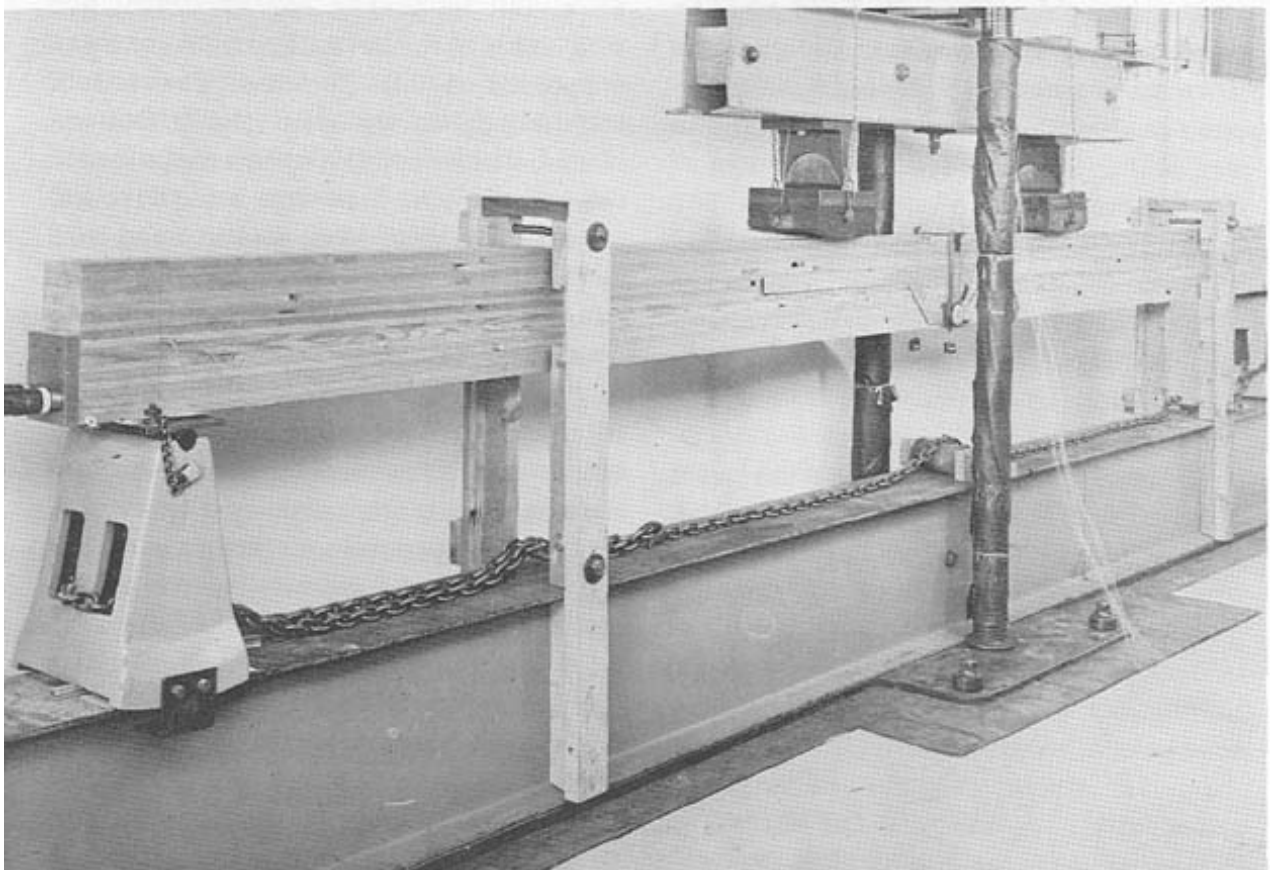
The stress in the steel strands was measured with electrical strain load cells (fig. 4). These cells consisted of a thin cylinder to which eight electrical resistance wire strain gages (4 longitudinal and 4 transverse) were attached. The corresponding gages on opposite sides of the cylinder were wired in series; then the four sets of gages were wired into a Wheatstone bridge type of circuit. This gives a load cell with a

sensitivity of about 2-1/2 times the axial strain caused by load applied to the cylinder. These load cells were calibrated in a testing machine.

The beams were prestressed immediately before they were tested. The steel strands were stressed one at a time with a hollow-core 15-ton hydraulic jack and hand pump. The strand chucks used to anchor the steel are designed so that the steel can be stressed through the chuck; upon release of the stress from the jack, the chuck automatically secures it. During the transfer of stress from the jack to the chuck, from 10 to 25 percent of the stress is lost due

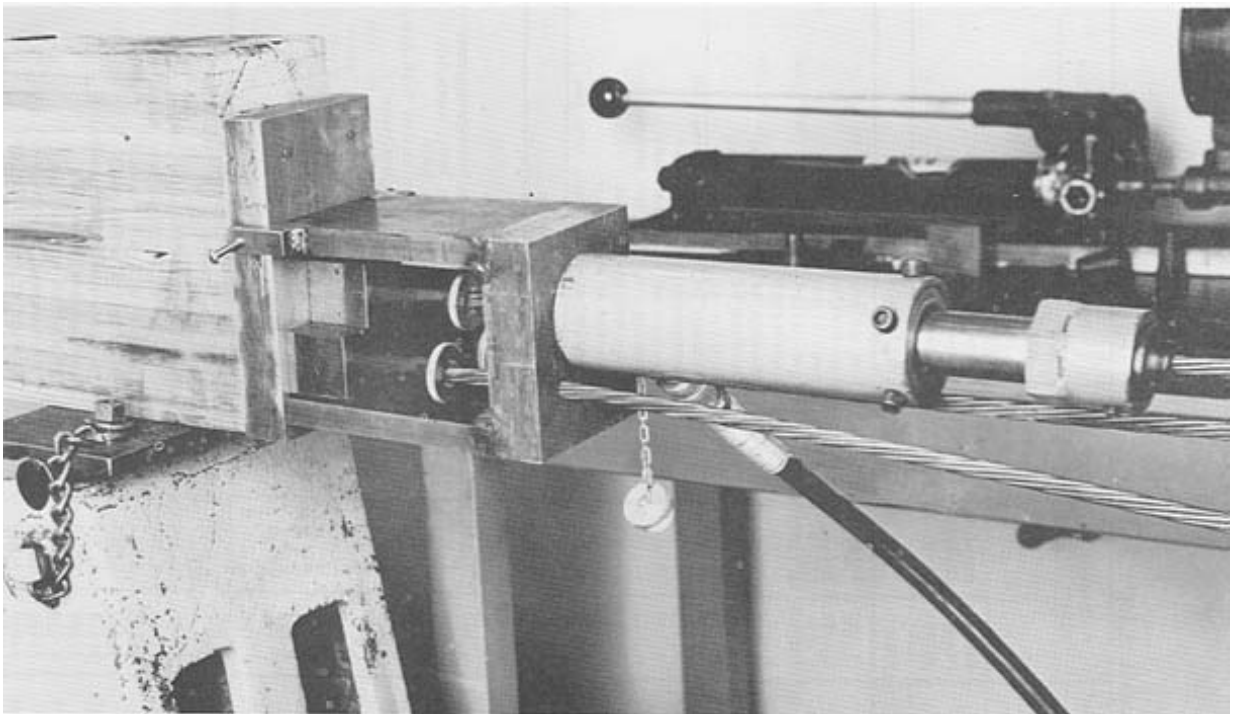
to the seating action of the jaws. Therefore, two steps of prestressing were required to obtain the exact design stress.

About 90 percent of the stress was applied (fig. 5) with the jack anchored against the chuck. When the stress was transferred to the chuck, the jaws in the chuck were securely set onto the strand. An extension "chair" was then placed over the chuck assembly (fig. 7) and the steel strands were stressed again with the jack anchored against the "chair." Shims were used between the strand chuck and the end bearing plate to adjust the stress to the exact design value. Load cells which were located at the other end



M 120032

Figure 6.--Arrangement of the apparatus for evaluating beams under two-point loading.



M 123 225

Figure 7.- Equipment used to adjust prestressing force to the design value. Shims were used between the strand chuck and the end hearing plate to adjust the stress to the exact design value.

of the beam were used to measure the initial prestressing force.

The extension “chair” was used to release the stress in the steel strands after the beams were failed. The prestressing jack was anchored against the “chair” and the strands were again stressed, thus

relieving the stress from the strand chucks. The chucks were then disassembled and removed and the stress in the strands was released by the jack. Special 2-inch-long shims were used between the chuck and end bearingplate to allow room for removal of the chuck.

PRESENTATION AND DISCUSSION OF RESULTS

Results of Flexure Tests

The results of the flexure tests are given in table 3. The proportional limit and modulus of rupture values given in columns 8, 10, and 11 were calculated from the standard flexure formula,

$$f = \frac{Mc}{I}$$

where f is the calculated stress, M is the

applied moment, c is the distance from the neutral axis to the outer face of the beam, and I is the moment of inertia of the cross section. For the prestressed beams, the gross section properties were used. To be theoretically correct, a correction in section properties should be made to account for the loss due to the ducts for the prestressing steel. This is a minor correction, which can be neg-

Table 3.--Results of flexure tests on 34 matched pairs of laminated beams

Group	Beam	Height ¹ in.	Moisture content pct.	Specific gravity ²	Maximum load lbs.	Load after failure lbs.	Proportional limit		Modulus of rupture		Ratio ³	Modulus of elasticity		Number of laminations of compression failures	Type and sequence of failure	
							Observed	Corrected	Point of failure ³	Between load points ⁴		Between load points ⁵	Between supports ⁷			p.s.i.
1-1600	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
	CONTROL BEAMS AND MATCHED BEAMS PRESTRESSED TO 1,600 p.s.i.															
	5-C	10.60	10.6	0.451	9,840	7,800	3,330		4,100	4,100	1.75	1,700	1,640	0	Tension at 1-1/8 in. knot	
	3-P	10.63	10.5	.452	17,130	13,460	5,800	5,260	7,180	7,180	1.75	1,860	1,680	0	Tension at 1-1/8 in. knot	
	6-C	10.64	10.4	.468	16,200	2,650	4,960	5,230	4,640	6,700	1.43	1,780	1,670	0	Tension at 2-in. knot	
	6-P	10.67	10.4	.468	19,700	10,430	5,760	5,230	8,100	4,390	1.34	1,710	1,700	3	Tension at 2-in. knot, comp., ten.	
	16-C	10.64	10.4	.480	11,810	4,190	4,140	4,810	4,640	6,650	1.33	1,750	1,720	0	Tension at 1-7/8 in. knot	
	16-P	10.68	10.5	.469	17,110	11,440	5,340	5,220	6,650	7,020	1.50	1,950	1,660	0	Tension at 1-7/8 in. knot	
	21-C	10.61	10.4	.468	13,130	5,870	4,400	5,240	5,440	7,300	1.31	1,920	1,760	0	Tension	
	21-P	10.67	10.6	.466	17,770	13,660	5,760	5,220	7,300	5,960	1.33	1,860	1,770	1	Compression, tension	
	23-C	10.65	10.5	.494	14,290	2,330	5,360	5,680	7,300	5,960	1.33	1,960	1,780	0	Tension at 2-in. knot	
23-P	10.63	10.5	.490	18,910	13,620	6,450	5,680	7,400	7,840	1.50	2,000	1,870	3	Tension at 2-in. knot, comp., ten.		
Average of controls		Average of prestress														
1-1800	CONTROL BEAMS AND MATCHED BEAMS PRESTRESSED TO 1,800 p.s.i.															
	8-C	10.66	10.5	0.474	18,100	1,400	6,170	6,450	6,020	7,440	1.49	1,990	1,820	2	Tension at 1-1/2 in. knot, comp., ten.	
	8-P	10.73	10.6	.474	22,000	12,000	6,910	6,450	8,960	8,960	1.45	2,050	1,920	8	Tension at 1-1/2 in. knot, comp., ten.	
	10-C	10.64	10.3	.460	13,420	7,080	5,360	5,740	5,560	6,210	1.13	1,850	1,680	0	Tension at 1-3/8 in. knot	
	10-P	10.61	9.7	.476	19,420	15,180	6,210	5,740	8,050	7,240	1.26	2,120	1,840	2	Tension 3/4 in. comp., ten. at 1-3/8 in. knot	
	11-C	10.68	10.7	.481	17,650	1,740	6,160	5,700	8,160	6,880	1.26	2,080	1,910	4	Compression, tension	
	11-P	10.68	10.7	.465	19,880	14,000	6,160	5,700	8,160	6,880	1.26	2,080	1,910	10	Compression, tension	
	13-C	10.69	10.7	.465	16,720	1,770	5,760	5,390	6,880	8,690	1.29	1,950	1,810	0	Tension at 1-3/8 in. knot	
	13-P	10.66	10.5	.471	21,120	10,370	5,760	5,390	8,690	6,140	1.29	2,050	1,920	4	Compression, tension	
	33-C	10.66	10.7	.478	14,920	9,850	4,530	4,490	6,140	7,900	1.31	1,920	1,860	0	Tension at 1-3/4 in. knot	
	33-P	10.64	10.7	.471	19,080	12,590	5,680	5,490	6,370	8,150	1.31	2,060	1,930	3	Tension at 1-3/4 in. knot, comp., ten.	
Average of controls		Average of prestress														
1-2000	CONTROL BEAMS AND MATCHED BEAMS PRESTRESSED TO 2,000 p.s.i.															
	13-C	10.66	10.6	0.473	16,620	1,800	5,760	6,120	6,320	6,840	1.16	1,950	1,820	0	Tension at 1-1/8 in. knot	
	18-P	10.62	10.6	.489	19,160	15,140	6,640	6,120	7,360	7,960	1.16	1,980	1,850	5	Tension at 1-1/8 in. knot, comp., ten.	
	24-C	10.64	10.6	.470	14,070	2,000	4,960	5,290	5,830	6,950	1.32	2,420	1,810	0	Tension at 1-3/8 in. knot	
	24-P	10.62	10.6	.475	16,750	14,370	5,810	5,290	6,950	8,760	1.32	1,950	1,840	0	Tension 1/5 cross grain	
	25-C	10.64	10.2	.499	15,970	5,430	6,210	5,710	6,610	8,760	1.25	2,250	1,990	1	Tension, compression, tension	
	25-P	10.62	9.9	.493	21,120	12,040	6,220	5,710	8,760	7,260	1.25	2,180	1,960	5	Compression, tension	
	26-C	10.62	10.5	.482	17,480	1,360	5,820	5,240	7,260	9,100	1.66	2,070	1,880	0	Tension at 1/2-in. knot	
	26-P	10.66	10.6	.490	22,110	12,300	5,760	5,240	9,100	4,170	1.66	2,050	1,850	5	Compression, tension	
	32-C	10.66	10.5	.468	11,000	7,200	3,700	5,700	4,170	4,530	1.66	2,080	1,850	0	Tension at 2-3/8 in. knot	
	32-P	10.64	10.7	.462	18,190	15,170	6,210	5,700	6,920	7,500	1.29	1,970	1,930	4	Tension at 2-3/8 in. knot, comp., ten.	
Average of controls		Average of prestress														

CONTROL BEAMS AND MATCHED BEAMS PRESTRESSED TO 2,400 p.s.i.													
1-2400	4-C	10.62	10.6	0.430	17,710	6,150	6,230	7,360	1.21	2,140	1,900	2	Tension
	4-P	10.65	10.5	.427	21,550	8,500	6,600	8,890		2,090	1,920	0	Compression, shear
	15-C	10.64	10.9	.477	19,000	1,700	5,800	7,860	1.16	2,000	1,950	7	Tension at 5/8-in. knot
	15-P	10.64	11.1	.470	22,100	12,550	6,200	5,800		2,000	1,990	0	Compression, tension at 5/8-in. knot
	28-C	10.65	10.5	.474	13,710	3,800	4,950	5,660	1.24	1,820	1,720	8	Tension at 2-3/8 in. knot
	28-P	10.65	10.4	.486	17,000	12,530	6,200	5,800		1,740	1,690	3	Tension at 2-3/8 in. knot, comp., ten.
	29-C	10.67	10.5	.463	14,500	3,100	4,520	5,740	1.23	1,710	1,590	0	Tension
	29-P	10.65	10.7	.467	17,100	14,200	4,950	7,060		1,650	1,580	6	Compression, tension
	31-C	10.63	10.6	.477	13,790	6,160	4,140	5,720	1.37	1,760	1,780	0	Tension at 2-1/8 in. knot
	31-P	10.66	10.8	.483	19,080	14,860	4,940	7,850		1,920	1,760	3	Compression, tension at 2-1/8 in. knot
	Average of controls								1.23				
	Average of prestress												
CONTROL BEAMS AND MATCHED BEAMS PRESTRESSED TO 2,800 p.s.i.													
1-2800	1-C	10.63	10.7	0.447	13,000	8,800	4,970	5,390	1.38	1,940	1,870	0	Tension at 1-3/4 in. knot
	3-P	10.64	10.6	.429	18,000	5,790	7,440	7,440		1,940	1,870	7	Compression, tension at 1-3/4 in. knot
	9-C	10.61	10.5	.461	15,980	1,330	5,410	6,640	2,040	1,780	2	Compression, tension
	9-P	10.65	10.4	.466	19,620	1,035	6,190	8,100		1,870	1,760	9	Compression, tension, shear
	11-C	10.63	10.5	.458	14,750	4,930	5,380	6,120	1.26	1,810	1,810	0	Tension 1:5 cross grain
	11-P	10.61	10.3	.457	18,630	16,410	6,230	6,090		1,950	1,800	7	Compression, tension 1:6 cross grain
	14-C	10.63	10.8	.497	20,500	1,000	5,800	8,500	1.03	2,070	1,920	3	Tension at 1-7/8 in. knot, comp., shear
	14-P	10.65	11.1	.484	21,200	11,900	6,180	8,740		1,960	1,890	9	Compression, tension at 1-7/8 in. knot
	22-C	10.64	10.9	.461	12,600	4,610	5,220	6,050	1.44	1,940	1,800	0	Tension at 1-1/2 in. knot
	22-P	10.64	11.0	.462	18,180	15,500	5,860	7,520		1,910	1,770	7	Tension 3d. lam., comp., ten.
	Average of controls								1.24				
	Average of prestress												
CONTROL BEAM AND A BEAM MODIFIED WITH ADDED 1-1/2-INCH-THICK CLEAR STRAIGHT GRAINED OUTER TENSION LAMINATION													
2	34-C	10.64	9.8	0.470	15,750	8,480	4,960	6,520	1.317	2,100	1,940	0	Tension
	34-P	12.14	9.5	.459	27,000	2,160	4,450	8,590		2,130	1,980	9	Compression, tension
CONTROL BEAMS AND BEAMS MODIFIED WITH ADDED 9/16-INCH-THICK CLEAR STRAIGHT GRAINED OUTER TENSION LAMINATION													
3	1-C	10.62	10.4	0.497	14,790	2,800	5,810	6,140	1.166	1,940	1,800	0	Tension
	1-P	11.17	10.9	.491	19,100	2,790	5,620	7,160		1,810	1,770	1	Tension, compression, tension
	13-C	10.60	10.1	.481	11,510	6,590	4,580	4,800	1.296	1,720	1,740	0	Tension
	13-P	11.25	10.5	.484	19,000	5,550	6,220		1,740	1,670	2	Compression, tension
MATCHED PAIRS OF BEAMS, ALL WITH A 9/16-INCH-THICK CLEAR STRAIGHT GRAINED OUTER TENSION LAMINATION													
4	2-C	10.60	10.3	0.495	23,100	2,350	7,500	9,620	0.960	2,120	2,010	7	Compression, tension 3d. lam., ten.
	2-P	10.60	10.2	.490	22,200	3,000	6,670	9,240		2,160	2,060	6	Compression, shear
	17-C	10.68	10.9	.498	22,940	1,400	5,740	9,420	.966	2,350	2,150	4	Compression, tension 2d. lam., ten.
	17-P	10.69	11.0	.503	22,200	14,050	6,970	3,100		2,300	2,120	3	Compression, tension
	20-C	10.64	10.4	.500	18,810	1,800	6,210	7,980	.954	2,070	1,980	4	Compression, tension
	20-P	10.62	10.7	.497	17,900	7,500	6,220	7,420		2,140	1,980	1	Tension 2d. lam., comp., ten.
MATCHED PAIRS OF BEAMS WITH NO MODIFICATIONS													
5	7-C	10.63	10.2	0.486	20,000	3,300	6,620	8,290	0.779	2,000	1,900	3	Compression, tension
	7-P	10.70	10.9	.480	15,790	5,040	5,720	6,460		2,000	1,840	0	Tension
	27-C	10.63	10.6	.485	15,000	3,050	4,560	6,200	.982	2,140	1,920	0	Tension at 1-5/8 in. knot
	27-P	10.63	10.6	.488	14,700	4,550	4,980	6,090		2,140	1,920	0	Tension at 1-5/8 in. knot
	30-C	10.64	10.4	.472	14,000	5,600	3,320	5,790	.960	1,940	1,880	0	Tension at 1-1/4 in. knot
	30-P	10.64	10.6	.484	13,440	5,100	5,560	5,560		1,910	1,810	0	Tension at 1-1/4 in. knot

1-All beams had a constant width of 5 inches.
2-Determined from volume at time of test and overdry weight.
3-Values in column 10 are based on moment at point of failure.
4-Values in column 11 are based on maximum moment when values in column 10 are based on some moment less than maximum.
5-Ratio of the modulus of rupture values given in column 10 for the matched pairs.
6-Values in column 13 are based on load deflection data between load points.
7-Values in column 14 are based on load deflection data between supports.

lected without causing significant changes in the results. Values of proportional limit for the prestressed beams given in column 9 were calculated using net section properties and were adjusted to include the stresses induced by prestressing at the compression face of the beams.

Two values are given for modulus of rupture, depending upon where failure occurred. The values in column 10, table 3, are based on bending moment at point of failure. If failure was in the region between a load point and a reaction, the value of the modulus of rupture based on maximum bending moment is included in column 11. The ratio given in column 12, table 3, is the ratio of the modulus of rupture of the beam (P) to the matched control (C) that is given in column 10. No ratio is given for beam pairs 9 and 24 because the control beams were erroneously tested upside down.

The modulus of elasticity values given in column 13, table 3, are calculated from load-deflection data between load points, and the values in column 14 are calculated from overall load-deflection data (not accounting for possible shear deflection). The difference between the two is an indication of the effects of shear deflection. A comparison between the modulus of elasticity for the prestressed beams and for their matched controls indicates that prestressing had relatively no effect on the stiffness of the beams. No change in stiffness was expected, since the prestressing strands were not bonded to the wood. The only factor that might have produced an apparent increase of stiffness was the increase in prestressing force that occurred due to the bending of the beam, but this in-

crease, discussed later in this report, was not large enough to produce a measurable change in stiffness.

Values given in column 15, table 3, are the number of laminations in which compression failures were visible at failure.

The type and sequence of failure are given in column 16, table 3. A careful evaluation was made of the failures to determine if a relationship could be found between the modulus of rupture and a concentration of strength-reducing features. No such relationship was apparent. One thing that was noticeable was that the very steep cross grain that surrounds some knots was extremely detrimental to strength. Failures occurred in this cross grain rather than through the knots. In some cases, the knot was small but the associated cross grain, approaching a direction perpendicular to the length of the beam, extended across most of the 5-inch width of the laminations.

In five of the beams, a tension failure occurred in lamination 2 or 3 before the No. 1 outer tension lamination failed. This was more noticeable in the beams that had the clear straight-grained outer laminations. Some designers are careful about not overstressing outer laminations but sometimes tend to neglect the inner laminations. It is true that the inner laminations are less important, but results with these five beams show that the inner laminations should be considered in the overall design because they can initiate failure.

Effective Prestress Level and Prestress Design

One of the objectives of this study was

to determine the effective prestress level for wood beams made from a low structural grade of wood. The effective prestress level was previously defined as the prestress level that, when induced in the tension side of a beam, will cause a wood beam made from a low structural grade of material to exhibit the compression-tension sequence of failure. This is possibly an elusive definition because it does not include anything about the design stresses or the strength of the material being prestressed. It is nearly impossible to give a sound definition of effective prestress level based on the design stresses or strength of the wood without knowing the relative difference in the tensile and compressive strengths of the wood being prestressed.

The compression failures that were visible in the beams at failure are given in column 15, table 3. The prestressed beams consistently failed in compression in all groups except 1-1600. In this group, no compression failures were visible in two beams, one lamination failed in compression in one beam, and three laminations failed in compression in the other two beams. Beam 24-P in the 1-2000 group did not have any visible compression failures. This beam was prestressed, loaded to 10,000 pounds, unloaded, unstressed, restressed, and then loaded to failure. This method of testing associated with the 1:5 cross grain in the outer tension lamination may have influenced the results of beam 24-P.

On the basis of these observed failures and the type and sequence of failure given in column 16, table 3, the minimum prestress level that produced the desired compression-tension sequence of failures was induced by the 1,800 p.s.i.

design. The designs which induced stresses larger than those induced by the 1,800 p.s.i. design also produced the compression-tension sequence of failures but, as predicted, the additional induced stresses had no beneficial effect. As shown in table 4, the average modulus of rupture of the prestressed beams in groups with larger induced stresses was approximately equal to the average for beams in the 1,800 p.s.i. design group. The stresses induced in the beams in this group were calculated to be 1,800 p.s.i. compressive stress in the outer tension face and 480 p.s.i. tensile stress in the outer compression face. The algebraic sum of these induced stresses is 1,320 p.s.i. compressive stress. Comparing them another way, if such a prestressed beam was loaded with an external moment sufficient to counteract the induced stress in the compression face, then there would still be a 1,320 p.s.i. induced compressive stress in the outer tension face. Based on these observations, the relative difference in the compressive and tensile strength of beams fabricated from L-3 grade Douglas-fir must be such that an approximate 1,300 differential of induced stresses will produce the desired type and sequence of failure. Therefore, it is believed that any design that induces stresses whose algebraic sum is approximately 1,300 p.s.i. compressive stress will cause the compression-tension sequence of failure in the grade of beams that were evaluated here. This 1,300 value should vary inversely with the change in the quality of the wood beams because of the relative changes in the tensile and compressive strengths of the wood as has been previously dis-

cussed. Based on the original definition and discussion of prestress level, the effective prestress level is determined to be a 1,300 p.s.i. compressive stress induced in the outer tension face which diminishes uniformly to a zero stress in the outer compression face. Such an induced stress level should produce a combined tensile force that was sufficient to utilize the existing compressive strength of a wood beam fabricated from L-3 Douglas-fir.

At this point a few comments are needed to clarify the theoretical design which will induce the effective stress level into a beam. A design that induces the effective stress level as it has been

previously defined and determined is not theoretically the most effective design. Heretofore the arguments for prestressing were to reinforce the tensile force component enough to insure complete utilization of the compressive strength of the beam. The most effective design would be one that would not only induce stresses whose algebraic sum was 1,300, or some other predetermined value depending upon the quality of wood being prestressed, but also induce these stresses in such a manner as to reinforce both the compressive and tensile force component of the internal stress couple.

As has been previously discussed and

Table 4.--Average modulus of rupture values for prestressed and control beams

Group	Beams	High	Low	Average	Ratio ¹
		P.s.i.	P.s.i.	P.s.i.	
1-1,600	Control Prestress	5,680 8,100	4,100 6,650	4,900 7,350	1.50
1-1,800	Control Prestress	7,240 8,960	5,560 7,900	6,370 8,350	1.31
1-2,000	Control Prestress	7,260 9,100	4,170 6,920	6,040 7,820	1.29
1-2,400	Control Prestress	7,860 9,140	5,660 7,020	6,470 7,990	1.23
1-2,800	Control Prestress	8,500 8,740	5,220 7,440	6,370 7,910	1.24
All of Group All of Group Group 3&4	Control Prestress P ²	8,500 9,140 9,240	4,100 6,650 6,220	6,030 7,880 7,830	1.31

¹Ratio of prestress to control values given in column 5

²Beams modified with a clear outer tension lamination.

as shown in figure 1, there are two components of the prestressing force, the axial component which is equal to $\frac{P}{A}$ and a moment component which is equal to $\frac{Pe}{Z}$. As is obvious from these components, the magnitude and location of the prestressing force are important factors which influence how stresses are induced into a beam. The axial component is directly proportional to the prestressing force regardless of its location, but the moment component is dependent upon both the magnitude and location. The induced stress in the outer tension face is equal to the sum of these two components and the induced stress in the outer compression face is equal to the difference of these two components. The magnitude of the prestressing which is required to induce stresses whose algebraic sum is equal to some desired value (1,300 for the grade of beams that were evaluated) is easily determined by adding the components that induce the stresses in the top and bottom faces of the beam. To make this algebraic addition, compressive stresses are assumed negative and tensile stresses are assumed positive.

$$\begin{aligned}
 & \text{(stresses at top)} \\
 & \quad + \\
 & \text{(stresses at bottom)} \\
 & \quad = -1,300 \\
 & \quad \left(-\frac{P}{A} + \frac{Pe}{Z} \right) \\
 & \quad + \\
 & \left(-\frac{P}{A} - \frac{Pe}{Z} \right) = -1,300 \\
 & \quad -\frac{2P}{A} = -1,300 \\
 & P = \frac{1,300}{2} A = 650 A
 \end{aligned}$$

This shows that the magnitude of the most effective prestress force is equal

to one-half of the desired algebraic sum of the induced stresses multiplied by the cross-sectional area of the beam. Any force greater than this would produce stresses whose algebraic sum was greater than 1,300; thus part of the steel would not be effective. The effective stress level, as it has been defined, can be induced into the beam if the location of the steel is such that the stress in the compression face is zero. Or

$$-\frac{P}{A} + \frac{Pe}{Z} = 0$$

$$e = \frac{d}{6}$$

where d is the depth of the beam and $\frac{1}{6} d$ is measured from the centroidal axis toward the tension face of the beam. For this placement of prestressing force, the induced compressive stress in the outer tension face is equal to $-\frac{P}{A} - \frac{Pe}{Z}$ which computes to be -1,300 p.s.i. for the grade of beams being discussed here. Thus, with the prestressing force equal to $650 A$ located at $\frac{1}{6} d$ from the centroidal axis, the tensile force component of the internal couple should be reinforced a sufficient amount to balance the compressive force component produced by the compressive strength of the beam. With such a design the compressive force component has not been reinforced.

As was stated previously, such a design is not the most effective design because, by again examining the components of prestressing, it is easily seen that the moment component can be changed by changing the location of the prestressing

force. This assumes that the magnitude of the prestressing force is held constant and equal to one-half of the desired algebraic sum of the induced stresses multiplied by the cross-sectional area. Any change in the location of the prestressing force from the $d/6$ distance from the centroidal axis would produce an equal change in the magnitude of the induced stresses in the outer faces of the beam. Suppose, for example, that the steel was moved to the centroidal axis, then the moment component is zero and a minus or compressive stress equal to P/A is induced in both the compression and tension face of the beam. If P is equal to $650 A$, then the compressive stress is equal to 650 p.s.i. As is seen, the algebraic sum of the induced stresses is still 1,300 and such a design should produce the desired compression-tension sequence of failure. The effectiveness on load-carrying capacity of a beam prestressed in such a manner is reduced because the induced compressive stress in the compression area of the beam acts as a negative reinforcement or a reduction in the compressive force component of the internal resisting couple. However, such a prestressed beam would probably have greater flexural strength than a comparable unstressed beam. Some value of compressive stress will be induced in the compression region any time the steel is moved closer than $d/6$ to the centroidal axis. Therefore if the minimum effective design is defined as one that induces stresses that always add to tensile or compressive force components of the internal couple, then for a minimum effective design the prestressing force for beams of the grade evaluated

here should equal $650 A$ and be located at $d/6$ from the centroidal axis toward the tension face.

The effectiveness or load-carrying capacity of the prestressed beam can be increased if the steel is moved farther away from the centroidal axis than $d/6$. Any increase in this distance adds an equal increment of induced stresses to both the tension and compression faces which acts to reinforce or increase the tensile and compressive force component of the internal resisting couple, thus increasing the load-carrying capacity of the beam.

Theoretically, the most effective location of the prestressing force would be at the outer tension face of the beam. For post-tensioning prestressing with unbonded strands, this location is not possible because, for this prestressing, the stress from the prestressing steel is transferred to the wood through end bearing plates. The allowable stress under the end bearing plates will dictate the required area of wood for safe transfer of stress and thus dictate the location of the prestressing force. In designing the end bearing plates, the steel was located at the lower 1/3-point of the height of the end bearing plate. Such a location should produce an approximate triangular end bearing stress distribution with maximum stress at the bottom or outer tension face of the beam and with no stress at the top of the end bearing stress area.

The maximum end bearing stress was computed to be slightly higher than the induced stress in the outer tension face (table 1). Thus, the end bearing stress limitation is an important factor to consider when designing post-tensioned

prestressed wood beams.

The steel could have been located at the center of the end bearing plates which would have produced a uniform end bearing stress pattern. With this pattern, the end bearing transfer area would extend from the bottom of the beam up to somewhere near the centroidal axis of the beam. At the top of the transfer area the beam would be subjected to a high end bearing stress, then immediately above the transfer area, no end bearing stress. This might produce a high stress concentration in an area where shear stress is already high and might cause the beams to fail in shear before flexural strength is reached.

During this investigation, no attempt was made to evaluate the stress transfer at the end of the beams and no failures were observed under the end bearing plates.

The beams with higher prestresses did exhibit more compression failures than the lower stressed beams. Figure 8 shows pronounced compression failures of one beam in the 1-2800 group. From previous discussions of utilizing the existing compressive strength, it could



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Figure 8.--Failure pattern that is produced with a compression-tension sequence of failure.

be concluded that as additional laminations are forced to fail in compression the load-carrying capacity of the beam should continue to increase. Refer to figure 2A: if y continues to increase, the magnitude of the compressive stress solid should increase, thus increasing the internal couple which resists external forces. This was found to be untrue. As the prestressed beams were loaded, the load continued to increase until about one-fifth of the laminations (3 or 4) had failed in compression. Then, with increased deflections, additional laminations would fail in compression before final tension failure of the beams occurred, but the load on the beam remained essentially constant or decreased slightly.

It was difficult to determine a reason for the additional compression failures with no increase in external load. They could be partially caused by the axial component of the prestressing force. This axial force induces compressive stresses across the full cross section of a beam (fig. 1C). The moment component of the prestressing counteracts the compressive stress in part, but not all, of the compression region. Therefore, the axial force may become detrimental after some initial compression failure. These observations are indirectly substantiated by the test results because the occurrence of the additional compression failures was more frequent as the prestressing level of the beams increased. As the prestressing level increased, the axial component of prestressing also increased.

Therefore, the prestress design of the beam should be such that the desired stresses would be induced into the beam

with a minimum axial component of prestressing force.

Increase in Strength

The modulus of rupture values for the beams are given in table 3. The average and range of modulus of rupture values for the prestressed beam groups are given in table 4. The values for both the control and prestressed beams of group 1-1600 are considerably lower than for all other groups. No good reason could be determined for the consistently low values in this group, since the laminations for all the beams and the beams for each group were chosen at random. The modulus of rupture values for group 1-1800 and the remaining groups are essentially equal. As was predicted, the extra induced stress above an effective stress level did not increase the strength of a prestressed beam. Even though the 1,600 p.s.i. design is below the effective design value of 1,800 p.s.i., the failure characteristics of group 1-1600 were such that the results of these beams could not be excluded from an analysis to determine how prestressing increases the strength of a wood member. In the following discussion all beams in group 1 were considered as a composite group of equally comparable beams.

The range of values of modulus of rupture for the 25 control beams is 4,100 to 8,500 p.s.i., with an average of 6,030. The range for the 25 prestressed beams is 6,650 to 9,140 p.s.i., with an average of 7,880. This shows a 31 percent increase in average values due to prestressing.

A comparison of the range of values for modulus of rupture shows that pre-

stressing decreased the variability in the strength of the wood beams. The cumulative frequency distributions for both control and prestressed beams in group 5 are shown in figure 9. The coefficient of variation of modulus of rupture is 9.6 for the prestressed beams and 18.2 for the control beams. The increase in modulus of rupture associated with the decrease in variability of strength indicates that wood beams can be significantly upgraded by prestressing.

Prestressing becomes more beneficial as the quality of the beams decrease. A check of the ratio of the modulus of rupture values of the prestressed beams to those of the matched controls given in column 12, table 3, shows that the highest ratios are associated with the control beams with the lowest strength. Also, a check of the ratios given in table 4 of the average modulus of rupture values shows that the highest percentage increase was produced by the 1,600 p.s.i. design. One might jump to the conclusion that this was the best design but it is believed that the high percentage increase is due to the consistently low values of modulus of rupture for the control beams. Based on the failure characteristics of this design group, the ratio of percentage increase probably would have improved slightly if the beams had been prestressed to a higher stress level. Figure 10 shows the relationship between the strength of the control beams and the percentage increase obtained by prestressing. The results of all the prestressed beams are plotted in this figure.

Again it is noted that the success of prestressing will depend upon the creep characteristics of the wood. Unfortunately these are not known. The data

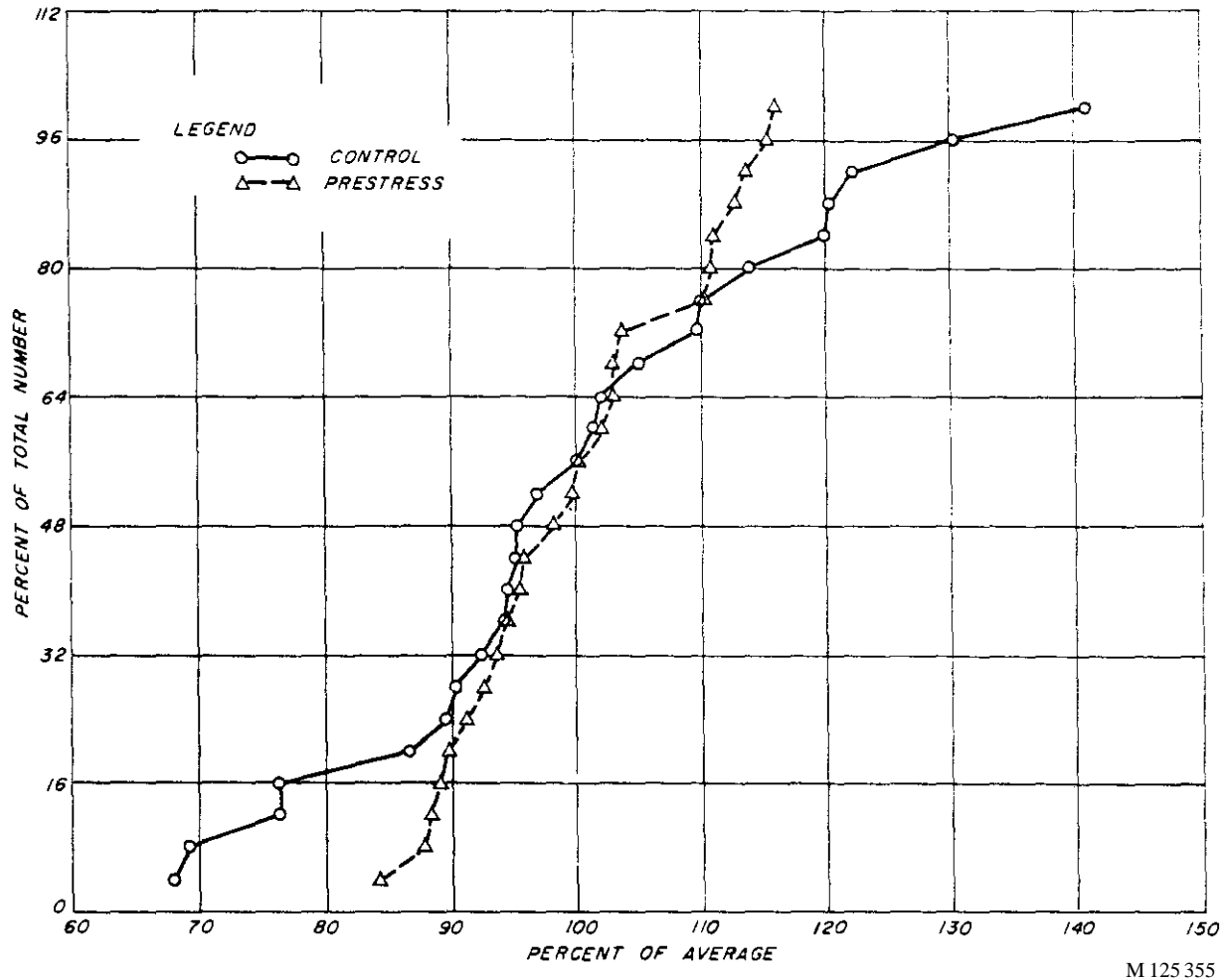


Figure 9.--Cumulative frequency distribution for prestressed and control beams.

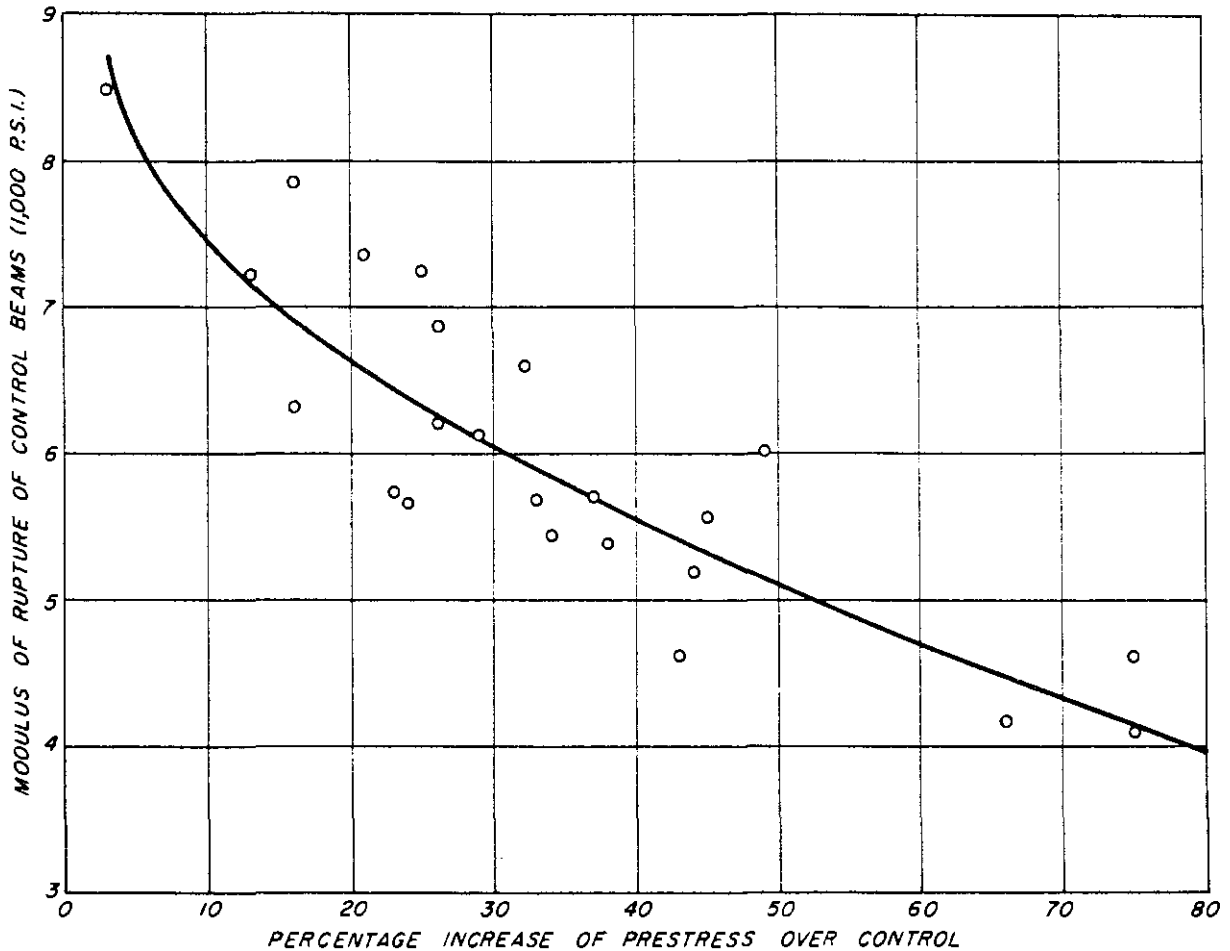
given here are the results of short time tests from which the problems of creep were nearly eliminated.

Mechanics of Prestressing

The calculated and experimental values of camber and induced stresses are given in table 5 as is the initial prestressing force. The experimental stresses are calculated from measured strains on the outer faces of the beams and the average modulus of elasticity of the beams that was determined by load-

deflection data measured between load points.

The calculated camber agrees reasonably well with the measured camber, but some variation is noted between the calculated design values and the prestress stresses. The accuracy of the experimentally determined stresses depends upon the accuracy to which the strains were measured. These were measured with 1-inch electrical resistance strain gages. The accuracy of the 1-inch gages on material having variable slopes of grain could be questionable. Another possible reason for the differ-



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Figure 10.--Relationship between the strength of a wood beam and the percentage increase that is obtained by prestressing. The curve indicates trend only and was not calculated as the line that best fits the data.

ences in stress is the exact location of the neutral axis of the beam cross section. The beams contained 19 laminations which had a considerable variation in their elastic properties. Therefore, the neutral axis based on the weighted properties of each lamination may have been different than the assumed axis based on the unweighted gross cross section.

The increase in prestressing force that occurs as a prestressed beam is loaded is shown in figure 11. A calculated increase in prestressing force is

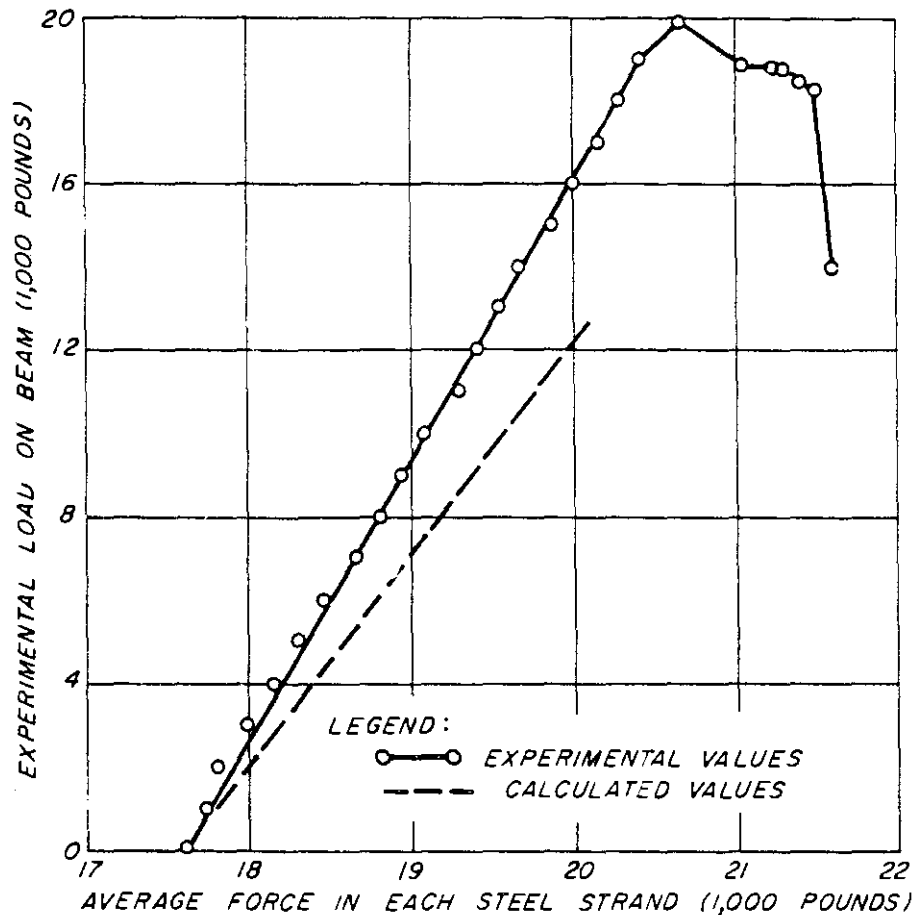
shown by the dotted line in the same figure. A comparison of experimental to calculated values shows that the steel stress does not increase as fast as would be predicted by theoretical calculations. The difference between the two values might be an indirect indication of creep relaxation features of wood. Wood is known to creep when first subjected to stress, and the time period over which the beam was being loaded and during which the increase in steel stress was measured was approximately 15 minutes.

Table 5.--Calculated and measured initial prestressed values of stresses and camber

Group	Beam	Prestress stresses ¹		Prestressing force	Camber	
		Top	Bottom		Calculated	Measured
		P . s . i	P . s . i .	Lb.	In.	In.
1-1, 600 ²		550	1,600	28,000		
	5-P	680	1,400	28,000	0.59	0.58
	6-P	480	1,400	28,000	.63	.61
	16-P	680	1,850	28,000	.56	.64
	21-P	630	1,450	28,200	.58	.60
	23-P	480	1,460	28,000	.54	.56
	1-1, 800 ²		480	1,800	35,200	
8-P		520	1,800	35,200	.55	.60
10-P		420	1,820	35,500	.56	.58
12-P		440	1,790	35,000	.55	.55
19-P		470	2,050	35,200	.56	.61
33-P		600	1,800	35,200	.56	.56
1-2, 000 ²			480	2,000	39,000	
	18-P	570	2,270	39,400	.67	.66
	24-P	600	1,790	39,000	.67	.62
	25-P	630	2,130	38,800	.60	.64
	26-P	510	2,200	40,000	.65	.70
	32-P	500	2,090	39,400	.67	.69
	1-2, 400 ²		420	2,400	53,100	
4-P		360	2,450	53,200	.69	.69
15-P		350	2,400	53,000	.72	.68
28-P		470	2,150	53,000	.83	.76
29-P		480	2,520	52,900	.87	.80
31-P		520	2,650	53,000	.75	.81
1-2, 800 ²			140	2,800	71,200	
	3-P	250	2,900	70,800	.78	.82
	9 - P	150	3,110	71,000	.80	.84
	11-P	170	2,810	71,200	.78	.80
	14-P	110	3,060	71,600	.78	.75
	22-P	380	2,810	71,200	.79	.88

¹Top stresses are tension and bottom stresses are compression. Stresses were calculated from measured strains on top and bottom faces and modulus of elasticity that was determined between load points.

²Values opposite the beam group numbers are the calculated design values.



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Figure 11.--Curve showing increase in force of the steel in Beam 12-P as the beam was loaded to failure.

Matching of the Beams

The beams in groups 4 and 5 were evaluated to determine if the beams of structural size containing 19 laminations could be successfully matched. Except for beam pair 7, the modulus of rupture values given in table 3 indicate that the beams were closely matched. The range of percentage differences of modulus of rupture values for five beam pairs, all of group 4 and beam pairs 27 and 30 of group 5, is from 2 to 5 percent with an average of 3.6 percent. These percentage differences represent beams having

modulus of rupture values from 5,560 to 9,620 p.s.i.

The difference in modulus of rupture values for beams in pair 7 is 22 percent. During the final planing of the laminations, lamination 2 of this pair was broken. This lamination was replaced in both beams with matched laminations. During this replacement, the lamination was turned over in one beam. This would cause a transverse displacement of the knots and other characteristics in one beam as compared to its matched control. Upon examination of the failures, it was evident that this transverse displacement

of knots caused a worse concentration of knots in beam 7-P than it did in 7-C. It is difficult to predict the effect of this concentration, but it could have been enough to have caused the difference in the strength of the two beams.

The method of matching is considered to be very good if the resawn matched laminations are placed in the beam pairs in exactly the same way so that the corresponding characteristics are in the matched beams in both the longitudinal and transverse directions.

Beams with Clear Laminations

The modified beams, beams marked with a P, of groups 2 and 3 had 19 L-3 grade laminations plus 1 outer tension lamination that was a clear straight-grained board free from all visible strength-reducing features. All beams of group 4 had 18 L-3 grade laminations plus 1 clear outer tension lamination. The clear lamination in group 2 was 1-1/2 inches thick and in groups 3 and 4 it was 9/16 inch thick. The results given in tables 3 and 4 indicate that the clear outer tension lamination had nearly the same effect upon the strength and type of failures as did the prestressing. The average values of modulus of rupture for the prestressed beams was 7,880 p.s.i., as compared to a value of 7,830 for beams of groups 2 and 3 with the one clear 9/16-inch-thick lamination. All the beams with the clear lamination had visible compression failures before final failure of the beam occurred.

The failures of beam 34-P (fig. 12) indicated that this beam had balanced strength properties. At final failure of

the beam, every lamination had failed either in tension or compression. The external load on the beam continued to increase as the compression failures were occurring. This increase in load-compression failure relationship is different than existed for the prestressed beams, as was discussed earlier.

The first reaction to the comparable results of the beams with clear laminations to the prestressed beams might be that prestressing has no beneficial effect if the same results are obtained by using clear outer tension laminations. This is a false conclusion. The results of the beams with the clear laminations are somewhat weighted because it would not be economically possible to produce such beams. These outer tension laminations were carefully hand-picked from a lumberyard supply. It would be impossible to select such laminations with the restricted uniform strength features



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Figure 12.--The failures of Beam 34-P indicate that this beam had balanced strength properties.

in any large quantities. If the outer laminations were made from a straight-run grade of clear material, there probably would still be a considerable variability in their strength properties. Prestressing; would produce more consistently uniform results.

One thing that is apparent from the results of the beams with clear laminations is that the outer tension lamination

does have considerable influence upon the strength of a wood beam. This influence may be more than is realized in present design methods. The reliability of laminated beams probably could be increased if rigid restrictions of allowable strength-reducing features were placed on the boards used as outer tension laminations.

CONCLUSIONS

1. An immediate effective prestress level (algebraic sum of induced flexural stresses) of 1,300 p.s.i. was sufficient to utilize the compressive strength of laminated beams made of grade L-3 Douglas-fir.

2. Prestressed beams of L-3 Douglas-fir were, on the average, 31 percent stronger than matched, unstressed beams.

3. Variability of prestressed beams of

L-3 Douglas-fir was about one-half the variability of matched, unstressed beams.

4. A clear, straight-grained outer tension lamination may have more influence on beam flexural strength than is recognized by present design methods.

5. Pairs of structural sized beam can be matched if the corresponding laminations in the beams are the resawn halves of a nominal 2-inch board.

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