Integrity of Structural Steel After Exposure to Fire

R. H. R. TIDE

INTRODUCTION

Perceptions of fire vary depending on the circumstance to which the individual is exposed. Controlled fires are rarely given much thought in our daily experiences. Uncontrolled fires, with the specter of buildings collapsing, the implied damage, potential injury and loss of life have created a very negative image. A negative connotation often exists that anything exposed to fire and heated to a high temperature must be damaged, regardless of the appearance of the structural members.

Exposure to fire will subject structural steel to thermally induced environmental conditions that may alter its properties. Assessing these altered properties requires a combined knowledge of metallurgical and structural behavior as the fire raises the steel temperature and the steel later cools. Knowledge of steel properties and behavior developed from basic steel production, thermal cutting, thermal or mechanical straightening (or curving), heat-treating and welding provides the requisite information.

Fire represents a transfer of energy from a stable condition to a transient condition as combustion occurs; examples are the burning of warehouse contents, office furniture, books, filing cabinet contents, or other material. During this process, the steel temporarily absorbs a significant amount of thermal energy. Subsequently, the steel structure returns either to a stable or unstable condition after cooling to ambient temperatures. During this cycle, individual members may become badly bent or damaged without affecting the stability of the whole structure. It is possible to predict the range of temperatures that a particular steel member of a building experienced during a fire using current heat transfer theories. Damaged members are indicative of energy redistribution within the member itself and possibly the whole structure. Assessing overall structural stability can proceed after the condition of the individual members has been established.

This paper primarily addresses the structural integrity of individual members and not the structure as a whole. The effects of elevated temperature on high-strength bolts and welded connections will also be discussed because of their impact on individual member behavior. To analyze these effects, it is important to review the process and temperatures by which steel is manufactured. This review is followed by an examination of previous work on steel shapes and connections exposed to elevated temperatures. This knowledge is then applied to evaluate the structural behavior of individual members.

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Following the background discussion, means of evaluating fire-damaged steel will be presented. Various test procedures will be reviewed to examine their ability to determine metallurgical or structural degradation of the steel properties. These tests commonly include visual observations and measurements, surface hardness readings, residual stress measurements, metallographic sectioning and testing for chemical and physical properties.

BACKGROUND

Prior to evaluating the integrity of fire exposed structural steel, it is appropriate to examine some relevant background information. A brief explanation of steel fire testing and protection (fireproofing) is given by AISC (1969). The ASTM E119 fire test referenced in this document specifies time-temperature and loading conditions that must be satisfied. After test completion, the structural steel assembly must satisfy certain acceptance criteria. One such acceptance criterion for structural steel columns is:

> "Regard the test as successful if the transmission of heat through the protection during the period of fire exposure for which classification is desired does not raise the average (arithmetical) temperature of the steel at any one of the four levels above 1000°F (538°C), or does not raise the temperature above 1200°F (649°C) at any one of the measured points."

Similarly, for steel beam assemblies, a representative acceptance criterion is:

> "For steel beams the temperature of the steel shall not have exceeded 1300°F (704°C) at any location during the classification period nor shall the average temperature recorded by four thermocouples at any section have exceeded 1100°F (593°C) during this period."

Unfortunately, local or overall damage and deformation, which can occur in a fire, are not addressed in these criteria. The absence of a statement acknowledging a level of deformation, which does not impact on the structural capacity of the steel member, can imply that virtually any deformation results in steel not acceptable for future use. This interpretation oftentimes results in rejection of otherwise metallurgically and structurally sound steel.

Because of the economics involved in protection before a fire or salvaging steel after a fire, research and investigation programs continue, Thomas (1992). Selected fire test literature is referenced because relevant steel temperatures or conditions are presented. Similarly, early repair and salvage case studies were located, which form the current knowledge and experience database. Elevated temperature tests have been conducted on structural steel to determine behavior under prescribed fire time-temperature conditions, protection conditions, ventilation and distance from the center of the fire, Butcher (1966, 1967a, 1967b), Jeanes (1986), Law (1967) and Thomas (1992). Depending on the test variable, the steel temperature may or may not have exceeded 1200 degrees F. In this work, the visual appearance and metallurgical or structural properties were not directly addressed. The primary test variable was whether or not the steel members passed a prescribed time-temperature-deflection criterion. It is inferred, based on previous research and production knowledge, that no significant metallurgical changes occurred.

Steel industry firms experienced in steel production and fabrication procedures have routinely repaired or straightened fire damaged steel. Early examples referenced in the literature include those by Corbit (1950), Dill (1960) and Stitt (1964). Unfortunately very little technical information was reported to indicate time-temperature and metallurgical conditions. However, Wildt (1972) formalized these experiences, coupled with more current research and case studies, without providing the necessary background information.

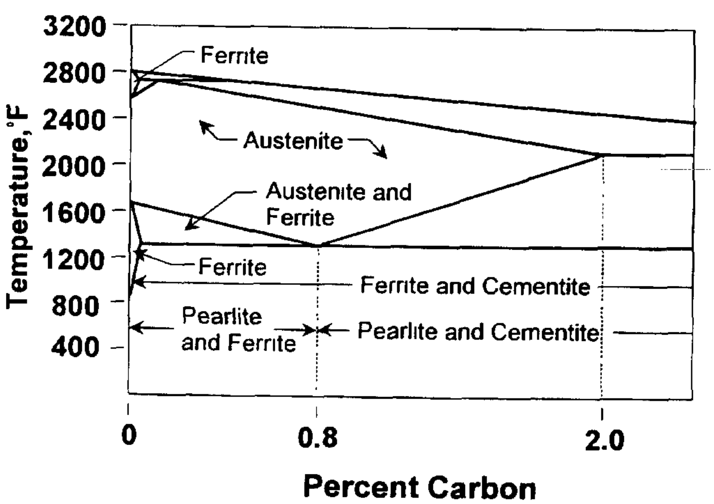

Fig. 1. Representative phase diagram for structural steel.

Based on the early fire research, the following points should be recognized when evaluating steel structures exposed to fire:

- The steel temperature is significantly reduced as the distance from the center of the fire increases.
- Ventilation of a fire compartment greatly reduces the temperature of steel at distances away from the center of the fire.
- The temperature of fire-protected steel in the vicinity of the fire is significantly less than unprotected steel at the same location.

REVIEW OF STEEL PRODUCTION AND PROPERTIES

Structural Shapes

Basic steel production temperatures and structural steel properties at elevated temperatures with respect to individual member behavior form the basis of evaluating fire-damaged steel. Generally, the steel ingot temperature prior to rolling ranges from 1900 degrees F to 2300 degrees F depending on ingot size, shape, chemistry and the size of the final product, USS (1985). For most hot rolled shapes, final rolling occurs when the steel is approximately 1600 degrees F or higher depending on mill procedures. For members other than structural shapes and plates, finished steel surfaces can be produced at temperatures as low as 1400 degrees F. Re-heating steel to elevated temperatures is a well known phenomenon, Dill (1960) and USS (1985). Stress relieving temperatures range from 1100–1200 degrees F, and annealing and normalizing temperatures are 1500–1600 degrees F.

The phase diagram, Neely (1979), shown in Figure 1 indicates that with normal structural steel chemical composition, rolling occurs when the grain structure is classified as austenite. Under normal conditions the austenite is transformed predominately into ferrite and pearlite as the steel cools to ambient temperatures. Depending on the chemical composition, material thickness and rolling pressures, the resulting structural steel shape has the ASTM specified range of yield strength (F_y), tensile strength (F_u) and modulus of elasticity (E). For all practical purposes, the latter property is a constant value at ambient temperatures for structural steel regardless of chemical composition and rolling processes.

Once rolling of the structural plate or shape is complete, the member is placed on a cooling bed. Residual stresses develop because of the differential cooling rate. Depending on subsequent cold working operations and fabrication procedures, an unknown magnitude and distribution of residual stresses occur. Residual stress development during steel manufacture has been addressed by Tide (1985, 1987, 1989), Hall (1969), SSRC (1988) and others, and will be addressed in a later section.

Heating of steel, such as during a fire, results in metallurgical changes that are predominantly temporary, although some may be permanent. Tests indicate that as the structural steel is heated to 1200 degrees F, the common structural properties F_y, F_u and E decrease as illustrated in Figures 2, 3 and 4, respectively. These curves represent ASTM A36 steel, but in a general sense can represent all common structural steels, USS (1981) and Lay (1982). The coefficient of thermal expansion is also temperature dependent, AISC (1989):

$\alpha = 6.5 \times 10^{-6}$ in./in. per degree F for temperatures less than 100 degrees

$= (6.1 + 0.0019T) \times 10^{-6}$ in./in. for temperatures between 100 and 1200 degrees F

where:

T = temperature, degrees F

Above 1200 degrees F, steel properties decrease so dramatically that they are of no structural interest.

Because of their relatively low carbon and other alloying content, structural steels usually regain close to 100 percent of their pre-heated properties provided the steel temperature does not exceed approximately 1330 degrees F, Avent (1992), Kirby (1986), Hineman (1983) and Smith (1980). This temperature coincides with a boundary condition shown in the phase diagram of Figure 1. Any temperature rise between 1330 degrees F and the minimum temperature for hot rolled structural shapes (1600 degrees F) has minimum impact on the structural properties once the steel has cooled to ambient temperatures. Restricting thermal heating to 1200 degrees F, AISC (1989), provides a safety factor against degradation of the metallurgical and physical properties. Both AASHTO (1992) and AREA (1989) also adopt the 1200 degree F temperature limit for heating steel during repair operations, with minor variations. Elevated temperature creep is an important consideration for steel supporting loads and subjected to elevated temperatures for extended time periods.

Heat-treated high strength alloy steels require special consideration after exposure to fire conditions. These steels are seldom used in building construction, but some bridges have been constructed with various proprietary products. Whenever encountered, the concepts presented in this paper must be modified accordingly.

Connections

High strength bolts warrant a separate mention because of their special manufacturing requirements. The increased strength of ASTM A325 and A490 bolts, above soft bolt strength, is obtained by combining chemical composition with a heat treating process. To minimize undesirable behavior, the hardness of the finished bolt is restricted to a fairly narrow range. As a result, the manufacturer carefully chooses the chemistry and heat process for the grade and size of ASTM A325 or A490 bolt. Because of relatively similar chemistry, the phase diagram shown in Figure 1 is still representative for high strength bolts.

In general, the bolt is heated to slightly above 1600 degrees F for a period of time to fully austenitize the grain structure. The bolt is then fully quenched to minimize any grain transformation as the steel cools. Subsequently, the bolt is tempered in the range of 800–1200 degrees F. The actual tempering temperature depends on the bolt chemistry and size so that the appropriate hardness range is achieved.

Experimental work by Wakiyama (1979) and post-fire examination of bolts removed from a building by Kirby (1991) and SCI (1991) indicate that, with one exception,

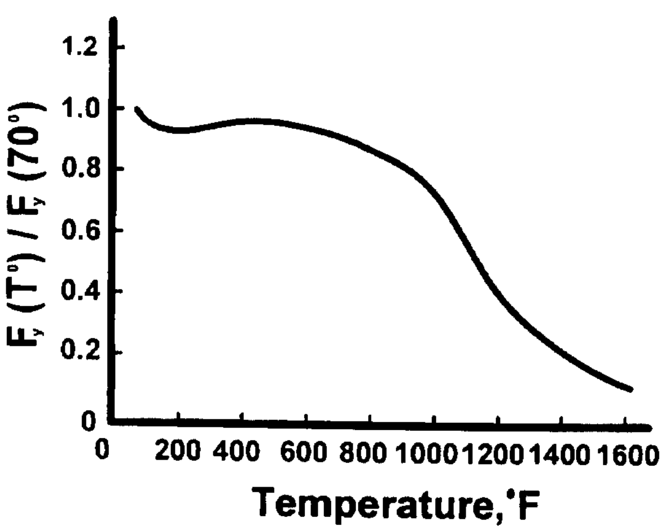

Fig. 2. Representative yield strength—temperature diagram for structural steel.

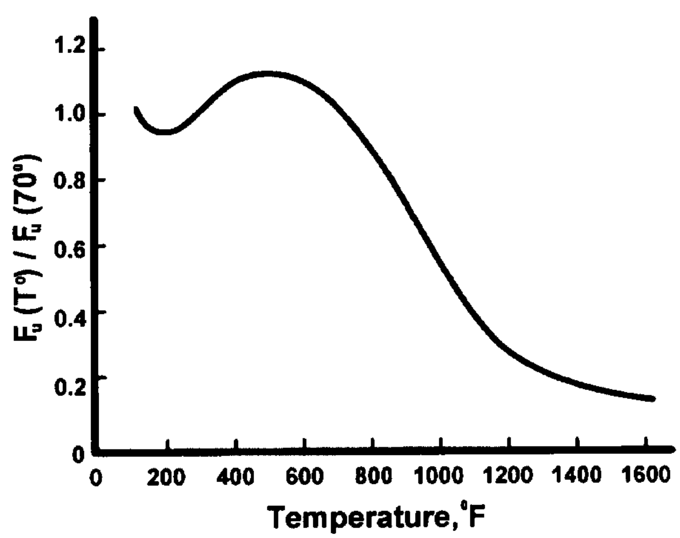

Fig. 3. Representative tensile strength — temperature diagram for structural steel.

exposure to fire does not alter high strength bolt properties. When high strength bolts are exposed to fire temperatures near the tempering value for 1–2 hours, relaxation of the pre-tensioning due to creep occurs. After the bolt cools to ambient temperatures, the original bolt strength is essentially regained, Kirby (1986).

Because the temperature range between tempering and phase transformation is narrow for high strength bolts, any exposure to fire above the tempering temperature increases the possibility that the bolts have reduced capacity. However, it is highly probable that bolts installed in steel members, which remain undeformed after a fire, have not been exposed to temperatures above the tempering value.

Weld metal exposed to elevated fire temperatures can be treated the same as the adjoining base metal when examining the metallurgical aspects. The welding process results in a residual stress magnitude nearly equal to the base metal yield strength. This concept is understood for those familiar with welding and the design specifications account for any localized yielding of the steel under service load conditions. The temperature increase resulting from a fire is comparable to post-weld heat treatment commonly used in the pressure vessel industry and addressed by the appropriate ASME codes. Chen (1985), Stout (1985) and Zhou (1985) have addressed this type of residual stress relief.

Wright (1990) examined the post-fire behavior of welds with tests of welded brackets removed from a bridge having 22 years of service. The temperature in the top and bottom flange, based on observed conditions, was predicted at 400 degrees F and 1000 degrees F, respectively. This testing revealed that the bottom flange bracket, which was exposed to a higher fire temperature, had a slightly improved fatigue life than the top flange bracket. The top flange brackets were close to the composite girder neutral axis and therefore subjected to a very low stress range over the bridge service life. Thus, it appears fire-induced stress relieving served to offset the higher stress range experienced by bottom flange brackets. Representative bridge girder data demonstrating the improved fatigue life for the bottom flange brackets is reproduced in Figure 5.

STEEL BEHAVIOR AT ELEVATED TEMPERATURES

Forces that develop in a steel member, as the temperature increases, are primarily dependent on the end restraint, and to a lesser degree on the intermediate bracing and supports. Steel expansion is temperature dependent and as the temperature of the steel increases, an unrestrained member will elongate according to:

$$\Delta L = \alpha L \Delta T$$

where:

L = original length, in.
ΔT = change in temperature, degrees F

A member fully restrained in the axial direction will develop axial stresses as the temperature increases according to:

$$f_a = E \alpha \Delta T$$

where:

E = modulus of elasticity, 29,000 ksi

To facilitate future discussion, values of α, f_a and ΔL are presented in Table 1 for beam lengths of 25 and 40 ft. These lengths were chosen because they represent the common length range encountered in building construction. Table 1

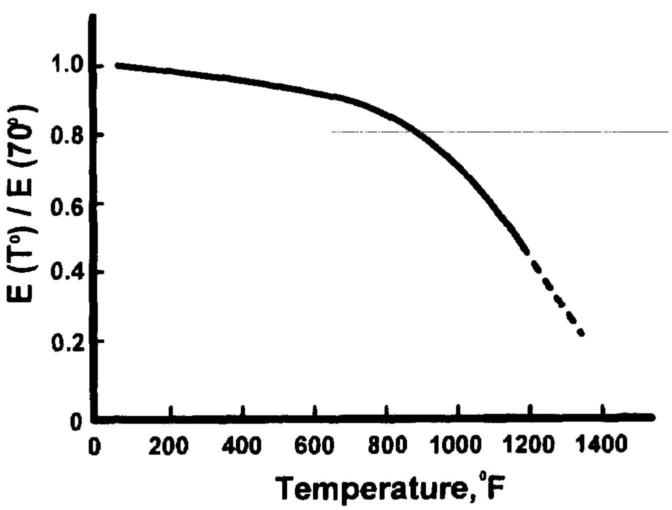

Fig. 4. Representative modulus of elasticity—temperature diagram for structural steel.

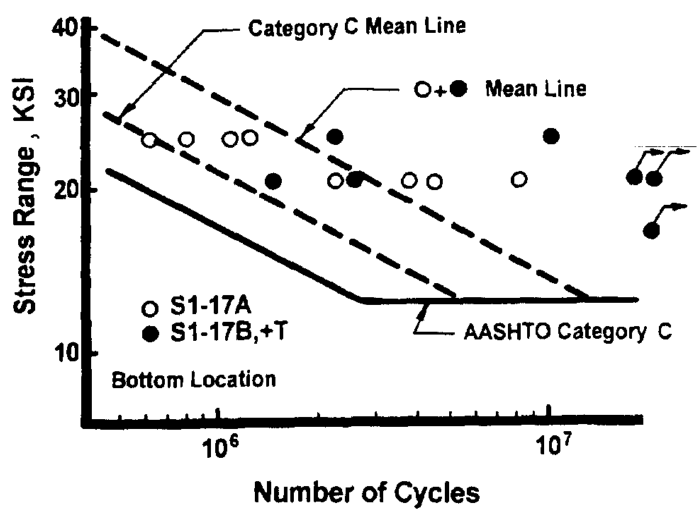

Fig. 5. Fatigue life of bridge brackets after fire exposure.

Table 1. Elevated Temperature Effects on Structural Steel					
Temp.[a] (°F)	**E ($\times 10^6$) (ksi)**	**Therm. Coef. ($\times 10^{-6}$-in./ in./°F)**	**Restrained Stress (ksi)**	**Elong. (25 ft.) (in.)**	**Elong. (40 ft) (in.)**
100	29	6.5	5.6	0.06	0.09
200	28	6.5	25	0.26	0.40
300	28	6.7	43[b]	0.46	0.72
400	27	6.9	62[b]	0.66	1.05
600	26	7.2	99[c]	1.08	1.72
800	24	7.6	—	1.52	2.43
1000	20	8.0	—	1.99	3.18
1200	12	8.4	—	2.48	3.97
1400	5	8.8	—	3.00	4.80

[a] Based on an ambient starting temperature of 70 degrees F, an average thermal coefficient was used for each increment. Elongation and stress based on unrestrained and restrained conditions, respectively.
[b] Approximately equal to or exceeds the yield strength of most structural steels.
[c] Exceeds the tensile strength of most structural steels.

indicates that either a member has significant room for unrestricted expansion or significant compressive forces will develop when restrained.

Evaluating fire-damaged steel in a building requires an assessment of the main steel members: columns and beams. Column behavior is well defined by the AISC (1986) formulae, Tide (1985):

$\lambda \leq 1.5$

$$F_{cr} = \phi EXP(-0.419\lambda^2)F_y$$

$\lambda > 1.5$

$$F_{cr} = \phi(0.877\lambda^{-2})F_y$$

where:

ϕ = resistance factor, 0.85
$\lambda = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}}$, slenderness
K = effective length factor
r = minimum radius of gyration

The buckling stress versus slenderness (Kl/r) predicted by these equations is plotted in Figure 6. Predicted buckling stresses represented by this curve, relative to a restrained condition (Table 1), suggest that for the practical slenderness range buckling would likely occur with a 300–400 degree F temperature change. However, full column or beam restraint is difficult to achieve in actual buildings, and some expansion will occur as the temperature increases. As a result, there is no direct means to predict member behavior in most structural applications as the fire temperature increases.

Local buckling of rolled or built-up steel members is given by SSRC (1988). The critical buckling stress F_{cr} is usually given in the following form:

$$F_{cr} = k\left[\frac{\pi^2 E}{12(1-\mu^2)(b/t)^2}\right]$$

where:

μ = Poisson's ratio, 0.3
b/t = width thickness ratio for the beam flange (one-half flange width), or the beam web
k = coefficient indicating the boundary conditions of the beam flange or web

Because most hot rolled or similar built-up shapes are compact, local buckling for nominally unrestrained straight

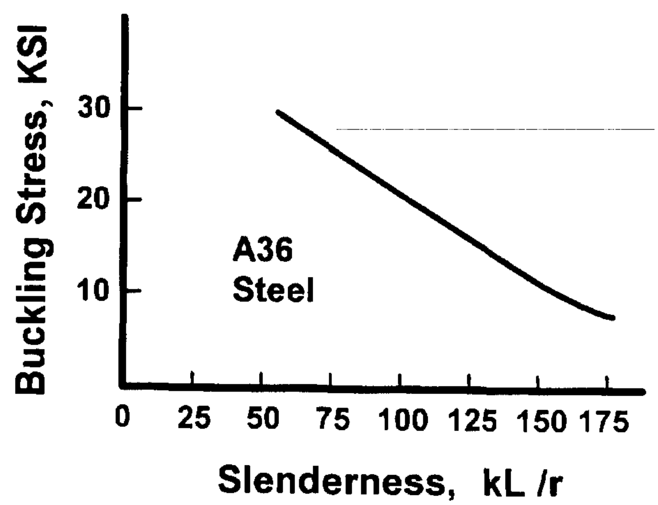

Fig. 6. Schematic prediction of column buckling stress.

beams or columns is not a consideration at temperatures below 1200 degrees F.

Minimal end restraint, out-of-straightness, or force eccentricity can cause severe local or overall buckling at temperatures above 600 degrees F. Buckling is likely to occur when the temperature is in the 1200–1400 degree F range because of the reduced F_y and E under these conditions. Experience gained while heat straightening or curving steel indicates that local buckling occurs suddenly without warning at temperatures above 1200 degrees F. The prescribed 1200 degree F limit provides a safety factor against local buckling and metallurgical changes, as mentioned previously. In addition, large in-plane beam deflections under dead load only are likely to occur at elevated temperatures. Flexural stress and deflection equations, using the reduced F_y and E, can be checked to verify this condition.

Large deflections due to creep are a consideration if the elevated temperature and load are sustained for a period of time. However, stress levels and fire duration in most building occupancies do not result in appreciable creep deflection due to the limited fire load and exposure time. Estimates of creep can be determined from published research data, ASTM (1955), Fields (1989, 1991), Finnie (1959), and USS (1970). Although if some creep occurs, its effect on an essentially straight member is unlikely to be significant or affect the performance of a refurbished building.

A review of fire test data reported by Saul (1956), Hapgood (1979), Smith (1980), Hineman (1983), SCI (1991) and Kirby (1991) confirms steel members that remain nearly straight after fire exposure are unaffected by the heat. The unchanged member geometry indicates the steel was not subjected to an elevated temperature that significantly affected the steel yield strength or modulus of elasticity. Furthermore, no consequential metallurgical change in the steel composition occurred.

Steel members having noticeable distortions are categorized separately in terms of their metallurgical and structural properties. Test data reported by Saul (1956), Dill (1960), Smith (1980), Kirby (1986), and Wright (1990) indicate that severely distorted steel may remain metallurgically unchanged because buckling and large deflections probably occurred at temperatures well below 1200 degrees F. Experience gained from heat straightening Saul (1956), Stitt (1964), Pattee (1969), Holt (1971), Stewart (1981), and Avent (1992) reinforces the following statement made by Dill (1960):

> "Steel which has been through a fire but which can be made dimensionally re-usable by straightening with the methods that are available may be continued in use with full expectance of performance in accordance with its specified mechanical properties."

Recent research, Avent (1992), Wright (1990), confirms that this statement is as true today as when it was first stated over 30 years ago and suggests that the criteria for evaluating fire exposed and damaged steel is a function of repairability, rather than inconsequential metallurgical changes.

For severely damaged structural steel members, where the deflections are excessive, metallurgical degradation is a moot point. It is usually more economical to replace the member than attempt salvage operations by heat straightening. But as noted above, the member can be heat straightened with an acceptable amount of metallurgical and physical property degradation. These members should be evaluated on a case by case basis. Preliminary guidelines have been prepared by Shanafelt (1984), which were advanced by Fields (1989, 1991), Avent (1992) and implemented by Wright (1990).

EVALUATING FIRE DAMAGED STEEL

Prior to making a site visit, the building's history and construction should be determined, which includes obtaining copies of the drawings and other pertinent documents. The building's age will provide ample indication that structural steel will be encountered, rather than cast or wrought iron. As indicated previously, knowing the specific ASTM steel designation is not critical because the behavior of fire exposed structural steel is designation independent. It should be recognized that it is common practice for a steel mill or fabricator to substitute higher strength (re-graded) steel members during construction to meet schedules, Tide (1987). Thus a building could have different steel grades mixed with the originally specified grade. Special consideration must be given when cast iron, wrought iron or heat-treated alloy steels are encountered and will not be discussed herein.

Building occupancy conditions and the fire history should be documented shortly after the fire. This information will provide an indication as to fire load and temperature exposure for each member. Steel protection means (fireproofing) and compartment ventilation conditions can affect the time-temperature history. For example, a long duration fire could result in minimum steel damage if the fire progressed through a well protected, ventilated and many compartmented building, where each compartment was exposed to fire for a short time. Theoretical studies by Iding (1977) that were calibrated with test results indicate that structural steel members in a building that had 2, 3 or more hour rated protection would not experience any meaningful temperature change. The AISI (1978) report indicates that column flange and web temperatures during a building fire probably did not exceed 500 degrees F. Fire tests on a building mock-up, Jeanes (1986), with various levels of protection confirm these results.

Many procedures are available to assess structural steel integrity after fire exposure, including visual observations, non-destructive testing and destructive testing (removing samples). The last method can incorporate chemical composition analysis, obtaining physical properties (F_y, F_u, ductility, toughness, etc.), residual stress determination and distribution, and metallographic observations. Each procedure will be assessed to examine its relative value in predicting structural steel conditions after fire exposure.

Visual Observations

In some cases, well documented, visual observations in the fire exposed building area can provide the most useful information concerning the predicted fire temperature and the resultant structural steel peak temperature. Fire temperature indicators can usually be found within the surrounding fire damaged material. Wood and paper ignite at 450 degrees F, and most plastics melt or burn between 180 to 300 degrees F. The effect of concrete exposed to fire is given by ASTM C856 (1995). Concrete paste begins to change color, initially changing to pink at approximately 550 degrees F and darkening to a deep red at 1100 degrees F. Surface spalls will begin to occur when concrete made with quartz aggregate is heated to approximately 1100 degrees F. The degree of spalling is dependent on rate of temperature rise, moisture content and maximum temperature for each type of aggregate. The steel surface itself can provide fire temperature information. Noticeable, tightly adhering mill scale indicates a steel temperature considerably below 1200 degrees F. Structural steel exposed to temperatures above 1200 degrees F will develop a coarse, eroded surface markedly different from the appearance produced by mill rolling, Dill (1960).

Although there are numerous types of coatings, markers and paints, which may be used on structural steel, they usually are not designed to withstand elevated temperatures. At temperatures above 600 degrees F they usually will have blistered, discolored or even flaked-off depending on the exposure to open flames. Any identifiable residue is an indication that the steel has not reached its critical temperature.

A temperature increase will cause an unrestrained member to increase in length or large forces will develop when restraint exists as previously illustrated in Table 1. Careful observations, measurements and analysis of displaced masonry infill walls, or similar elements will provide information as to probable thermal forces and peak temperatures.

The preceding items are important for establishing fire temperature and observing post-fire behavior. However, assessing the structural steel geometry after the fire is foremost in order to evaluate its condition. After fire exposure, it is convenient to categorize the members as follows:

- **Category 1:** *Straight members* that appear unaffected by the fire. This includes members that have slight deformations not easily detected by visual observations (within 4 or 5 times ASTM A6 rolling tolerances).
- **Category 2:** Members noticeably deformed but could be heat straightened if economically justified.
- **Category 3:** Members severely deformed that only under extreme circumstances would repair be given any consideration.

Once the condition of each individual member is determined, the safety of the whole structure can be established. A member inventory should be performed before an assessment of repair or replacement can begin. The biggest challenge often encountered with this evaluation is convincing the interested parties that the basic steel properties of Category 1 and 2 members were unaffected by the fire.

Camber and sweep of each fire exposed, structural steel member should be determined using appropriate measurement techniques (plumb bob, stringline, laser). A Category 1, 2 or 3 designation should then be assigned to each member. Most often, the Category 2 or 3 designation can be assigned without measuring because of severe local buckles or excessive deflections. Illustrative examples of member categorization are presented in Figures 7 to 9. In Figure 7,

Fig. 7. Considerable thermally induced beam twisting.

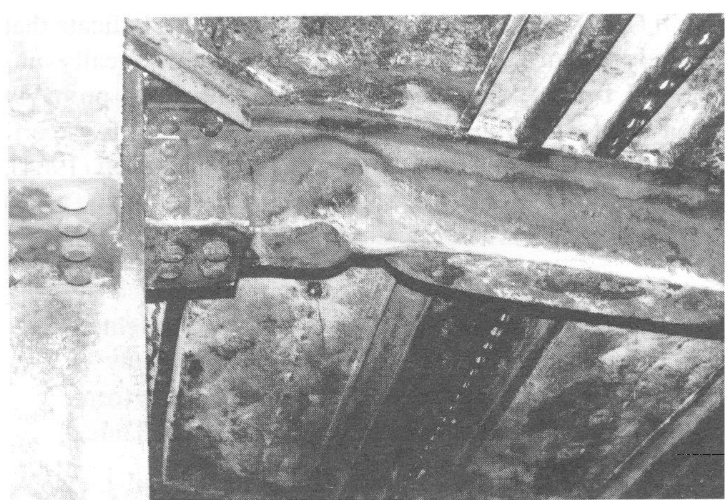

Fig. 8. Severe local buckle in beam.

the two twisted steel beams would be classified as Category 2. Members like this would usually be discarded for economical reasons even though heat straightening is possible. Similarly, the buckled beam shown in Figure 8 would probably be replaced for economical reasons. In comparison, the severe local buckling in the column shown in Figure 9 would normally be repaired under loaded conditions. Because of supported load above and the location, the cost to replace the column justifies in-place repair by either reinforcing plates, heat straightening, or both. However, for this particular case there was unacceptable vertical displacement, and therefore the column was replaced after jacking up the column shaft so that the floor grade could be reestablished.

Earlier studies demonstrate that Category 1 straight members require only minimal consideration, Avent (1992), Dill (1960), Kirby (1986), Smith (1980), and others. Metallurgical or structural degradation does not occur with a Category 1 appearance. For any significant metallurgical degradation to occur, temperatures would have to exceed 1330 degrees F. Prior to reaching this elevated temperature level, buckling or large deflections would certainly occur. Slightly deformed Category 1 members, with deformations greater than rolling tolerances, must be analyzed to determine the repair level. Depending on individual circumstances, the analysis will determine if these members can be accepted unconditionally, heat straightened, stabilized with supplemental braces, or reinforced with plates and shapes.

Category 2 members require additional attention because the decision to repair or replace is often a function of the nearby members' condition. A beam is easy to replace when compared to a column supporting several floors (Figure 9). If a Category 2 member is heat straightened, the change in metallurgical and structural properties will be inconsequential. Rehabilitation or replacement of Category 2 members is usually dependent on expediency, economics or overcoming the human psychological rejection of what appears to be damaged steel.

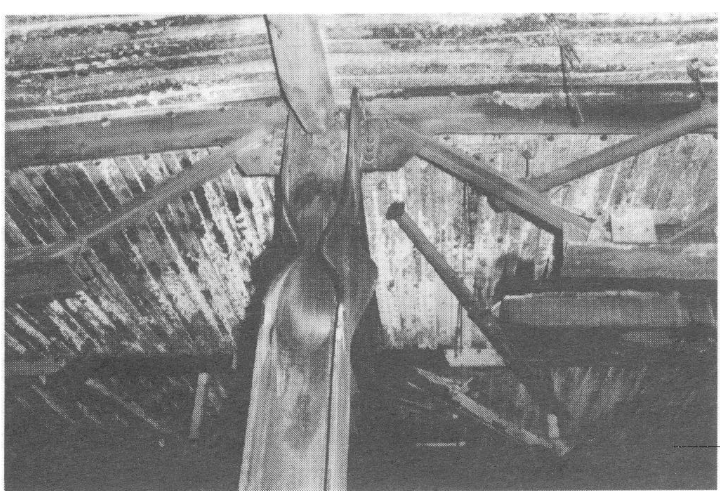

Fig. 9. Severe local buckle in column.

In most cases, Category 3 members are obvious and usually rejected without much consideration. Salvaging a Category 3 member would most likely occur in a very critical location where removal is inappropriate or impossible. Repair and reinforcement is then implemented as required.

Inspecting connections is imperative for beams that will be retained. Connection behavior is different than main member behavior when the temperature increases because of their relative compactness. The axial force developed by a restrained member will impose large forces on the end connections. Generally, the beam will buckle or deform to accommodate the axial force. Under these conditions, connection distress is easy to identify; when a Category 1 steel beam cools, if the connection has fractured, the steel beam will pull away from the adjacent member revealing the damage.

It is common to see fractured connections at the ends of buckled beams. As the buckled beam cools and shortens, the connection material, bolts or welds, will be torn apart similar to the connection of Figure 10. This type of bracket failure behavior occurs because the AISC Specifications contain a higher safety factor or reliability for bolts and welds than plain steel.

Bolts heated to the tempering temperature and held there for several hours will generally have a reduced pre-tension force once they return to ambient temperatures, Wakiyama

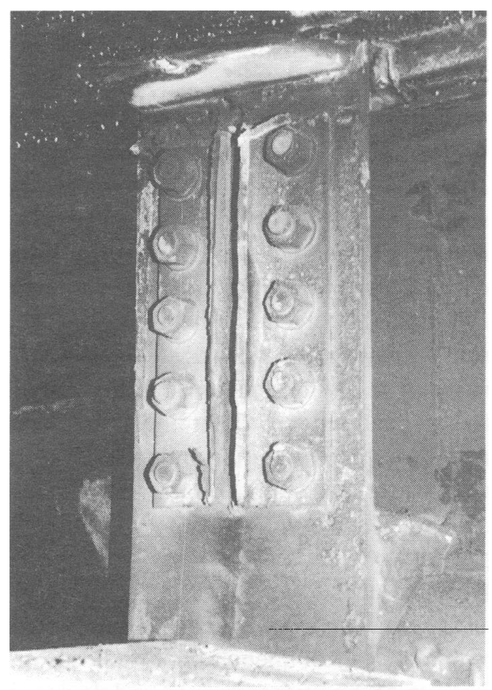

Fig. 10. Connection failure at end of buckled beam.

(1979). For Category 1 beams, achieving this critical temperature is unlikely because the member geometry suggests this temperature was not reached. Because of the variability of bolt installation procedures and quality control, it is impossible for a visual inspection to determine temperature effects on high strength bolts. Changes in pre-tensioning or metallurgical properties of high strength bolts can only be determined by non-destructive or destructive testing. Destructive bolt tests by Kirby (1991), provide guidance on testing procedures that can be considered.

Connections at the ends of Category 2 and 3 members to be salvaged are usually refurbished along with the beam; therefore evaluation of the connections is not warranted. If the beams are restored in place, then the bolts, brackets and welds should be given special attention.

Non-destructive Testing

There are several non-destructive test procedures applicable to examine fire-exposed structural steel. The most common technique, and the only one to be discussed in this paper, is surface hardness testing. This test can be utilized to determine the occurrence of a steel transformation due to the heating and accelerated cooling cycle. The various hardness measurement techniques provide an empirical determination of the approximate tensile strength.

When austenite steel (above 1600 degrees F) is cooled quickly to ambient temperatures (fully quenched), hardened steel results. Given a similar steel chemistry, fully quenched steel has an increased tensile strength compared to steel that cools under normal production conditions. Consequently, hardness readings on fire exposed steel supposedly indicate that the steel was raised to an elevated temperature and then cooled, thus causing some potential metallurgical degradation.

Three basic misconceptions exist with this philosophy. As explained previously, any structural steel heated to 1200 degrees F or greater will probably not remain straight under its own weight, thereby precluding a Category 1 classification. Therefore, a hardness test on a Category 1 member, and also Category 2, will have limited or no value, because the hardness would be unaffected.

The usual variations in structural steel chemistry are broad enough to result in a wide range of hardness readings when conditions are conducive to obtaining full quenching. Any increased hardness due to heating and quenching in a fire will only increase the tensile strength variability. Unless hardness readings were taken before the fire, it is unlikely that any meaningful prediction of tensile strength change can be made using this technique.

In a building environment, it is impossible to obtain conditions conducive to full quenching. Fire hoses or sprinklers apply a fraction of the water required and do not create a full quenching environment, AREA (1989) and Dill (1960). Water from fire hoses actually applies a temporary, non-symmetrical cooling pattern that can precipitate or contribute to buckling. Fire protection on the steel member also reduces the likelihood of any significant hardness change. Hardenability of steel is well established in regard to thermal cutting, Wilson (1987), and welding of steels, Stout (1987).

The yield strength is the primary variable when evaluating structural steel, not the tensile strength. No accurate correlation presently exists between tensile strength and yield strength. Various empirical correlations do exist, but they are usually applicable to specific products and the correlations are used for quality control during steel production. Because of the tensile strength and hardness measuring technique variability, any attempt to predict yield strength from hardness readings or a change thereof would be of questionable quality and almost meaningless.

Destructive Testing

Sample removal from the fire-damaged steel members can provide specific physical property and residual stress information. However, any evaluation must recognize that even the pre-fire conditions may have a limited range of accuracy. Steel chemistry most likely is not influenced by fire exposure. Therefore, temperature effects on the physical properties (F_y, F_u and ductility), residual stress distribution and grain structure will be reviewed.

Physical Properties

Structural steel exposed to temperatures above 1200 degrees F for a period of time generally experiences an inconsequential reduction of yield strength, tensile strength, and ductility, Dill (1960) and Kirby (1986). The reduction amount depends on steel chemistry, original mill rolling conditions, and actual duration and magnitude of temperature exposure. Studies by Avent (1992), Kirby (1986), Pattee (1969) and Smith (1980), suggest that Category 1 and 2 steel will have inconsequential changes in F_y, F_u and ductility.

When sampling and testing for yield strength, it should be recognized that yield strength varies within the member cross-section and along its length. The AISC (1986) Specification has long recognized that localized yield strength can fall below the specified minimum. Yield strength of the flange steel is usually less than the web steel yield strength, where the sampling coupons are obtained. Similarly, the coupon test speed specified by ASTM usually exceeds the rate at which load is applied to buildings, and can influence yield strength. Galambos (1978) documented this variation and the data suggests it is possible to randomly obtain steel coupon strengths 13 percent less than the specified value. The yield strength variation could possibly be larger depending on data interpretation, sampling location and location within the ingot from where the coupon was obtained, Tall (1969). Steels produced from a continuous casting near-shape process may have less variation.

Yield strength variations theoretically result in a slight safety factor increase or reduction, which has been known for years. Tests, ASCE (1971) and Lehigh (1965), have demonstrated that local deviation of yield strength below the specified minimum would not effect the member's overall capacity. Therefore, post-fire measurement of yield strength must recognize that random sampling could, within the stated percent limit, indicate a value below the specified minimum. This strength variation has no impact on the structural performance or safety.

Charpy V-notch (CVN) testing of removed samples to determine toughness should only be considered if fatigue loading or a brittle fracture potential exists. Based on existing research, Shanafelt (1984) and Wright (1990), there is no reason to suspect normally acceptable steel would have a reduced toughness due to fire exposure.

Residual Stresses

Residual stress evaluation is performed by removing instrumented areas or coupon samples from the fire exposed steel member. Prior to coupon removal, either mechanical or electrical strain gages are attached and calibrated with an initial reading taken. The coupon is then carefully removed from the member and the strain is measured until the recorded strain data stabilizes. The difference between initial and final readings indicates the internal member strain at the gage location. Placing gages at carefully chosen locations permits a reasonable prediction of the resultant stress distribution in the member.

Converting the strain distribution into stress components and predicting the stress portion attributed to the fire requires information that is not readily available or impossible to obtain. Furthermore, the results may be heavily influenced by the physical condition of nearby steel members. For example, severely buckled steel beams in one building bay could exert large tensile forces on straight beams in an adjacent bay. The force distribution in the straight beams could approach the steel yield strength. Once the severely buckled steel beams are removed, the apparent residual stresses in the straight beams would be significantly reduced or disappear altogether.

This is one example of the issue's complexity. To make a meaningful assessment of fire induced residual stresses, all contributing factors must be determined. One main factor is the residual stress that develops during the original mill rolling, as described by SSRC (1988), Tide (1985, 1989) and Tall (1969). The stress magnitude can approach one-half the yield strength in tension or compression, depending on the mill or the shape's size. Welded, built-up shapes have residual stresses of equal or greater magnitude, depending on plate origin and welding sequence. Fabrication and erection procedures are factors that can also influence the residual stress distribution. Routine heat straightening, cambering, fit-up and shoring means can introduce or affect the calculated force distribution as predicted from the instrumented coupons.

Although residual stresses influence the steel member capacity, Tide (1985, 1987) and AISC (1986), plastic design research summarized by ASCE (1971) and Lehigh (1965) indicates design specifications accurately predict steel member behavior. The combination of externally applied forces and internal residual stresses are accommodated by redistribution when plastic hinges form. Residual stresses will increase deflections above those theoretically predicted for residual stress free members. The deflections can become significant as the applied load approaches the member's ultimate capacity. However, the deviation from theoretical behavior is not significant at or below service loads, and usually cannot be detected.

The steel must be heated above the lower phase transformation temperature of 1330 degrees F before any temperature induced residual stress increase can occur. At lower temperatures, stress relieving may occur, especially if the steel is protected. Considering all factors involved, Category 1 and 2 members are unlikely to have any fire induced residual stress increase that would affect the behavior and safety of a structure. Restraint induced forces are more likely to cause yielding resulting in a change and re-arrangement of residual stresses. For the usual loading conditions associated with steel-framed buildings, the effect of residual stresses at service loads is innocuous. Shakedown theory and testing, Eyre (1970 a, b), indicates after a few cycles of loading that causes yielding, the effect of the residual stresses is essentially eliminated. The deflection behavior of the structure during any subsequent loading is based on elastic behavior.

Metallography

Steel coupon samples are removed and the surface is polished to a mirror-like finish. The surface is then etched with a weak acid solution, which exposes the steel grain structure. A grain structure that differs from the usual mill rolled grain configuration would suggest a metallurgical or structural change in the steel.

Structural steel's grain structure is usually flat or plate-like due to the rolling process, Neely (1979) and USS (1985). At a temperature around 1200 degrees F, the basic steel carbide constituents undergo a transformation known as "spheroidization of pearlite," Kirby (1985) and USS (1985). The quantity of spheroidal carbides depends on the temperature level above 1200 degrees F and the length of time the steel is held at this elevated temperature. Some spheroidal carbides may develop during initial steel production because the final rolling temperatures of most structural steel range from 1600 to 1800 degrees F, or higher. Pense (1988) indicates that this was observed while conducting other research, Chen (1985) and Zhou (1985). Any presence of spheroidal carbides could suggest a reduction of the yield strength because of the change in grain structure from that produced during rolling. However, research indicates that any reduction in strength is inconsequential, Avent (1992) and Kirby (1986). Hapgood (1979) concluded that the absence of spheroidal carbides indicated that the steel was not subjected to unusually high temperatures.

Consequently, the formation of spheroidal carbides in a Category 1 or 2 member is highly unlikely during a fire and their development may have also occurred during production. The elevated temperatures required for their formation during a fire would cause large member deformations, which are inconsistent with the above member classifications.

SUMMARY AND CONCLUSIONS

This paper is a synthesis of research and experience dealing with many different aspects of structural steel production, fabrication, fire exposure and testing. Pertinent information was obtained from laboratory testing of structural steel performed while attempting to improve protection criteria. Other documented tests were related to stress relieving investigations concerning pressure vessels, theoretical studies and actual cases of heat straightening twisted steel members exposed to elevated temperature during a fire. The referenced material, as well as experience, was accumulated over many years. Over this same period, numerous fire investigations were conducted or visits were made to fire damaged structures.

A first impression upon arriving at a fire scene is usually very negative because of the immense destruction and adverse environmental conditions. Field assessment of fire damaged steel members requires a systematic approach to determine their condition. Based on practical experience, a member inventory should be performed and each member should be classified. A classification system consisting of the following three categories has been suggested: Category 1—essentially straight members; Category 2—noticeably deformed, but repairable members; and Category 3—severely deformed members that generally are uneconomical to repair. Visual observation and measuring the member geometry provide the most practical means to classify and assess the potential for damage in a fire-exposed member. The repair or replacement of an individual member is dependent on classification, location in structure, and economics.

Research indicates that it is highly unlikely a Category 1 or 2 member was ever subjected to an elevated temperature at or above 1200 degrees F for any length of time so that consequential metallurgical changes could occur in the steel. Furthermore, unless a building has an unusually heavy fire load, 2 or 3 hour rated protection will prevent the steel temperature in main structural members from being elevated to a critical level precluding any abnormal deformations. Severe deformation would occur prior to significant metallurgical change in a structural member, thus resulting in a Category 3 classification.

Non-destructive and destructive testing provides interesting information on the steel's post-fire pedigree. However, with respect to changes in the physical properties or internal forces, the information obtained is often inconclusive and contradictory because exact pre-fire conditions and data at the same locations cannot be obtained. Furthermore, any residual stress measurements obtained prior to removal of nearby severely damaged (Category 3) members are likely to be misleading. Regardless, any residual stress conditions that are encountered in Category 1 and 2 members (essentially straight) can be accommodated by the effected structural members and frames according to plastic design and shakedown theorems, as proven by testing. Experience indicates that it is prudent to initially obtain a small sample of destructive test specimens from Category 1, 2 and 3 fire zone steel members and control samples from areas outside the fire zone. The data obtained from this testing will probably show an insignificant change in the steel's physical properties and therefore indicate that additional destructive sampling is not justified.

In conclusion, it can be simply stated: "If it is still straight after exposure to fire—the steel is OK". A similar statement was made over 30 years ago, and is still applicable to this day. With this statement in mind and the points raised in this paper, assessing the structural capacity or integrity of a fire exposed steel structure or steel member can be adequately determined.

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