



Comparative

Fire Test

of

Timber and Steel

Beams

# Comparative Fire Test of Timber and Steel Beams

## INTRODUCTION

The fact that large wood structural members provide a substantial degree of fire endurance was recognized as early as 1800 when heavy timber framing assumed importance in the design of factory buildings. When a group of cotton and woolen mill owners organized in 1835 for mutual protection of their property from damage by fire, this system of construction increased in prominence and became identified as "mill construction." The outgrowth of this organization was the formation of what is now known as the Associated Factory Mutual Fire Insurance Companies.

A long history of over 150 years of excellent performance in numerous severe fires has established the superior fire endurance of heavy timber framing. The term, "mill construction," originally applied to this type of building, has gradually disappeared in favor of the more definite term, "heavy timber construction," as now recognized in major building codes throughout the country.

With the advent of modern structural glues and gluing techniques in recent years, the engineered timber fabricating industry has greatly expanded the scope of architectural application for "heavy timber construction."

Originally used for factories and warehouses, it has also become a preferred method of construction for churches, schools, and other places of assembly. There are many outstanding examples of this new method of timber framing which utilizes glued-laminated structural members in a variety of sizes and shapes to fit a broad range of architectural forms.

For many years, heavy timber framing has been considered a preferred risk by fire insurance rating bureaus and a superior type by fire protection authorities. The records include numerous examples in which buildings of heavy timber construction have been restored to use at minimum expense after severe fire ex-

posure. In many cases, removal of char by sandblasting or other means was the only repair work needed.

Expansion of American industry during the 20th Century was accompanied by the widespread use of unprotected steel framing in the types of buildings for which heavy timber originally had been used. One reason given for this change was the fact that timber burns and steel does not. Those who manufacture non-combustible building materials have placed great emphasis on the need for such materials in building construction.

Such emphasis on noncombustibility has served to confuse the building public. As a result there are some who believe that any material which will not burn will automatically endure exposure to fire. This is far from correct. It is, therefore, important that those responsible for designing or constructing buildings have an understanding of the true meaning of fire endurance.

Wood burns when exposed to fire and a wood member will lose strength as its cross-section is reduced by charring. When structural steel is exposed to fire it will lose strength in the range of 800 to 900 degrees Fahrenheit and will fail to support load beyond 1200 degrees F. Thus, reliance on noncombustibility alone is not a proper basis for evaluating endurance to fire exposure.

As a service to the building public, the lumber industry has sponsored a full-scale test to substantiate the fire experience record of heavy timber. In this test, identically loaded, unprotected timber and structural steel members were exposed simultaneously to fire of standard intensity. Since exposed timber and steel framing are widely used for commercial, industrial and assembly buildings, it was considered proper to evaluate their relative fire endurance properties in a test of this type.

This report deals with the results of such a comparative test conducted by the Southwest Research Institute, San Antonio, Texas.

## Test Criteria

The objective of this test project was to determine the comparative performance of fully loaded timber and exposed structural steel roof framing members to fire temperatures reasonably simulating those existing in actual fires. The following test criteria were established to provide a practical and equitable procedure for the collection of factual data:

1. The test structure should be sufficiently large that the timber and steel members to be evaluated could be of a size and span representing full-scale roof framing.
2. The test enclosure should be such that both framing systems could be exposed simultaneously to equivalent fire conditions, and so arranged that each system could react independently.
3. A roof load calculated to develop the design capacity of each member should be applied throughout the period of fire exposure.
4. Exposure temperatures in the test enclosure should follow those set forth in the Standard Time-Temperature Curve, as specified in American Society for Testing Materials Designation E-119, which is the standard reference in testing for fire endurance.

## Test Structure and Equipment

**Test Structure**—The test was conducted at Southwest Research Institute, in a structure measuring 20 feet in width and 60 feet in length. It is a reinforced concrete frame building enclosed by concrete-block

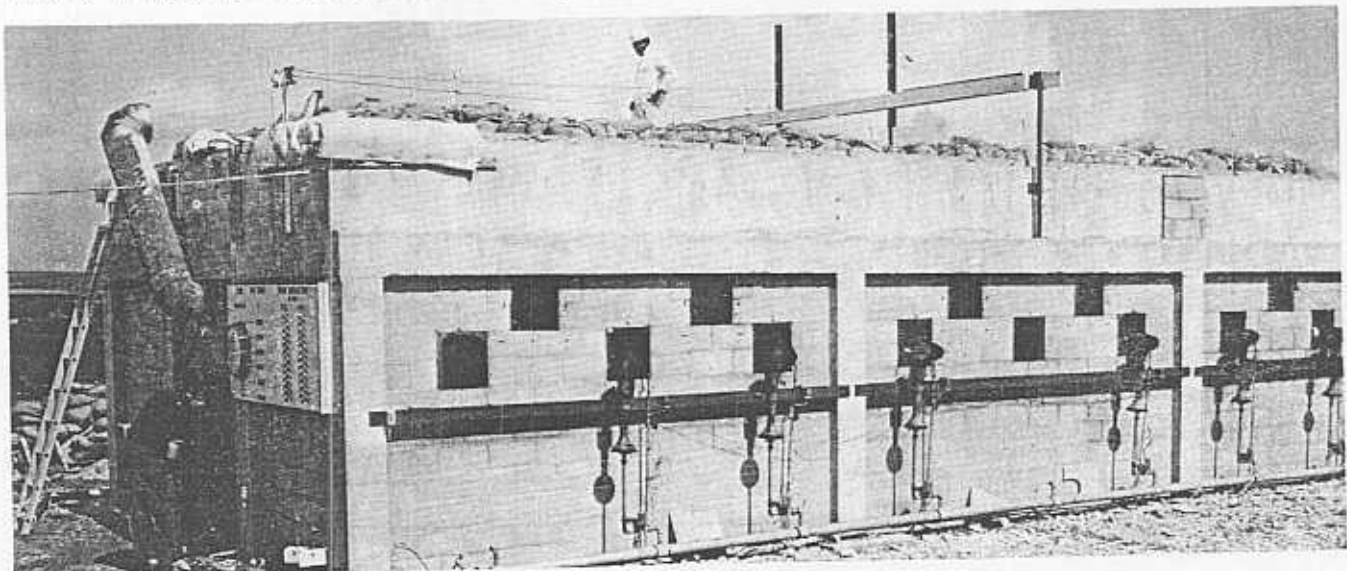
panel walls provided with ports for mounting gas burners and with vents for combustion control, as shown in Figure 1. The upper half of the interior wall is surfaced with two inches of insulating block. For this test, a wall with pilasters was constructed at the three-quarter point to provide end-support for the beams.

**Roof Framing System**—The two beams to be evaluated were installed as the supporting members of the roof structure as illustrated in Figure 2. The clear span for the roof framing members was 43 feet, 3 inches, with each beam supporting half of the total roof load.

The left panel was supported by a 16-inch rolled steel beam (16 WF 40) designed for the applied roof load in accordance with recommendations of the American Institute of Steel Construction. The right panel was supported by a 7" x 21" glued-laminated timber beam, using casein glue and without chemical treatment, designed in accordance with the National Design Specification for Stress-Grade Lumber and Its Fastenings, recommended by the National Lumber Manufacturers Association, and the design standards of the American Institute of Timber Construction. Both beams were supplied with 2 inches of camber to offset initial deflection.

The roof deck construction consisted of bulb-tee sections spaced at 32 $\frac{5}{8}$  inches on center and attached to the top edges of the beams and to the exterior walls. One-half-inch gypsum form board was placed on the bulb tees to receive the lightweight concrete deck which was poured to a depth of 2 $\frac{1}{2}$  inches. To provide lateral support to meet design calculations for the steel beam, two tee sections (T2 x 2 x 3.56) were attached to the top edge of the steel beam at third points. Attachment

Figure 1. Exterior view of the test structure showing type of construction, location of gas burners and other details.



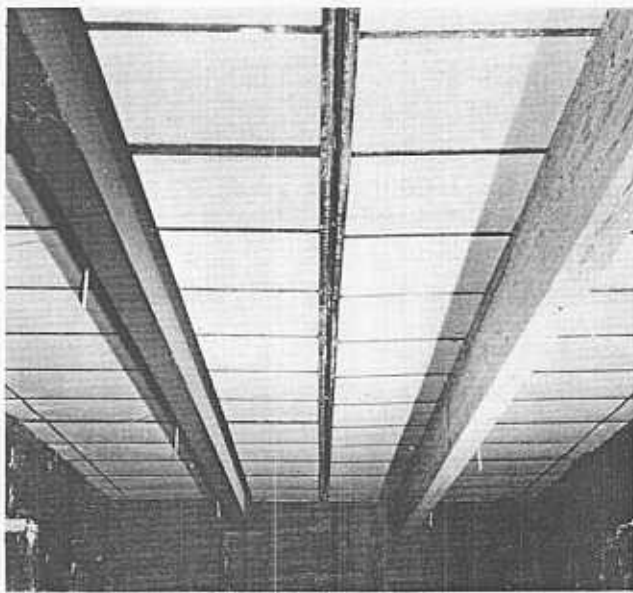


Figure 2. Interior view of the test structure showing characteristics of each roof support system, longitudinal joint and location of thermocouples.

of all tee sections to the framing members was by fillet weld to the steel and lag screws to the wood.

The 7" x 21" wood beam was selected because it met the requirements of the design. The induced stress was 1552 pounds per square inch and the calculated deflection was 2.32 inches or  $1/224$ . This deflection is within limits commonly specified in building codes and was the controlling factor in selecting the wood beam.

The 16 WF 40 steel beam was selected because it met the requirements of the design and is a stock item.

The calculated deflection was 1.51 inches or  $1/344$ . The induced stress was 12,524 pounds per square inch. This beam was designed to recognize the effect of depth to span ratio by installing lateral supports at the third-points in the span.

The two sections of the roof deck were entirely separated by a longitudinal joint, 2 inches wide, which was covered with a flexible insulating blanket. This allowed each panel to move independently for a vertical distance of 36 inches without loss of heat in the structure. A typical cross section of the test structure showing the roof deck construction is shown in Figure 3.

**Roof Load**—The total design load on the roof consisted of an applied live load equivalent to 30 pounds per square foot of roof surface, plus the dead load weights of the deck construction and the test beams. This resulted in a total load of 12,346 pounds for the wood beam and 12,432 pounds for the steel beam. The slight difference in total load is due to the lesser weight of the wood beam.

The live load consisted of bagged sand carefully weighed and positioned over each beam in the amount necessary to provide the 30 pounds per square foot required. A view of the test structure with sand bags in place over the beams is shown in Figure 4.

**Heat Source**—Heat was supplied by six industrial-type gas burners positioned on each side of the test

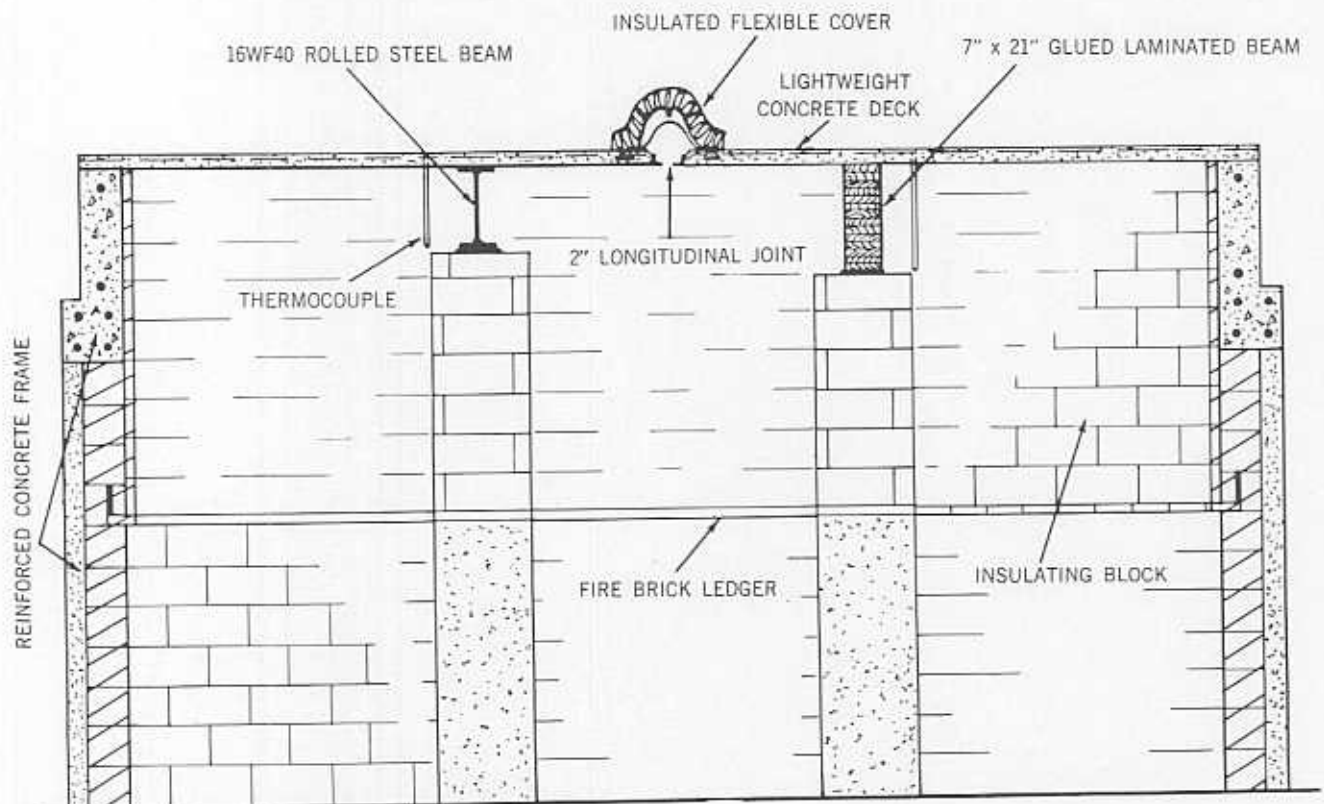


Figure 3. A section through the test structure showing construction details.

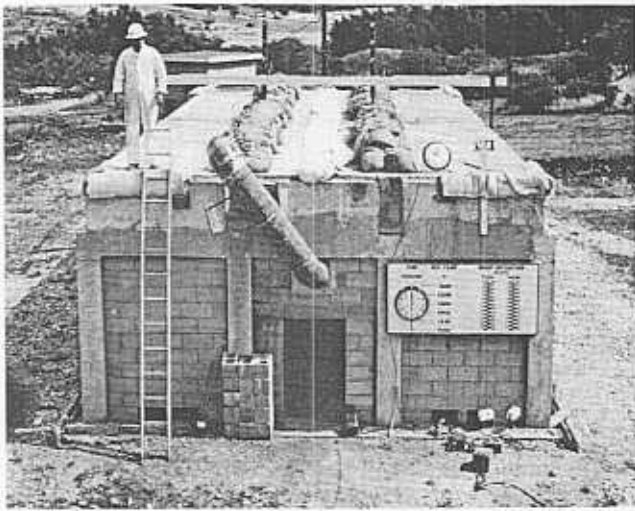


Figure 4. Exterior view of front and roof of test structure showing sand bags in position and details of the deflection measuring device.

structure, as shown in Figure 1. The building was provided with intake and exhaust venting to permit uniform combustion. The gas flow was regulated at a main control valve to furnish pressure of 20 pounds per square inch at the individual burners. In addition to the main control valve, each burner was provided with a manual control to permit more precise temperature

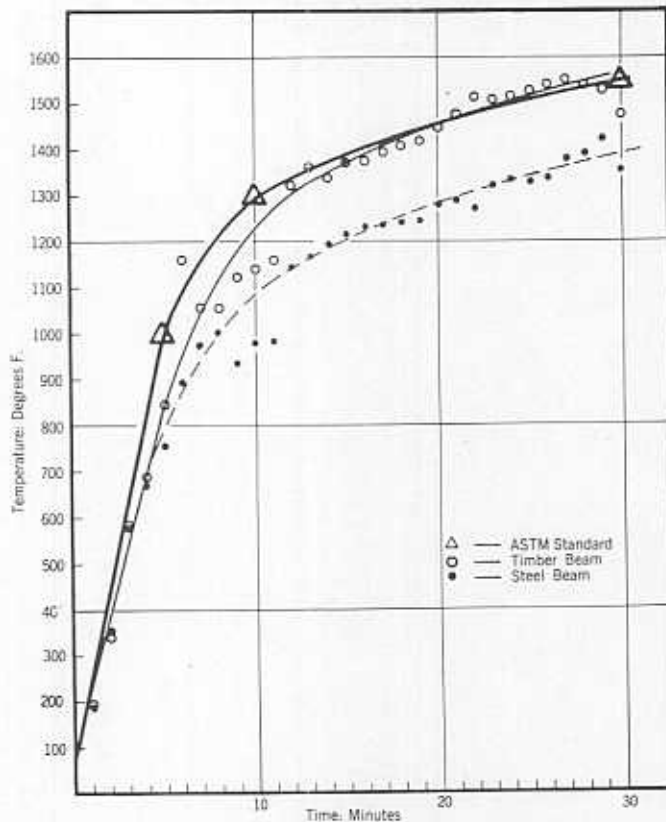


Figure 5. Conformance of test-chamber temperatures to ASTM Standard Time-Temperature Curve.

adjustment in the test structure. During the test, flow of gas was regulated to provide uniform test chamber temperatures to follow the ASTM Standard Time-Temperature Curve.

**Temperature Control**—A Minneapolis-Honeywell (Brown), eight-point recording potentiometer was located close to the main gas control valve. From this point, observation of the recorded temperatures enabled the test engineer to make prompt adjustments of fuel flow to insure that the Standard Time-Temperature Curve, specified in ASTM Designation E 119, was being followed.

This instrument recorded temperatures at eight thermocouples which were located at fifth points of the span of each beam and 6 inches to the side, as shown in Figures 2 and 3.

**Deflection Recording**—The measuring device consisted of graduated vertical rods mounted above the deck, at the midpoint of each beam, and calibrated to indicate deflection at the horizontal line formed by a cross beam supported independently from the walls of the building. Details of the device are shown in Figures 1 and 4. Continuous deflection readings were made through an eight-power viewing scope mounted on a tower 50 feet from the test structure. A clock, positioned

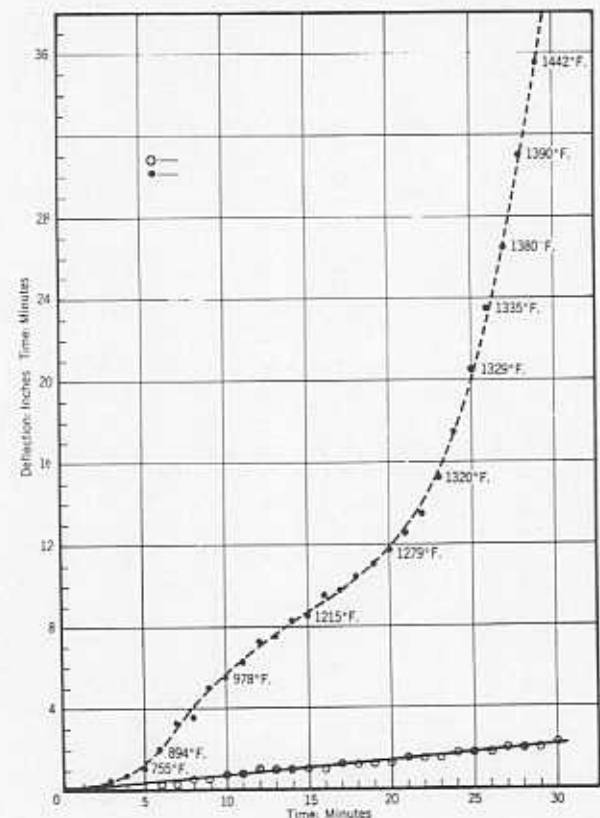


Figure 6. Comparative deflection of wood and steel beams during the test.

on the front edge of the roof, enabled the test engineer to record time and deflection simultaneously.

### Progress of The Test

After the 12 gas burners were lighted to start the test, the flow of gas was regulated to control temperatures, during the period of fire exposure, within the limits of accuracy specified in ASTM Designation E 119.

This is illustrated in Figure 5, which is a graphic comparison of the average temperatures near the wood and the steel beams, with those required by the Standard Curve. An analysis of this information indicates that the area under the average test chamber temperature curve was within 10 per cent of the area under the Standard Curve.

The principal evidence of progress was the deflection of the steel-supported roof panel. This progress is illustrated graphically by plotting deflection in inches against time in minutes, as shown in Figure 6.

Deflection of the steel-supported panel occurred early in the test and continued at a reasonably uniform rate for 20 minutes. At this point, a sharp rise in deflection rate of the steel beam developed and increased rapidly until the conclusion of the test.

After 29 minutes of fire exposure, the steel-supported roof panel had exceeded the 36-inch capacity of the deflection measuring rod. By comparison, the wood beam at this time had deflected a total of only 2 inches. At 30 minutes, the steel beam collapsed into the structure, bringing its section of the roof down and making it impossible to maintain temperature control to continue the test.

When the burners were shut off, flaming of the wood beam ceased promptly. To substantiate this

Figure 8. Exterior view of the test structure after the test showing workmen unloading the wood supported panel prior to removal for inspection and measurement of the wood beam.

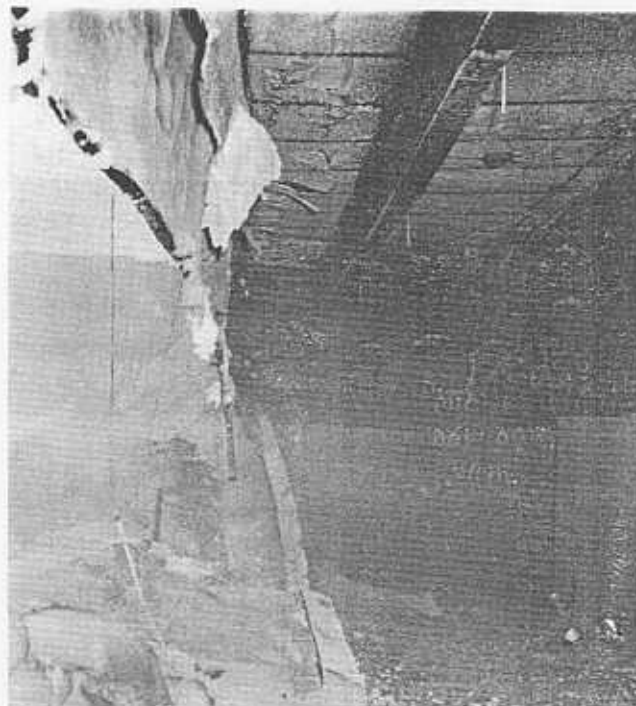


Figure 7. Interior of the test structure immediately after the steel had collapsed indicating that flaming on the wood member had ceased as soon as the source of heat was removed.

statement, Figure 7 is a view of the under side of the roof before a fire hose was applied to cool the structure, prior to disassembly of the roof panels. The water was also used to extinguish the afterglow, on the laminated wood beam, to obtain an accurate measurement of the amount of char on the wood beam at the time of failure of the steel beam.

The wood supported panel was unloaded and removed, intact, from the structure, as illustrated in Figure 8. The beam was then sawed in two at the midpoint to determine the amount of undamaged wood remaining after the fire exposure.

### Test Results and Conclusions

This test compared the fire endurance of an untreated, heavy timber beam with that of an unprotected, heavy steel beam having equivalent structural capacity. The results indicate that the fire exposure was unbiased and that it conformed to the ASTM Standard Time-Temperature Curve.

Although the recorded temperatures adjacent to the steel beam did not represent as severe an exposure as that for the wood beam, the average test chamber temperatures were within the specified limits of accuracy defined in ASTM Designation E 119. Thus, it is evident that the objective of the test was achieved in an impartial and factual manner.

There are three significant results of the test: 1) the deflection pattern for the steel beam; 2) the amount of undamaged wood remaining in the glued-laminated beam at the time of failure of the steel; and 3) the relative endurance of the two framing systems under design load.

A statement of these results and the conclusions indicated are as follows:

1. Six minutes after the burners had been lighted, the temperature near the steel member was 894°F. and the deflection was 2 inches. At 14 minutes, when this temperature was 1194°F., the deflection had increased to 8¼ inches. This relatively uniform increase continued for 20 minutes, when the deflection reached 11¾ inches at a temperature of 1279°F.

After 20 minutes of fire exposure, the deflection rate increased rapidly until it had reached 35½ inches at 29 minutes. The temperature near the steel at this time was 1422°F.

Although the steel supported panel fell into the test chamber after 30 minutes of exposure, it is evident that its structural integrity was in doubt long before this.

The deflection-temperature relationships, indicated by these results, are in agreement with the generally recognized behavior of steel structural members when exposed to building fires. That is, unprotected steel begins to lose strength at 800° to 900°F., and will fail to support load over 1200°F. The condition of the steel beam, pictured in Figure 9, provides additional evidence to support this conclusion.

2. The wood beam continued to support its full design load, throughout the test, with a maximum deflection of only 2¼ inches at 30 minutes. The uniform deflection rate of the wood beam demonstrates the dependability of heavy timber framing under fire conditions.

At the conclusion of the test, the wood beam was

sawed through at a representative section, revealing a depth of char penetration of approximately ¾-inch on each side and 5/8-inch on the bottom. This is illustrated in Figure 10. The photo also shows that, as the char progressed, the loss of wood substance resulted in shrinkage of the individual laminations.

While penetration of char at the glue line was slightly greater, the deflection record demonstrates that the integrity of the casein adhesive bond was maintained during fire exposure.

Thus, after 30 minutes of fire exposure, during which temperatures in excess of 1500°F. were recorded, 75 per cent of the original wood section remained undamaged and the beam continued to support its full design load.

3. A glued-laminated heavy timber beam and a heavy structural steel beam, supporting equivalent applied loads, were exposed to identical fire conditions. The fire exposure conformed to the recognized standard used for evaluating the fire performance of structural materials.

Both framing members, exposed to the fire, were stock items without protective covering.

The wood beam had received no chemical fire retarding treatment.

The test results clearly demonstrate that the fire endurance of an unprotected and untreated heavy timber beam is substantially greater than that for a comparable unprotected steel beam. Under fire conditions, a structural steel framing member may be expected to fail through excessive deflection long before significant damage has occurred in a timber member.

Figure 9. Condition of the steel beam after the fire test exposure.

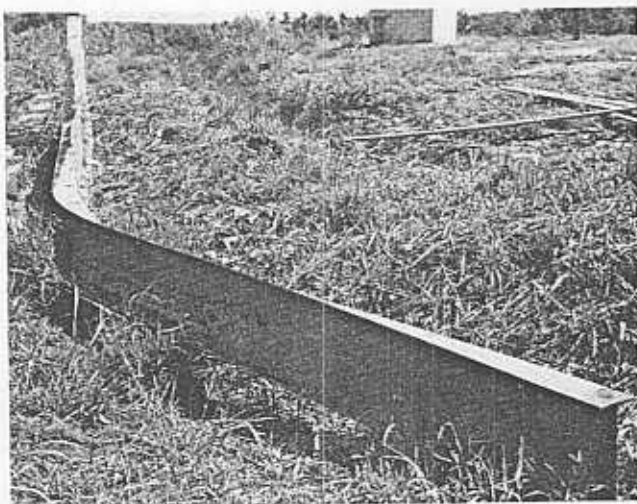
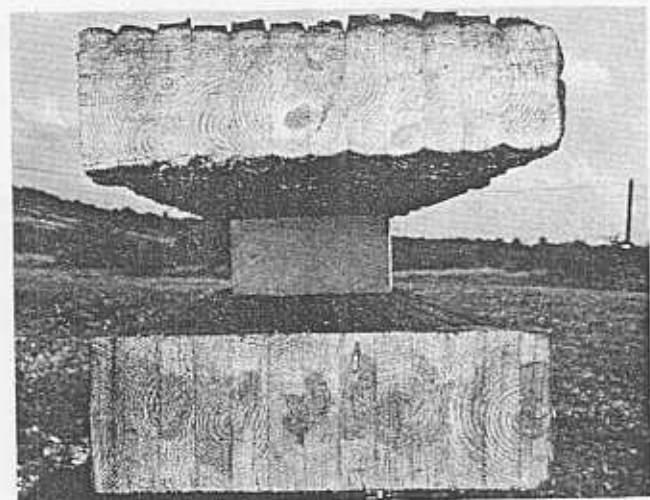


Figure 10. Condition of the wood beam after the fire test exposure.



---

**AMERICAN FOREST & PAPER ASSOCIATION  
AMERICAN WOOD COUNCIL  
1111 19th SREET, N.W., SUITE 800, WASHINGTON, D.C. 20036  
email: [awcinfo@afandpa.org](mailto:awcinfo@afandpa.org)  
website: [www.awc.org](http://www.awc.org)**

---