Restrained Vs. Unrestrained Fire Ratings: A Practical Approach

Many structures have reserve strength capacity which can be utilized for resisting loads resulting from extraordinary events

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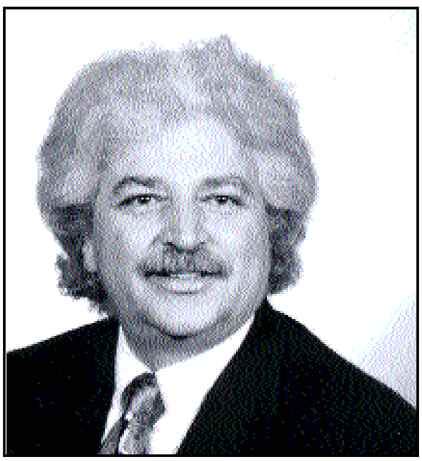

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In the past, buildings framed with fabricated structural steel were mostly considered restrained construction for the purpose of establishing structural fire resistance ratings and determining fire proofing requirements. This was based, mainly, on interpretive information in Appendix X3 of ASTM E119 (ASTM-E119-88, 1994).

However, recently there has been a trend to classify many such structures as unrestrained. The Uniform Building Code, UBC, (ICBO, 1994) places the onus of proving that a structure is restrained (all structures, not only steel) on the Structural Engineer of Record. The BOCA National Building Code (BOCA, 1996) issued an interpretation, in 1993, stating that the support conditions in actual buildings must be considered when applying restrained/unrestrained ratings. In 1995, a significant change was approved by the Standard Building Code Congress International to specifically include Appendix X.3 of ASTM E119 in the 1997 edition of Standard Building Code, SBC, (SBCCI, 1994). The available information as to how to determine if a steel structure (or any structure) is restrained or unrestrained is confusing at best, and the additional cost of fire proofing a steel structure that is considered unrestrained can be substantial. It is the authors' opinion that most welded, bolted or riveted steel frame construction can be considered restrained, and this article presents a practical procedure to evaluate this for specific cases.

In the past few years, changes in the Underwriters Laboratories, Inc., (UL) Fire Resistance Directory , ULFRD, (UL, 1996) resulted in certain fire tested floor or beam assemblies, which used to be classified as restrained for fire resistive purposes, to now be interpreted as being unrestrained. The additional cost for fireproofing these structures due to the perceived change in classification can be substantial depending on the size and type of structure. In the past UL incorporated Appendix X.3 of ASTM E119 (also Appendix C of UL 263) in the ULFRD resulting in most steel structures being considered restrained. The following is the pertinent excerpt from table X3.1 of ASTM E119 which states that structures that meet the following description can be considered restrained:

"....II.Steel Framing:
(1) Steel beams, welded, riveted or bolted to the framing members
(2) All types of cast-in-place floor and roof system (such as beam and slabs ....) where the floor or

roof system is secured to the framing members...."

Beginning in 1992 ULFRD included a commentary in the introduction quantifying the stiffness provided by the UL test chamber. The stiffness in two directions is given as EI/L equal to 700,000 kip-inches for the 17' and 850,000 kip-inches for the 14' span. It has been wrongly interpreted by some to imply that this is the minimum stiffness required to produce restrained construction. Further, even if an attempt is made to match the stated stiffness, is it sufficient to simply provide the same EI/L? The actual properties or conditions in the field (continuity, composite beam/slab design, simple beam connections, etc.) are what produce restraint. The reasons for abandoning the ASTM guidelines for determining whether a structure is restrained or unrestrained are not clear to the authors. Certainly, considerably more money is being spent on fire proofing due to the new interpretations. However, in some cases buildings that would otherwise have been designed in steel are designed in alternative materials to avoid dealing with this issue.

To exacerbate the problem, fire marshals and building officials interpret the codes differently resulting in different amounts of fire proofing for the same members depending on the location of the structure, making it more difficult for design professionals to determine how this issue will be interpreted. In some cases, by the time an interpretation is obtained the structure is already designed and under construction. To avoid confrontations at this late state some design professionals classify the steel structure as unrestrained for fireproofing purposes.

RESTRAINED VS. UNRESTRAINED

This article discusses the effects of fire on composite steel structures. The concepts defined, developed and discussed can, however, be used for other types of structures. It is assumed that the structure is composed of simply supported beams and girders. Consistent with fire engineering and fire testing assumptions, the potential fire is applied from the bottom of the assembly. This results in the highest temperatures in the bottom flange of the steel beam or girder. The top flange which is connected to the slab and the slab itself remain at relatively lower temperatures. Since the yield strength of steel (and most other materials for that matter) reduces at elevated temperatures, the first section at which the beam yields is the point of maximum positive bending moment.

In case of a beam with end restraint, the formation of the plastic hinge at the point of maximum positive moment does not lead to a failure because the moment is redistributed to the ends at which there is available unutilized capacity. However, in case of an unrestrained beam, the first plastic hinge leads to the ultimate failure. Therefore, restrained structures can sustain higher temperatures than unrestrained structures without a collapse.

ASTM E119 recognizes the positive effects of restraint by allowing more liberal failure criteria for restrained assemblies than for unrestrained ones. The main difference in the acceptance criteria for the two types of assemblies is the following:

> *For unrestrained structures the test failure is defined when either the average temperature in the steel beam has reached 1100 F or the maximum temperature at any point in the steel beam has reached 1300 F. The time at which this occurs is established as the hourly rating of the assembly.*
>
> *For restrained structures, on the other hand, the same temperatures (average temperature in the steel beam of 1100 F or maximum temperature at any point in the steel beam of 1300 F) are allowed at half the rated time or a minimum of one hour. The steel is then allowed to reach temperatures beyond the above as long as the ultimate load capacity is not exceeded. The time at which the ultimate load capacity is reached (or twice the time at which the maximum/average temperature limits were reached, whichever is lower) then becomes the hourly rating for the restrained assembly.*

It should be recognized that thermal restraint is not necessarily the same as structural restraint. Thermal restraint can be in the form of "thrust restraint" or "rotational restraint."

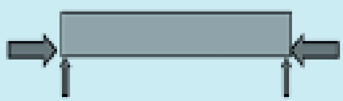


Figure 1: Thrust Restraint

Thrust restraint (Figure 1) increases the ultimate capacity by compressing (pre-stressing) the bottom flange. Although this is beneficial it is difficult to calculate the equilibrium thrust. At one extreme, if the bottom flange is totally restrained, a small rise in temperature will cause the bottom flange to buckle. At the other extreme if there is no restraint thrust forces cannot develop.

Rotational Restraint (Figure 2), on the other hand, produces negative end moments which also reduce the positive moment at mid-span. The negative end moments can be resisted either by reinforcing in the slab (which remains cooler) or by the simple beam connections and the capacity of the steel section itself for negative moment at elevated temperatures.

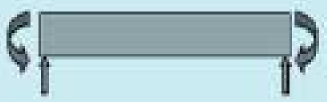


Figure 2: Rotational Restraint

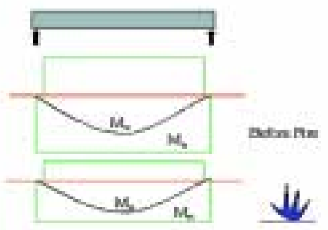


Figure 3: No restraint

Figure 3 shows the required flexural strength (M_u) and nominal flexural strength (M_n) for an unrestrained beam before and after fire. Notice that before fire there exists some negative nominal flexural strength (possibly less than the positive due to longer unbraced flange lengths) in the beam, but it drops to zero at the ends because of the absence of connection capacity. Also, notice that both M_u and M_n have reduced after fire. M_u reduces due to reduced load factors and M_n due to lower capacity (see discussion in the following section).

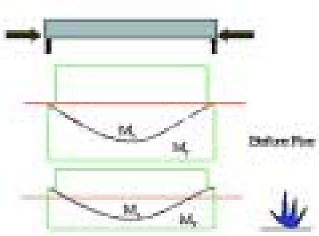


Figure 4: Thrust Restraint

Thrust restraint (Figure 4) results in shifting the moment diagram by imposing negative moments equal to the equilibrium thrust times the eccentricity from the neutral axis. Notice the step in the negative nominal flexural strength.

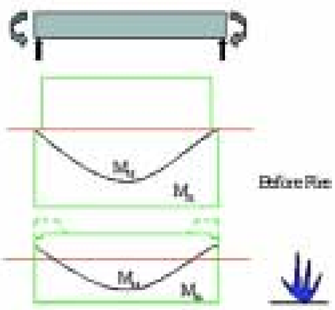


Figure 5: Roational Restraint

Rotational restraint (Figure 5) results in shifting the moment diagram by imposing negative moments equal to the flexural strength of the restraint (connection capacity or composite action). In the case of composite action, notice the additional negative nominal flexural strength resulting from the existence of reinforcing in the slab.

Analysis And Design At Elevated Temperatures

Load Combination and Resistance Factors for Fire Exposure. ASCE 7-95 (ASCE 7-95, 1996) includes a section (section 2.5) on load combinations for extraordinary events, such as fire, explosions and vehicular impact. This load combination (Equation 1; Equation C2.5.3 in ASCE 7-95) recognizes the small probability of such occurrences by utilizing load factors which are lower than for normal load combinations. It is the authors' opinion that no resistance factors need to be used on the resistance side of the equation. Additionally, the structure only needs to withstand the fire without failure and, thus, no serviceability criteria are applicable.

$$1.2D + A_k + (0.5L \text{ or } 0.2S) \qquad \textbf{(1)}$$

where: D = Dead Load
L = Live load
S = Snow Load
A_k = Load effect resulting from extraordinary event

Steel Temperature Time-History. The first step in computing the nominal flexural strength is estimating the temperatures at various locations in the beam. The standard ASTM E119 fire test requires monitoring of temperatures at specified locations along the length and depth of the beam. The readings from these thermocouples, over time, then become the temperature time-history and are part of the record of the fire test. UL and other fire testing laboratories provide this information to the sponsors of each tested assembly. UL will not release this information to other parties, unless the sponsors approve. Alternatively, the information can be obtained directly from the sponsors (such as fireproofing manufacturers, AISI, etc.).

Analytical methods, utilizing principles of thermodynamics, also exist for calculating this time history given a temperature time-history input and the thermal properties of the materials involved. FIRES-T3 (Bresler, B., et. al, 1977) is a public domain finite element program that accomplishes that. Its use, however, is cumbersome and requires knowledge of thermodynamics beyond the level that a structural engineer usually possesses. The authors are currently involved in a research effort to gather available temperature time-history information and/or augment it with FIRES-T3 modeling.

At any point in time, the typical temperature variation within the composite section is shown in Figure 6, where T_C, T_{tf}, T_w, T_{bf} are the resulting temperatures in the concrete slab, top flange, web and bottom flange respectively, at that particular time.

Once the temperatures of the individual components (slab, top flange, web and bottom flange) are obtained, the yield strength at that temperature of each indi-

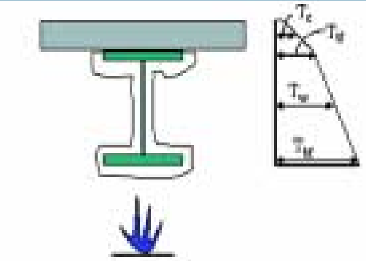


Figure 6: Typical Temperature Variations

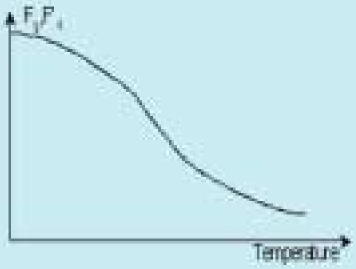


Figure 7: Yield strength/ temperature relation

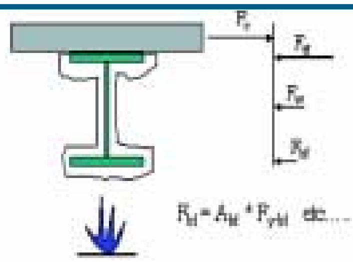


Figure 8: Forces At Mid-Span

vidual component can be calculated from published yield strength versus temperature relationships (Boring, D. F., et. al., 1981; CRSI, 1980) such as the one shown in Figure 7.

Positive Nominal Flexural Strength at Elevated Temperatures. Figure 8 shows the resulting forces in the individual components, at a particular point in time, obtained by multiplying the yield strength at the components temperature, at that time, by the component area.

The following steps outline the procedure followed for evaluating the positive moment capacity:

- It is convenient to start with an assumption that the concrete slab contains the compression zone and the steel beam provides the tensile resistance. This assumption is generally valid because the temperature of the top surface of the concrete slab does not increase appreciably (there is actually another failure criterion of E119 that keeps the temperature at the top of the concrete slab around 300 F) during the exposure to the fire.
- It is also assumed that the section will develop its full plastic capacity. That is, the steel will be fully yielded in tension and the concrete in the compression will be at a strain of 0.003. This assumption is also valid because the fire loading is a limit state loading case and no serviceability requirements need to be satisfied under fire.
- Using these assumptions and the yield strength of the bottom flange, web and the top flange it is possible to compute the total capacity in flexural tension, F_T.

$$F_T = F_{tf} + F_w + F_{bf} \qquad (2)$$

where F_{tf}, F_w, F_{bf} are the yield capacities of the top flange, web and the bottom flange respectively at their corresponding temperatures T_{tf}, T_w and T_{bf}

- Compute the depth of the equivalent rectangular compression block, a, using Equation 3.

$$a = \frac{F_T}{0.85 f'_c b_f} \qquad (3)$$

where f'_c is the compressive strength of concrete and b_f is the effective flange width. If a is less than the total slab depth, then the first assumption above is valid. If not, then a procedure similar to the one used for concrete T-beams when the neutral axis falls in the web of the T can be used.

- The nominal flexural strength, M_n, is computed by summing the moments of F_C, F_{tf}, F_w, F_{bf} about the neutral axis. This can be easily accomplished by realizing that each of the tensile forces (F_{tf}, F_w, F_{bf}) forms a couple with a portion of the compressive force (F_C). The component contributed to the nominal flexural capacity by each of the tensile forces is thus the force multiplied by the lever arm, between the tensile component and F_C.

Negative Nominal Flexural Strength at Elevated Temperatures—Restrained Assemblies

Rotational Restraint Provided by Connections: All connections provide some degree of restraint. Even the simplest double angle shear connections provide in the order of 20 kip-feet ultimate moment capacity. It is generally assumed that the connections are not subjected to elevated temperatures. This is a valid assumption because they are connected to other beams and cooler members and is consistent with the way fire tests are conducted. The beneficial effect can be directly taken into account by reducing the required positive flexural strength as shown in Figure 5. However, the steel section just beyond the connection might have less capacity for negative moments at the elevated temperatures and must also be checked. The resulting internal forces are shown in Figure 9. Use the methodology in the following section to calculate the nominal flexural strength.

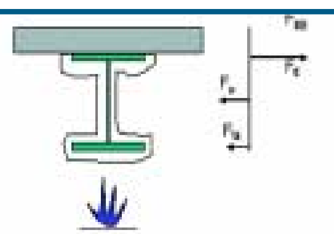


Figure 9: Forces at beam end

Rotational Restraint Provided by Composite Action (Slab Reinforcement): If the slab has reinforcement parallel to the axis of the beam, the reinforcement will provide the flexural tension and the connection will provide the flexural compression component. Since the connection is generally not subjected to very high temperatures, the critical condition for computing moments is in the beam at a small distance from the connection. At this point, part of the steel beam section provides the compressive force and the slab reinforcement possibly, in combination with the top flange of the steel beam provides the tensile force. The procedure for computing the negative moment capacity is outlined below.

It is convenient to start with an assumption that the neutral axis lies within the web of the steel section. Therefore, the reinforcing steel and the top flange provide the tensile capacity and the web and the bottom flange provide the compression capacity.

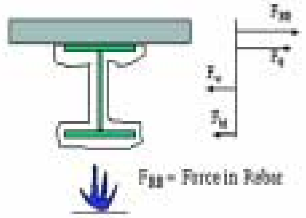


Figure 10: Rotational restraint (composite action)

- It is also assumed that the section will develop its full plastic capacity. That is, the steel will be fully yielded in tension and compression. This assumption is also valid because the fire loading is an ultimate loading case and no serviceability requirements need to be satisfied under fire.
- Using the assumptions and the yield strength of the bottom flange, web and the top flange it is possible to compute the total capacity in flexural tension, F_T.

$$F_T = F_{RB} + F_{tf} \qquad \textbf{(4)}$$

Where F_{RB} is the tensile yield force of the reinforcing steel. It is assumed that the reinforcing steel is at low temperatures and capable of developing close to its full yield strength. F_{tf} is the yield force capacity of the top flange at its temperatures T_{tf}.

- Using the assumptions and the yield strength of the bottom flange and the web, it is possible to compute the total capacity in flexural compression, F_C.

$$F_C = F_w + F_{bf} \qquad \textbf{(5)}$$

Where F_w and F_{bf} are the compressive capacities of the web and the bottom flange respectively at their corresponding temperatures T_w and T_{bf}.

The steps above are repeated using different locations of the neutral axis until

$$F_T = F_C \qquad \textbf{(6)}$$

- The moment capacity is computed by summing the moments of F_{RB}, F_{tf}, F_w, F_{bf} about the neutral axis.

Thrust Restraint. Calculating the equilibrium thrust is a complex endeavor requiring consideration of the surrounding structure. A simplified procedure for calculating the equilibrium thrust is currently being developed by the authors.

Recommended Procedure

The reason a structural assembly needs to be described as restrained or unrestrained is to determine the amount of fireproofing required to satisfy a pretested assembly. The question that should really be asked is: "How much fireproofing is required to achieve a certain hourly rating"?, or, asked a different way: "For my structure can I use the fireproofing thickness prescribed for restrained structural assemblies in the ULFRD?" The following recommended procedure answers the latter question in a systematic way.

Calculate the Required Flexural Strength. The effect of fire is taken into account by the reduction of nominal flexural strength at elevated temperatures. The effect of fire on the load side of the equation can be neglected, since this is a first order analysis. Equation 1 can be rewritten as follows:

$$1.2D + (0.5L \text{ or } 0.2S) \qquad \textbf{(7)}$$

Based on the loads from Equation 7 calculate the total static (simply supported beam) required flexural strength (M_u)

Calculate the Nominal Flexural Strength.

- Obtain Temperature Time-History: For the particular assembly or beam rating utilized in the design (e.g. UL-D916, UL-D925, etc.) obtain the temperature time-history data for the "restrained" test. If this data is not available for the particular assembly, generalized or similar data may be utilized.
- Find the Average Temperature in Each Component: Review the test data and extract the temperatures for the top flange, web and bottom flange at mid-span of the beam.
- Find the Yield Strength for Each Component:Based on temperature/yield strength relationships (such as Figure 7) find the yield strength of each component at the elevated temperatures.
- Calculate the Nominal Flexural Strength: Calculate the nominal flexural strength (M_n) as described above.

Compare Total Static Required Flexural Strength to the Nominal Flexural Strength. If the nominal flexural strength (M_n) is higher than the total required static flexural strength (M_u) then the beam size is sufficient at these elevated temperatures and no further calculations are required.

If Needed Utilize Connection Rotational Restraint. If the nominal flexural strength (M_n)

computed above is less than the total required static flexural strength (M_u) then utilize the connection rotational restraint and check both the connection capacity and the capacity of the steel section adjacent to the connection in accordance with section 4.d.i. Add the average of the negative nominal flexural strengths at the two ends of the steel member to the positive nominal flexural strength to obtain the total static nominal flexural strength (M_n). If M_n is greater than M_u then the beam size is sufficient at these elevated temperatures and no further calculations are required.

If Needed Utilize Composite Action Rotational Restraint (Reinforcing in Slab). If the total static nominal flexural strength (M_n) computed above is less than the required total static flexural strength (M_u) then utilize the composite action rotational restraint and check the steel section. Add the average of the negative nominal flexural strengths at the two ends of the steel member to the positive nominal flexural strength to obtain the total static nominal flexural strength (M_n). If M_n is greater than M_u, then the beam size is sufficient at these elevated temperatures and no further calculations are required.

If Needed Utilize Thrust Restraint. If the total static nominal flexural strength (M_n) computed above is less than the required total static flexural strength (M_u), then utilize thrust restraint and check the steel section in accordance with section 4.d.iii. If M_n is greater than M_u, then the beam size is sufficient at these elevated temperatures. If not, then the steel member is inadequate at these elevated temperatures and more fireproofing is required.

Example

Figure 11 shows the typical layout of a composite floor structure from an actual project. The fire rating is based on UL #D916.

The floor consists of a 5¼" structural lightweight concrete slab including a 2" steel deck. The span of the beams is 29' and that of the girders is 24'. The beams are uniformly spaced at 8'. Slab reinforcement consists of #4 bars spaced over the girders @ 24" c/c.

The uniformly distributed dead load is 60 psf and the uniformly distributed live load is 80 psf. The live load is reducible.

Note that the number of shear studs used for both the beams and the girders develops full composite action between the steel members and the slab.

Design of a typical beam

For a simply supported beam, the maximum moment is $M = wl^2/8$, where w is the uniformly distributed load and l is the span. The dead load on the beam w_D = 0.48 k/ft and the live load w_L = 0.52 k/ft (note that the live load includes a reduction of 18.6%). Under normal operating condition, the governing load combination is 1.2D+1.6L.

$$w_u = 1.2w_D + 1.6w_L = 1.2*0.48+1.6*0.52 = 1.4 \text{ k/ft}$$

Using the w_u, the factored moment can be computed as:

$$M_u = \frac{w_u l^2}{8} = \frac{1.4 * 29^2}{8} = 147.2\, k/ft$$

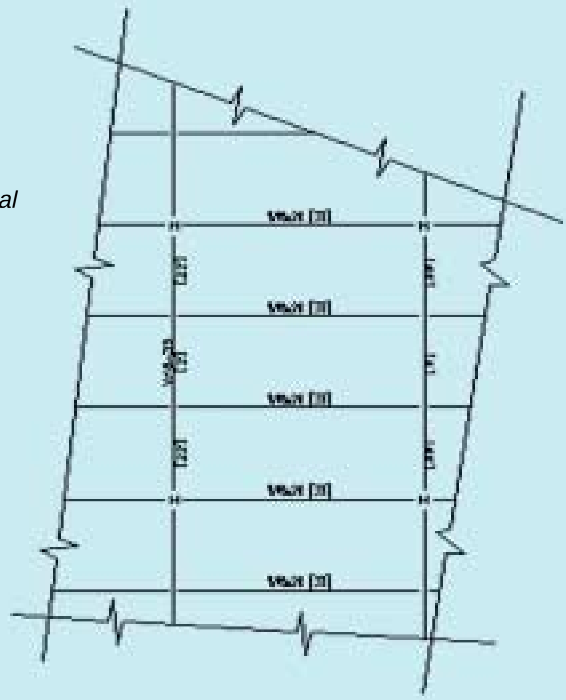
Using LRFD (AISC,1994) methods for computing the resistance of composite sections, ϕM_n=244.5 k.ft. Note that this beam is part of the unshored composite construction and the limit on construction load deflection governs the design of this member.

Analysis of a Beam Under Fire Loading

- ***Required Flexural Strength Under Fire.*** Using the governing load case under fire given in Equation (7)

$$w_u = 1.2w_D + 0.5w_L = 1.2*0.48+0.5*0.52 = 0.84 \text{ k/ft}$$

Figure 11: Typical floor layout



The factored moment under fire loading is:

$$M_u = \frac{w_u l^2}{8} = \frac{0.84 * 29^2}{8} = 88.3\,k/ft$$

• ***Nominal Flextural Strength Using AISI Temperatures.*** The first step in computing the nominal flexural strength is the evaluation of temperatures. As a first step, the temperatures are obtained from Figure 23 of *Fire Protection through Modern Building Codes* (Boring, D. F., et. al., 1981). The temperatures given are an average of the fire tests conducted at Ohio State University. However, no details on the tests assemblies are provided. After two hours of exposure to the fire load, the temperature of the bottom flange is 1300 degrees F and that of the top flange is 600 degrees F. Using Figure 22 of the same book, the yield strength of the bottom flange, f_{ybf} = 15 ksi and that of the top flange f_{ytf} = 32 ksi. The yield strength of the web is assumed to be an average of the flanges, f_{yw} = 23.5 ksi. Using these values, the properties of W16x26 and Equation (2) the tensile force F_T can be computed as

$$\begin{aligned} F_T &= F_{tf} + F_w + F_{bf} \\ &= f_{ytf}A_{tf} + f_{yw}A_w + f_{ybf}A_{bf} \\ &= 60.7 + 91.3 + 28.5 \\ &= 181 \text{ kips} \end{aligned}$$

The depth of the equivalent concrete compression zone can be computed using Equation (3) and F_T. The width of the compression flange is computed using LRFD as $l/4$ = 87".

$$a = \frac{F_T}{0.85 f'_c b_f} = \frac{180.5}{0.85 * 3 * 87} = 0.81''$$

The nominal flexural strength is computed by multiplying the tensile forces by the individual lever arm to the center of the gravity of the compression zone.

$$\begin{aligned} M_n &= F_{tf}L_{tf} + F_w L_w + F_{bf}L_b \\ &= 304.6 + 1075.8 + 579.9 \\ &= 1959.9 \text{ kip-inches} \\ &= 163.3 \text{ k/ft} \end{aligned}$$

Where L_{tf}, L_w and L_{bf} are the respective lever arms. The lever arms can be computed by subtracting $a/2$ from the distance between the top of the slab and the centers of gravity of each of the beam elements. Since $M_n > M_u$, no further calculations are necessary.

• ***Nominal Flexural Strength Using UL Fire Test Results***. In this case, the temperatures are obtained from a specific fire test conducted at the Underwriters' Laboratories, Inc. (UL File R4339-41). The temperatures recorded are for a W8x28 with ½" of fire proofing. The test assembly is slightly different from the D916 specification (which could not be obtained). The differences are such that the test temperatures may be slightly higher than those that could result form D916. After two hours of exposure to the fire load, the temperature of the bottom flange is 1500 degrees F, the web temperature is 1440 degrees F and that of the top flange is 1160 degrees F. Using Figure 22 of *Fire Protection Through Modern Building Codes*, the yield strength of the bottom flange, f_{ybf} = 5 ksi, the yield strength of the web f_{yw} = 6 ksi and that of the top flange f_{ytf} = 17 ksi. Using these values, the properties of W16x26 and the method followed above, the positive moment capacity can be computed as 61 k-ft. It can be seen that this is lower than the acting moment M_u = 88.3 k.ft.

The W16x26 (with the amount of fireproofing which is equivalent to the ½" for W8x28) is not adequate without some restraint. Therefore, the negative moment capacity provided by the support has to be evaluated.

No Reinforcement Bars Provided on Top of Girders: In this case, the negative moment capacity is controlled by either the connection or the bare steel section (concrete provides no tensile capacity). The connection is assumed not to be exposed to fire. The smallest of the typical connections can provide moment capacities of at least 20 k-ft. However, the beam a few inches from the connection is generally at the elevated temperatures because of exposure to the fire and its capacity must also be checked. The yield strengths of the flanges and the web are given above. Using these yield strengths and the properties of W16x26, the location of the neutral axis can be calculated.

In the present case, the neutral axis is 1.23" from the top of the beam. The moment capacity of the steel beam can be computed by taking moments about the neutral axis.

$$\begin{aligned} M_n &= F_{tf}y_{ft} + F_{tw}y_{tw} + F_{bw}y_{bw} + F_{bf}y_{bf} \\ &= 45.2 + 3.2 + 152 + 132.3 \\ &= 333 \text{ kip-inches} \\ &= 28 \text{ k/ft} \end{aligned}$$

where subscripts *bw* and *tw* indicated properties relating to the top (above the neutral axis) and bottom (below the neutral axis) portions of the web respectively.

This moment is higher than the assumed connection capacity, therefore the minimum of the two (20 k-ft.) is taken as the negative moment capacity. After redistribution, the total static moment capacity of W16x26 is 61+20 = 81 k-ft. This capacity is slightly less than the acting moment. The effect of the reinforcement bars provided on top of the girders can then be investigated.

Reinforcement Bars Provided on Top of Girders: In most composite construction, some reinforcing is provided on top of the girders to prevent cracking of the concrete. This reinforcement can provide significant improvement in the negative moment capacity because the reinforcement is relatively cool (approx. 300 degrees F). In the present case, 4' long #4 bars at 24" on center are provided on top of the girder. It can be assumed that all reinforcement within the bf participates in resisting the moment. This results in a total rebar area A_{st} = 0.75 in^2.It is assumed that the yield strength of the rebar is reduced by 15% because of the elevated temperature. (CRSI, 1980). Using the same procedure as above, but with the additional tensile steel from the rebar, the neutral axis can be computed as 0.28" down from the top of the beam. The negative moment capacity can be computed as:

$$\begin{aligned} M_n &= F_{RB}y_{RB} + F_{tf}y_{tf} + F_{tw}y_{bw} + F_{bw}y_{bw} + F_{bf}y_{bf} \\ &= 151 + 3 + 85 + 121 + 145 \\ &= 504 \text{ kip-inches} \\ &= 42 \text{ k/ft} \end{aligned}$$

After redistribution, the total moment capacity is 103 k/ft., which is greater than M_u.

General Discussion

This article represents only a portion of a more encompassing effort to distill existing information and develop design guidelines for fireproofing requirements of steel wide flange beams. The information herein is a progress report of the current state of this ongoing effort. This methodology is based on the method currently used in the U.S. to determine fireproofing requirements. Namely, "typical" beams are subjected to a standard fire in a standard fire-furnace and their behavior compared to prescriptive pass/fail criteria (ASTM - E119); the fire proofing thickness required for "actual" beams in the field is then based on these isolated tests. Many other countries have been using a "fire engineering" approach to predict the behavior of the "whole structure" to more realistic fire based on the available fuel load for the particular building.

Deflections govern design of many practical steel beams. The size of the section is, often, governed by the serviceability criteria and not by the strength criteria. This means that many sections have a reserve load capacity. Furthermore, the low probability of fire warrants lower load factors. On the other hand, fire reduces the load carrying capacity of the section. In many cases the combination of these factors can produce steel beams that have sufficient load carrying capacity under fire (with the fireproofing thickness prescribed for a restrained beam) without taking into account any restraint.

The smallest of "simple connections" still provides nominal strength in the order of 20 k/ft. This capacity (or the capacity of the steel section adjacent to the connection) can be used to add to the member's nominal flexural strength.

Many designers provide reinforcing bars, in the slab, over the girders to minimize potential cracking. This reinforcing can provide negative moment capacity for the beams (which are usually designed as simply supported) which can be used to offset the reduced nominal flexural strength of the beam at elevated temperatures. If needed, reinforcement over beams may be economically added near columns to increase the nominal flexural strength of girders at elevated temperatures.

The beams and girders in the example have been designed utilizing full composite action. If partial composite action is used in the design, the ultimate capacity of the shear studs must be calculated and compared to the tensile force in the reinforcing bars. If the shear stud capacity is less that the tensile force capacity of the reinforcement then FRB should be limited to the shear stud capacity.

Practically designed structures have reserve strength capacity which can be utilized for resisting loads resulting from extraordinary events, such as fire.

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