Attachment
to
08-WTP-146

Fire Resistant Design Approach for the WTP

R10931427

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RPP-WTP PDC

Document title: **Fire Resistant Design Approach for the WTP**

Contract number: DE-AC27-01RV14136

Department: Civil, Structural, and Architectural

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Document number: 24590-WTP-RPT-CSA-08-002, Rev 0

Reviewed by: Marianne Peralta

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Date of issue: 30 July 2008 7/30/08

Issue status: Approved

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History Sheet

Rev	Date	Reason for revision	Revised by
0	July 30, 2008	Initial Issue	Mark Braccia

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Definitions

Authority Having Jurisdiction (AHJ) - The Office of River Protection (ORP) is the AHJ for fire coating and cladding. As the AHJ, ORP exercises the authority to accept the adequacy of the proposed fire coating and cladding.

Design Basis Fire - Since WTP has not completed deterministically fire modeling calculations for all facilities or areas, for the purposes of this document only, the fire postulated in ASTM E-119 for evaluation of building construction and materials used in 2-hour assemblies is chosen as the design basis fire. The ASTM E-119 test fire exposure, while not representative of all fires, is recognized by literature sources as severe. For purposes of structural analyses, fires represented by an ASTM E-119 exposure are considered reasonably conservative because ASTM E-119 temperatures resulting from an unmitigated fire are not expected throughout the WTP due to the fire hazards and combustibles identified in the preliminary fire hazard analyses as well as installed fire protection features, including but not limited to, fire sprinkler systems, automatic fire alarms to summon fire department emergency services, facility construction of fire resistive and non-combustible materials, facilities made of compartmentalized fire areas bounded by qualified fire rated barriers, and compliance with applicable National Fire Protection Association Codes and Standards.

Fireproofing - The adding of fire retarding materials to delay the thermal heat transfer from a fire.

Fireproofing Hourly Rating - The rate established by testing in accordance with ASTM E-119, typically the number of hours that the assembly can continue to support service loads and keep with in prescribed heat transfers limits.

Gypsum Board Cladding - Layers of gypsum board are attached to the member to insulate the steel from the effects of fire, providing the required protection of the structural steel. Typical 5/8-in. gypsum board has the following characteristics: density of 18.75 lb/ft^3, weight per unit area used for calculating dead loads of 2.5 lb/ft^2, and compressive strength of 400 lb/in^2 (57,600 lb/ft^2).

High Density Cementitious - Portland cement-based aggregate formulation. Specification values for high density cementitious used on the WTP Project (Monokote Z-146) are 40 lb/ft^3 average dry density, 10,000 lb/ft^2 bonding strength and 550 lb/in^2 (79,200 lb/ft^2) compressive strength.

Intumescent - Intumescent fire coating is a thin film, paint-like material that can be applied to structural steel and connections by spraying or brushing to protect the steel from heat and fire. Upon heating, the intumescent material expands, creating an insulating char layer that protects the steel from high heat. Specification values for intumescent used on the WTP Project (A/D FireFilm II) are bonding strength of 10,800 lb/ft^2, and compressive strength of 157,680 lb/ft^2 at 10 %; deformation and weight are negligible.

Light Weight Fiber Board - Crushed volcanic rock manufactured into lightweight durable boards used to wrap columns and beams. Light-weight fiberboard being considered by the WTP Project (Albi Dry Clad) has a density of 10.5 lb/ft^3 and compressive strength of 6.5 lb/in^2 (936 lb/ft^2).

Low Density Cementitious - Gypsum-based cement with additives and mineral aggregates. Specification values for low density cementitious used on the WTP Project (Monokote MK-6/HY) are dry density of 15 lb/ft^3 average density, bonding strength of 300 lb/ft^2, and compressive strength of 9.7 lb/in^2 (1,397 lb/ft^2).

Primary Structural Steel (for the purposes of fire protection designation) - The columns, girders, beams, trusses, and spandrels that are fireproofed to ensure building stability and to prevent building collapse during and after fire.

Secondary Structural Steel (for the purposes of fire protection designation) – The beams, trusses, and spandrels that are not fireproofed, and not required to support stability of the building during and after a fire; these are, however, required to support the seismic design, or the weight load of concrete.

Executive Summary

The Hanford Tank Waste Treatment and Immobilization Plant (WTP) Project's primary objective for fire resistant design of structural steel is to preserve life safety and ensure that fire impacts on structural steel do not negatively impact nuclear safety. Secondarily, fire resistant design ensures that building structural integrity is maintained (property protection) and safeguards the environment. The fire resistant design criteria for WTP structural steel are compliant with the design codes, regulations, and standards as stated in the Project's safety requirements documentation.

The WTP approach systematically evaluates the WTP facility structures to ensure applicable codes and life and nuclear safety requirements are met. Confinement of nuclear materials is ensured (other than indirect impacts) by the fact that major confinement areas, hot cells, black cells, and melter caves have self-supporting heavily reinforced concrete walls and slabs, which are free-standing and do not rely on any structural steel members (fireproofed or non-fireproofed) for support. Furthermore, the impact of structural steel on Safety Class (SC) and Safety Significant (SS) equipment required to maintain confinement and/or nuclear safety will continue to be evaluated as part of the Integrated Safety Management process, and the steel requiring fireproofing will continue to be appropriately identified.

Nuclear safety is further maintained by dividing the building into fire areas, performing fire hazard analysis, providing fire barriers, and fireproofing structural steel where necessary so that facility collapse from fire is prevented by redundant structural load paths. In addition, to minimize operational risk and damage to the facility, the WTP plant is provided with multiple levels of defense-in-depth controls, including the fire barriers, administrative procedures for combustible control, fire suppression systems, and a fire protection program that includes fire alarms, detection systems, and automatic fire department notification.

BNI recognizes that in the unlikely event that all non-safety controls fail, and a major fire event occurs, there may be fire damage to non-SS/non-SC structures, systems, and components that would require post-fire assessment and subsequent repairs and/or replacement prior to restart of the plant as part of the post-fire recovery action. Because the likelihood of a catastrophic fire event is low due to defense in depth, DOE has accepted this residual commercial risk.

The WTP strategy for fire protection of structural steel is to provide 2-hour fire protection to essential structural steel members (primary steel) required to support the structure outside the self-supporting containment areas during and after a fire. The structural steel members not essential for the structural integrity of the facility are not fireproofed and are designated as secondary steel. Structural analyses for the Low-Activity Waste (LAW) Facility were performed in 2007 to demonstrate that the overall facility structure remains stable if non-fireproofed members in a fire area are rendered completely ineffective during a fire. Similarly, concrete slabs were evaluated to demonstrate that they still meet code requirements, considering increased slab spans resulting from the failure of the secondary steel. These evaluations have been reviewed by the DOE, Peer Review Team, and the Defense Nuclear Facilities Safety Board (DNFSB) staff.

In 2007, DNFSB technical staff raised additional concerns regarding the thermal growth of non-fireproofed secondary steel and the potential effects on the primary steel frame during and after a major fire.

In response to this issue, this report analyzes a representative portion of the structural steel framing of the LAW Facility at elevation 28 ft that contains both fireproofed and non-fireproofed steel. The framing

area was selected based on its geometry and configuration to produce high forces and displacements for the design basis fire. Analysis has been performed for progressively elevated temperatures based on extreme unmitigated fire in accordance with ASTM E-119, using finite element methods considering the design approach from AISC 360-05. Analysis includes thermal expansion of secondary members, and their interaction with the primary members, while reducing the material properties of all of these members due to elevated temperatures. The result shows that the primary members would have plastic deformations; however, they would still have significant capacity remaining to support gravity loads during and after a catastrophic fire in accordance with the code requirements. The evaluation also shows that, fireproofed or not, structural members will plastically deform when subjected to an unmitigated 2-hour fire matching the ASTM E-119 standard.

Pretreatment and High-Level Waste facility framing will be assessed to determine if they contain configurations that warrant additional evaluation.

1 Background and Introduction

The Hanford Tank Waste Treatment and Immobilization Plant (WTP) is located on the Hanford Nuclear Reservation in southeastern Washington state. Approximately 55 million gallons of highly radioactive and chemical wastes are stored in 177 underground tanks, some of which date back to World War II, at Hanford near the Columbia River. The WTP is part of the River Protection Project, and involves design and construction of waste treatment facilities that will vitrify some of this waste into a sturdy glass that will be sealed in structurally robust welded stainless steel containers. The low-level waste glass will be stored at the Hanford Site and high-level waste glass will be stored offsite.

The WTP consists of three main processing facilities, an Analytical Laboratory (Lab), and supporting infrastructure. The three main processing facilities are the Pretreatment (PT) Facility, the High-Level Waste (HLW) Facility, and the Low-Activity Waste (LAW) Facility. The PT Facility is where waste is introduced into the plant and split into two streams: high-level waste and low-level waste. The high-level stream is sent to the HLW Facility, where it is turned into glass for eventual repository disposal, and the low-level stream is sent to the LAW Facility, where it is turned into glass for onsite disposal. These facilities are designed and constructed in compliance with the *Safety Requirements Document Volume II* (SRD), 24590-WTP-SRD-ESH-01-001-02, which includes national codes and standards.

BNI took over management of the WTP project in 2001. At that time, the extent of fireproofing was very limited, and an equivalency approach to fire protection was under discussion. In 2005, this equivalency approach was abandoned and it was determined that the primary structural steel frame would be fireproofed (FP) to support identified fire barriers. In 2005, the project's *Structural Design Criteria* (SDC), 24590-WTP-DC-ST-01-001, was revised and Section 4.19 was added to address fire related design considerations. Section 4.19 (Appendix A of this report) of the SDC describes the criteria and methods to be used to address fire resistance design analysis for structural steel.

Confinement for WTP facilities is in accordance with DOE Standard *Fire Protection Design Criteria* (DOE-STD-1066-97), Section 9.2.2, which states:

> *Where required by the FHA or SAR, the structural shell surrounding critical areas and their supporting members should remain standing and continue to act as a confinement structure during anticipated fire conditions including failure of any fire suppression system not designed as a safety class item. Fire resistance of this shell should be attained by an integral part of the structure (concrete slabs, walls, beams, and columns) and not by composite assembly (membrane fireproofing).*

Compliance of WTP primary confinement structures is achieved through design of hot cells, black cells, and melter caves as heavily reinforced concrete slabs, and walls as free-standing and not reliant on structural steel for stability.

As part of the Integrated Safety Management (ISM) process, secondary structural steel was evaluated to identify the potential indirect impacts of a fire. These ISMs identified and will identify the secondary structural steel that must be insulated or otherwise protected, so that the Safety Class (SC) or Safety Significant (SS) equipment and structures (see Section 4.3. and 4.4 of the facility specific preliminary safety analysis reports [PSAR]) necessary to provide confinement or other essential nuclear safety functions continue to function. The existing evaluations are documented in meeting minutes such as CCNs 108487, 166144, 119707, 150221, 152063, 154023, 159359, and 159370. This is an ongoing process, which is part of the ISMs associated with the topography of the facility, that includes evaluation of the impacts of loss of strength/deflection of the structural steel, and will evaluate impact of thermal expansion of structural steel. These evaluations include the consideration of impacts on the confinement boundary, SS/SC fire barriers, and other SS/SC systems, structures, and components (SSC). As the design evolves (locations of SSCs, particularly piping and controls, is still being determined for some facilities) or items are added, removed, or modified, these changes are evaluated. These evaluations will include the impact of a fire that affects the structural steel (e.g., loss of structural strength and/or thermal expansion). Secondary structural steel requiring insulation based on the indirect impacts discussed above is included with the primary structural steel, so that the remaining secondary structural steel does not have any impact on nuclear safety. This process implements the requirements to maintain the facilities in a safe state during normal operations, off-normal conditions, and following accidents (see SRD Safety Criterion 1.0-5, 4.3-4, 4.3-7, and 4.5-3). The implementation of this requirement will be documented in the facility specific fire hazards analyses (per Criterion 4.5-3 if the SRD) supported by the facility specific PSARs. The facilities are designed to ensure nuclear safety during all accident conditions, including fires. Section 3.4.1 of the facility specific PSARs addresses specific fire design basis events, but the impact of fire on structural steel is addressed as part of the ISM topography reviews, as discussed above, rather than as specific design basis events.

Following a fire event, the facilities are designed to remain in a safe state. In some cases, the repair of SS SSCs or one train of an SC redundant system may be required to support the long term maintenance of nuclear safety following the fire.

Restart of facility operation would not occur until the facility was inspected and it was verified that the SSCs of the facility had not been compromised. The facility SSCs will be verified to provide a reasonable assurance of safety before the facility is returned to operations. Compromised SSCs would be repaired. The requirement is that, prior to restart, the facility must meet the nuclear safety requirements and prevent the loss of confinement of significant quantities of radioactive material, consistent with the requirements of the SRD for normal operation and potential off-normal and accident conditions.

The Project's safety requirements and the selected codes and regulations provide methods to achieve the primary objectives of life safety, fire protection, and nuclear safety. Life safety codes, standards, and regulations provide requirements necessary to achieve the following three primary goals:

- Allow occupants to evacuate the facility during the fire event.
- Provide sufficient time for emergency personnel to enter and exit the facility during the fire event.
- Prevent collapse of the structure.

DOE-STD-1066-97 adds nuclear safety requirements to ensure confinement is maintained and prevent release of radiation or nuclear materials during and after a fire event. DOE-STD-1066-97 also includes considerations beyond life and nuclear safety and establishes property protection as a goal in a DOE facility. The WTP performed evaluations in accordance with DOE-STD-1066-97 in project calculations 24590-WTP-U1C-FPW-00001, *Maximum Possible Fire Loss (MPFL) for LAW, HLW, PTF, BOF, and Laboratory Facilities*, and 24590-WTP-U1C-FPW-00004, *Recovery Time from Fire*, defining the maximum dollar impact and the time required to bring the facility back into operation. For example, the analysis for the LAW Facility shows areas affected by fire taking from 2 to 9 months to recover. The long lead time for equipment purchase is a greater factor in the recovery time than repair or replacement of the structural steel.

In addressing DOE-STD-1066-97 requirements, the Project also compartmentalized each facility into multiple fire areas to limit the spread of fire during an event. Each of the three main facilities and the Lab contain multiple fire areas separated by minimum 2-hour fire barriers. When considering the loss of any one area, the remaining areas contain adequate structure and bracing to prevent progressive failure and collapse.

In response to concerns raised by Defense Nuclear Facilities Safety Board (DNFSB) technical staff regarding the Project not fireproofing all structural steel, a letter dated July 19, 2007 (CCN 160917), to the DNFSB stated the following:

- The WTP SDC, Revision 12, would be revised to address how each facility shall be designed to preserve confinement capability and protect important to safety (ITS) SSCs, while accounting for degradation of the non-fireproofed (NFP) steel members as the result of a fire.
- Applicable codes and standards would be met.
- The WTP strategy would be to provide fire protection for selected structural steel members based on their role in supporting the primary steel structure during and after a fire.
- ORP acknowledged the risk that an unmitigated fire could affect non-safety-related equipment necessary to operate the plant.
- ORP considered the risk of damage to non-safety-related equipment from unprotected steel in a credible fire scenario to be extremely unlikely since the WTP is provided with multiple levels of defense in depth controls, including
  - Fire barriers
  - Administrative procedures for combustible material control
  - Automatic fire sprinkler protection systems
  - Fire alarm and detection systems
  - Automatic fire department notification
- If a fire does occur, damage to other affected SSCs must be assessed and repair and/or replacement made prior to the restart of the plant as part of the post-fire recovery.

As an additional measure, the letter committed to demonstrate that NFP structural members with reduced material properties due to a fire would not be relied upon to support the building. Furthermore, BNI would develop a technically sound methodology for identifying structural steel members that do not require fire-resistant coating. Finally, this structural analysis would be provided to support the conclusion that NFP structural steel members could fail without affecting the availability of the structure or adjacent safety systems to perform their safety functions.

This report provides analysis of a representative portion of the structural steel framing of the LAW Facility to evaluate effects of the expansion of NFP members on the primary FP members due to elevated fire temperatures, conforming to AISC 360-05.

2 WTP Structural Systems and Historical Performance of Structures Under Fire

The WTP has selected robust structural systems to achieve its mission of life and nuclear safety. The materials used are durable and noncombustible. The following table summarizes the structural systems used for the three main process facilities and the Lab.

Table 1 Structural Systems Used for the PT, HLW, and LAW Facilities and the Analytical Laboratory

Facility	Performance Category*	General	Below Grade	Process Areas – Area of Primary Confinement	Support Areas
PT	PC-3	615,219 sq ft floor area (not including 15,150 sq ft of basement area) Roof is 119 ft above grade (top of stacks above roof = 200 ft above grade) 5 story	Concrete walls and earth supported slabs	Concrete cells - two 6-ft thick concrete walls and slabs Concrete design ignoring left in place beams	1-ft thick concrete slabs supported by steel beams and columns
HLW	PC-3	453,227 sq ft 5 story 91 ft roof elevation	Concrete walls and earth supported slabs	Concrete cells - Two 5-ft thick concrete walls and slabs Concrete design ignoring left in place beams	1-ft thick concrete slabs supported by steel beams and columns
LAW	PC-2	257,427 sq ft floor area (not including 42,276 sq ft basement area) 72 ft above grade 4 story main process building; 2 story annex building	Concrete walls and earth supported slabs	Concrete cells - Two 3-ft thick concrete walls and slabs' Concrete design ignoring left in place beams	1-ft thick concrete slabs supported by steel beams and columns
Lab	PC-2	90,074 total sq ft 41 ft 5 in. high 2 story	Concrete walls and earth supported slabs	Concrete hot cell – Concrete design ignoring left in place beams	1-ft thick concrete slabs supported by steel beams and columns

* Performance Category does not influence fire resistance design requirements.

The WTP facilities are robust radiochemical facilities that contain extensive amounts of piping, ductwork, equipment, and heavy concrete members, with thicknesses often determined by shielding requirements. Unlike commercial office buildings, these facilities do not have significant combustible materials, either fixed or transient construction.

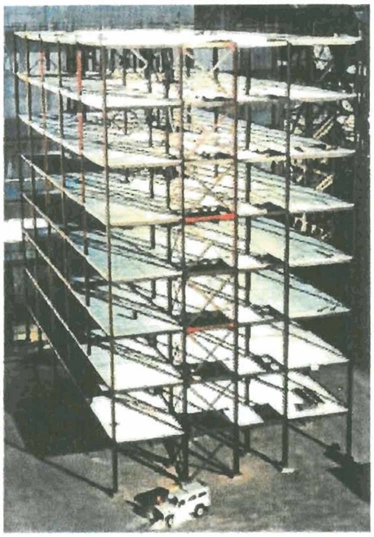
Much of the steel in the WTP is left-in-place shoring. The shoring beams were designed to support the wet weight of concrete and the construction loads. After the concrete has set, the beams are no longer relied on to support the concrete for gravity loads, but may be used to support commodities. The beams also are designed to support heavy moving equipment loads, during both construction and major maintenance operations. In select cases they also resist seismic loads; however, seismic and fire events are not considered concurrent events. The concrete slabs are heavily reinforced and are designed to span between the concrete walls. Note that the confinement of nuclear materials is ensured by the fact that major confinement areas, hot cells, black cells, and melter caves are all free-standing, heavily reinforced concrete walls and slabs that do not rely on any structural steel members (FP or NFP) for support. Concrete slabs in commercial buildings typically act composite with the steel. Making all the beams effective allows commercial buildings to use relatively thin slab construction, usually significantly thinner than the minimum 12-in. slabs used on the WTP. However, historically, steel and concrete buildings have performed remarkably well in catastrophic fire events, despite designs that are less robust than the WTP.

The First Interstate Bank in Los Angeles (Figure 1), which burned on May 4, 1988, is an example of how a structural steel building can withstand an extreme fire event without collapsing, and return to service after repair. The First Interstate Bank was constructed of a steel frame with spray-on fire-resistant coating, a minimal fire-suppression system, and a high volume of readily combustible contents, much greater than anything envisioned for the WTP. The fire started on the 12th floor and spread to successive floors via the outer walls, through broken windows. It took 2 days to extinguish, and was the most challenging and difficult high-rise fire in the city's history.

Figure 1 First Interstate Bank Building, Los Angeles, May 14, 1988

The industry has also conducted limited large-scale testing on the effects of catastrophic fire on concrete slab steel framed buildings, notably the 1993 Cardington Facility Test in Cardington, England. At Cardington, a multi-story, full-scale test facility was constructed and tested under extreme fire conditions (Figure 2).

Figure 2 Cardington Facility Test, Before and After



Before

After

Tests showed that even under extreme fire loading conditions, structural collapse did not occur. However, unprotected and protected steel did deform. Deformation cannot be avoided in fires that approach the ASTM E-119 fire temperature curve in temperature and duration. Fire protection can delay deformation by slowing temperature rise in steel members, but plastic deformation will eventually occur under these extreme circumstances.

All columns were protected using sprayed fire protection for a 90-minute fire resistance period based on a limiting temperature of 1022 °F. The perimeter columns were protected by two methods: (1) protection on all faces; and (2) protection on three sides, with the external flange unprotected (Figure 3). The internal columns had protection applied up to the underside of the connecting beams, with the connection area remaining unprotected (Figure 4). The external columns were protected to full height up to and including the connection, and also for a short length of the connecting beams (Figure 5). All beams were NFP steel.

The following are some observations and results from the test. The lateral mid-span displacement of unprotected beams ranged from 2.0 in. to 2.4 in. in 50 minutes. These beams spanned lengths of 20 ft to 29.5 ft. The internal columns displaced laterally between 0.87 in. and 2.4 in. in about 80 minutes. These columns were approximately 9.5 ft tall. The slab reached deflections ranging between 4 in. and 3 ft after 1 hour. It was a composite floor system with a 2.2-in. metal deck and a 2.8-in. concrete slab. The 3-ft deflection was at the center of one of these thin slab sections. The four-bolt connection experienced strains from 0.0012 to 0.0025. The top bolt was strained to 0.0025 after 5 minutes. The next bolt down reached a strain of 0.0025 in 18 minutes, and the fourth bolt reached a strain of 0.0012 in 25 minutes. These bolts had a measured ultimate strength capacity of 126 ksi.

Figure 3 Cardington Facility Test Perimeter Columns

Figure 4 Cardington Facility Test Interior Columns

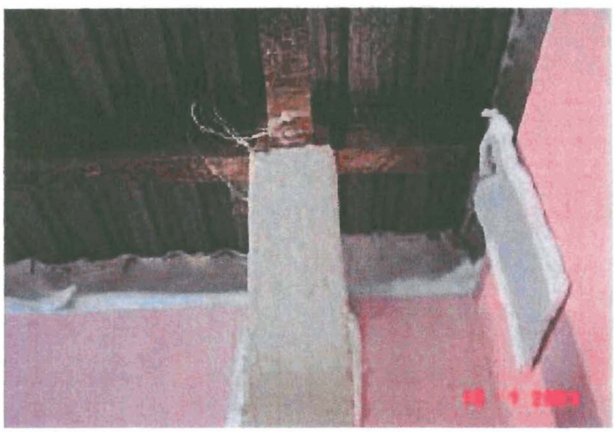
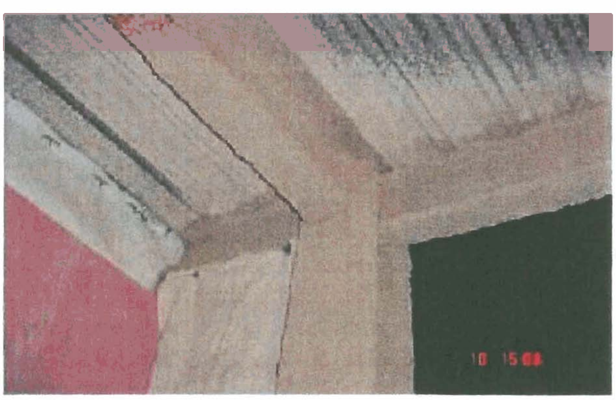


Figure 5 Cardington Facility Test Exterior Columns



The above examples show that buildings of less robust design are able to perform their function under extreme fire events. The fires they were subjected to were equal to or less intense than the design basis fire used in these evaluations. The lateral deflections in the 2-in. range and deformations are consistent with the results of WTP modeling and evaluations. These structures also relied on all members to support the gravity load case in a fire. WTP relies on only those members that are fireproofed, and designs the concrete to span between them.

3 WTP Guidance for Structural Steel Fire Resistance

The requirements and selection guidance for FP design of the WTP structures are addressed in guide 24590-WTP-GPG-CSA-00006, *Structural Steel Fire Resistance*; portions of which are excerpted below.

The regulatory requirements for fire coatings of structural steel for the WTP include the following:

- IBC 2000 establishes the fire resistance rating for building elements of specific building types in Table 601. DOE Order 420.1A requires that a fire hazards analysis be performed for the WTP facilities. It also requires redundant fire protection systems for safety class application and requires the use of DOE-STD-1066-97.
- DOE-STD-1066-97 establishes additional fire protection requirements beyond those contained in the IBC to assure the adequate protection of DOE facilities.
- The preliminary fire hazards analysis (PFHA), PSAR, and ISM processes evaluate the effectiveness of the fire protection design and assess the need for any additional fire coating and cladding that is required based on safety considerations.
- Project structural design criteria are being used to determine the amount of steel requiring fire coating and cladding, while demonstrating the fire coating and cladding applied is adequate to prevent the structural collapse of the facility in the event of a fire and failure of an unprotected structural steel member.

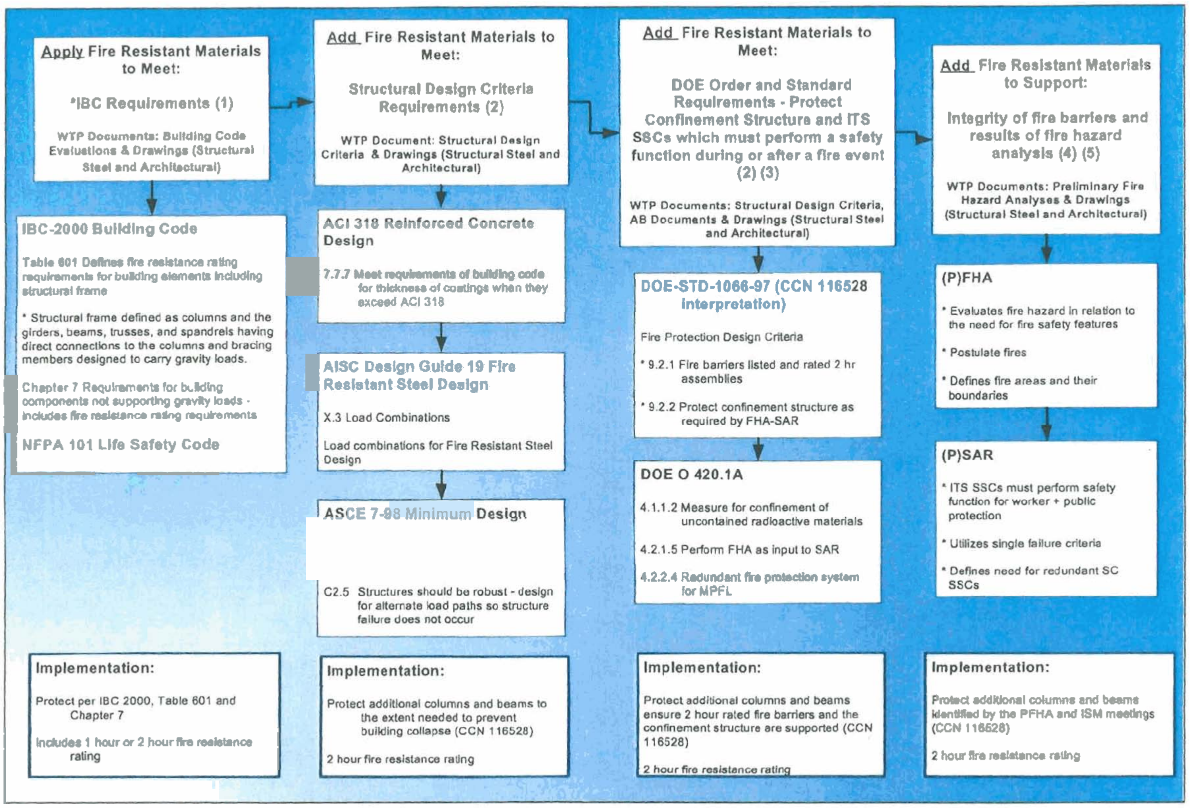
Once the requirement to provide fire resistance for the structural steel has been established, the *Structural Steel Fire Resistance* guide establishes the logic and criteria used to select the type of FP material to be applied. Criteria considerations include existence of the Underwriters Laboratory (UL) (or other nationally recognized testing laboratory) listed design for the application of fireproofed material and physical attributes associated with the type of the FP material such as weight, durability, and required thickness (space considerations).

In addition, the guide contains tables that list the types of FP materials that can be specified and establishes the limitations associated with each approach to providing fire resistance of the structural steel. Specific options for providing fire resistance for the structural steel at the WTP include intumescent, high density cementitious, low density cementitious, gypsum board cladding, and lightweight fiberboard. The selection of the type of fireproofed materials has engineering, construction, operations, and procurement implications, which are factored into consideration.

Each of these fire protection methods provides thermal protection and has been tested and listed in accordance with the ASTM E 119 test standard fire exposure. One minor difference is that cementitious and gypsum designs thermally insulate members at the start of a fire exposure while intumescent coatings lag activation when exposure temperatures reach in the range of 270 to 500 °F. Although this exposure lag is expressly dependent on W/D ratio (where W is the weight of the steel shape in lb/ft and D is the heated perimeter of the inside surface of the insulation in inches), full activation of intumescent fire coatings occur within the first 5 minutes of a ASTM E-119 test since the fire exposure in the first 5 minutes of tests reaches a 1000 °F. However, whether intumescent, gypsum, or cementitious, each system is required to protect the steel from an ASTM E-119 exposure so the average steel temperature is under 1000 °F (538 °C) or any individual thermocouple reading does not exceed 1200 °F (649 °C) during the 2-hour exposure test. Proprietary tests in UL and Factory Mutual of each of these various systems reveal that each of these materials maintains the temperature of the steel below the ASTM E-119 acceptance temperatures stated during the fire duration period, but they do not actually prevent the steel from internal heating during the exposure.

Figure 6 outlines the hierarchy of these fire-resistant requirements for structural steel.

Figure 6 Requirements for Structural Steel Fire Resistant Materials



Appendix B provides the detailed selection criteria and results for the PT, HLW, and LAW facilities and the Lab, which can be summarized as follows:

- Address all life safety issues.
- Prevent structural collapse of the facility during credible fire scenarios.
- Ensure that the structure surrounding critical areas and areas requiring separation of safety related equipment has a fire resistance rating of at least 2 hr.
- Provide adequate protection of structural steel in non-critical areas to ensure that failure of steel in these areas does not affect the structural integrity of adjacent critical areas.

4 WTP Structural Criteria and Design Basis Codes

The design requirements and design basis codes are in the *Structural Design Criteria*, 24590-WTP-DC-ST-01-001. The codes used depend on the Performance Category (PC) defined for the facility. The primary steel codes are discussed below. Also discussed are excerpts from these codes related to extreme loading and temperature conditions. The design basis steel codes for the Project do not explicitly address fire related design. However, later editions of these codes refer to AISC 360-05, which provides the engineer with approaches for considering fire accidents in the design. Calculations assessing the effects of the thermal expansion of structural steel members, including the effects of reduced material properties on the primary and secondary structural members, using AISC 360-05, Appendix 4, as guidance, were performed on the LAW structure in support of this report. Appendix D contains the code provision, the associated code commentary, and an explanation of how WTP considered the provisions in the evaluations.

4.1 Nuclear Steel Code ANSI N690 Requirements (Applicable to PC-3 and PC-4 Structures)

The nuclear facilities steel design code ANSI N690-1994 is used for the WTP PC-3 and PC-4 facilities (HLW and PT). Part 1 of the code addresses elastic design and is the basis for the design of the WTP structure, and Part 2 addresses plastic design. The code does not explicitly address fire but does address extreme temperatures on a limited basis in Section Q1.5.8, Design Based on Ductility and Local Effects:

> *In meeting the load combinations defined in Table Q1.5.7.1 for the allowable stress limits defined in Sections Q1.5.6 and Q1.5.7, elastic analysis is assumed with the following exceptions:*
>
> *a. For the abnormal, abnormal severe, and abnormal extreme load conditions, the load effects Ta, Ra, Yr, Yj, or Ym may be determined using inelastic analysis with limits on ductility factors equal to one-half the values at the onset of plastic instability, but not to exceed the values given in Table Q1.5.8.1*

In later editions of this code, ANSI 690-06, extreme temperature was removed and refers to the companion code to AISC/AISC 360-05, as discussed in Section 4.3.

4.2 Steel Design Steel Code AISC M016-89 Manual of Steel Construction - Allowable Stress Design, Ninth Edition (Applicable to PC-1 and PC-2 Structures)

This code uses allowable stress design methods and works in conjunction with UBC 1997 requirements. AISC M016-89, *Manual of Steel Construction*, is the governing code for the design of the WTP PC-1 and PC-2 facilities (LAW and Lab). This code also recognizes ductile design in Chapter N. The buildings were designed using AISC M016-89 and are assumed to absorb energy and deform during extreme events, such as seismic. Chapter N, Commentary Plastic Design, states:

> *The elements of plastic design section (N7) are portioned so they will not only achieve full plastification of the cross section, but also will remain stable while being bent through an appreciable angle at a constant plastic moment up to the point where strain hardening is initiated.*

More recent editions of the steel code used Load and Resistance Factor Design (LRFD) methods and were renamed as AISC 360-05.

Note that fire resistant materials on structural steel cannot prevent plastic deformation under extreme fire conditions; thus, maintaining linear elastic behavior is not expected by AISC 360-05.

4.3 ANSI/AISC 360-05, *Specifications for Structural Steel Buildings*

Although not currently in the WTP Authorization Basis, AISC 360-05 provides guidance to the engineer for fire-related design. This code does recognize that designing fire protection to building codes meets all the requirements of the steel design code, Design Requirement, B 10:

> *Compliance with the fire protection requirements in the applicable building code shall be deemed to satisfy the requirements of this section and Appendix 4.*

This section of the code also contains a highlighted user note:

> *User Note: Design by qualification testing is the prescriptive method specified in most building codes. Traditionally, on most projects where the architect is the prime professional, the architect has been the responsible party to specify and coordinate fire protection requirements. Design by Engineering Analysis is a new engineering approach to fire protection. Designation of the person(s) responsible for designing for fire conditions is a contractual matter to be addressed on each project.*

WTP has complied with the building code. However, DOE has directed WTP to evaluate the potential post-fire deformation in accordance with the engineering approach noted above.

Appendix 4, Structural Design for Fire Conditions, of this code provides for two additional methods of design for fire conditions: qualification testing and engineering analysis. Evaluations in this report follow the provisions in Appendix 4 regarding engineering analysis. Key code provisions were followed in this study and are summarized in the following table.

Table 2 Key Code Provisions of ANSI/AISC 360-05

Code Provision	Application on WTP
4.1	
4.1.1. Performance Objectives	The current WTP fire hazard analysis has prescribed fire areas or areas and fire barriers fireproofed to meet codes and regulations. WTP does consider the "elements that are not part of a separating element, the governing limit state is a loss of load bearing capacity." Localized evaluations of elements supporting the fire barriers have been conducted, which render the secondary beams ineffective, but include their weights.
4.1.2. Design by Engineering Analysis	Steel framing has been evaluated to the ASTM E-119 standard fire time temperature curve. WTP considers an unmitigated catastrophic fire of 2-hr duration. No reliance on administrative control or fire suppression systems can be used.
4.1.3. Design by Qualification Testing	WTP does not use this approach.

Code Provision		Application on WTP
4.1.4.	Load combinations and Required Strength	The WTP's current SDC provide fire load cases in accordance with the following: *The additional load combinations (for fire events) shall be in accordance with ASCE-7 Section 2.5 (Ref. 2.1.9) and the AISC Design Guide 19 (Ref. 2.1.16).* Overall stability and local stability of slabs and FP beams use this criterion. Expanding member evaluation (see Appendix E) used the LRFD method and AISC 360-05 values. Note: "Snow" is not included for interior floors.
4.2.	Structural Design for Fire Conditions by Analysis	
4.2.1.	Design-Basis Fire	WTP defaults to using the ASTM E-119 standard fire, using the maximum temperature and assuming the materials reach the 2-hr test limits.
4.2.1.1.	Localized fire	See section above
4.2.1.2.	Post-Flashover Compartment Fires	Not used. Approach is based on developing actual fires based on building, which is not considered in the WTP.
4.2.1.3.	Exterior Fires	
4.2.1.4.	Fire Duration	Fire is not based on combustible loading, the WTP assumes duration of 2 hr and ASTM E-119 material test temperatures are reached.
4.2.1.5.	Active Fire Protection Systems	Fire suppression system is ignored for the bounding analysis, even though most of the areas are fully sprinkled.
4.2.2.	Temperatures in Structural Systems Under Fire Conditions	Without a performance based fire, much of this section is not applicable. However, the material properties are reduced based on Table 4.2.1. It is also recognized that the NFP heat up earlier in the fire, and different stages of the fire are evaluated.
4.2.3.	Material Strengths at Elevated Temperatures	Material properties are reduced, the temperatures are increased.
4.2.4.	Structural Design Requirements	Selected approaches for evaluating the expanding members are based on the three listed approaches. The WTP starts with an individual member and expands outward case-by-case to consider a global area, which includes two typical bays with columns, girders, and secondary members.
4.2.4.1.	General Structural Integrity	WTP has been designed to taking advantage of compartmentalization, and calculations have been prepared showing that adequate bracing is provided in the non-affected fire areas to meet the intent of this section.

Code Provision	Application on WTP
4.2.4.2. Strength Requirements and Deformation Limits	Finite element models are developed and evaluated for forces, stresses, and deformation for a variety of cases and temperatures. The code states that deformation (even excessive) is acceptable once the overall load bearing capacity is maintained, which calculations show. Deformations are controlled by the design basis fire, because the FP members reach average temperatures of 1000 °F, which sets the maximum deflection of beams and frames. Vertical deformation at peak temperature is evaluated. No acceptance criteria have been developed other than prevent collapse of the structure. Design temperature: Deformation of the fireproofed girders at the 1000 °F design temperature is still significant. However, this deformation occurs even if all members were FP.
4.2.4.3a. Advanced Methods of Analysis Comments	
4.2.4.3b. Simple Methods of Analysis	The design cases considered are based on this approach.
4.2.4.4. Design Strength	Material properties are reduced as temperature increases, and evaluated to code allowables and related criteria.
4.3. Design By Qualification Testing	Not used.

AISC/ANSI 360-05, Appendix 4, and its application to WTP is discussed in Appendix D of this report.

Appendix 4 of AISC/ANSI 360-05 does not provide explicit deformation limits failure criteria. At the 2008 Fire Conference, a presentation by Mahmud M.S. Dwaikat and Prof. Kodur, V.K.R., Civil & Environmental Engineering, Michigan State University, used L/20 as a plastic deformation failure criterion. This value has been used in European design codes. This appears to be consistent with the deflection experienced in the Cardington Facility test that reached the ASTM E-119 temperatures and has been adopted by European codes. None of the analysis for WTP shows deformation approaching the L/20 criterion.

All three codes recognize the need to evaluate the amount of plastic deformation beyond yield for plastic design. ANSI N690's Table Q1.5.8.1 gives ductility ratios. The general steel codes AISC M016-89 and AISC 360-05 state: "bent through an appreciable angle at a constant plastic moment up to the point where strain hardening is initiated."

4.4 Structural Design Criteria, Rev 12

The current WTP SDC addresses fire related design in Section 4.19 (provided in Appendix A). The SDC does not recognize that plastic deformation will occur under extreme fire load cases, even in the FP primary steel. This is an unrealistic design constraint; therefore, Section 4.19 will be revised to state:

> *The failure mechanism of secondary members due to fire shall also be considered in the design of primary members. The failure of the secondary member shall not impact the primary members <u>such that they cannot carry the fire design load case post fire</u>, e.g., primary members shall be <u>evaluated for the possible deformation</u> caused from secondary member expansion, distortion or other deformation in a fire event. The primary members*

shall retain adequate post fire capacity to carry vertical loads. Post-fire, DOE will assess and, if necessary repair or replace damaged structural members as required so as to restore the load-carrying capacity of the structure for all other load cases, including the seismic load cases.

5 Fire Resistant Design Analysis

Structural framing for the LAW Facility has been evaluated in this report. Appendix C of this report provides drawings of the LAW Facility steel framing at each elevation. These drawings show the fire areas and a reflected ceiling plan of the steel framing above. They also show the relative fire hazard levels from the PFHA. This report documents the systematic approach used to ensure facility performance in case of a fire.

Integrity of the facility structure is demonstrated by first evaluating the entire structure and then focusing on local areas. The areas down to individual beam configurations are investigated for the effect of a design basis fire on the FP and NFP beams (Figure 7). Figure 7 shows a typical area of the LAW Facility with both FP and NFP members. The lighter colored beams have 2-hour intumescent fireproofing. The coatings extend out on to the secondary steel at connections, which is recommended industry practice. Evaluation was broken down into three areas of focus as follows (the first two areas have been completed in 2007 and reviewed by DOE, Peer Review Team [PRT], and DNFSB staff):

- Overall stability
- Capability of slabs to support fire case loads
- Effect of the expansion of NFP beams on the FP beams, considering the changes in material properties at elevated temperature for both members

Figure 7 Photograph of LAW Showing FP and NFP Members

5.1 Overall Stability

Overall stability of the LAW Facility is evaluated by taking individual fire areas and assuming that all NFP members in the area are rendered ineffective, then evaluating the ability of the remaining areas to provide lateral and overall stability to the structure. In addition, localized evaluations have been made of columns to evaluate the effect of losing lateral support from members rendered ineffective, thus increasing the design length of the columns. This analysis has been completed for the LAW Facility, and calculation 24590-LAW-SSE-S15T-00019 has been issued. This calculation has been reviewed by the DOE, PRT, and the DNFSB staff.

Below is an excerpt from 24590-LAW-SSE-S15T-00019, which validates overall stability of the LAW Facility. Figure 8 shows fire area boundaries at elevation 28 ft. Individual fire areas are shown with their numbers inside diamonds. The LAW fire areas shown in the PFHA are bounded by 2-hr rated walls with appropriate HVAC fire dampers, fire rated doors, and penetration seals where applicable.

WTP is not required to consider fire events in multiple fire areas simultaneously for strength and stability evaluations. When a fire is assumed to occur in one fire area, the remaining structural bracing members in other fire areas are reviewed to determine if adequate lateral bracing is present in a post-fire condition to provide stability for the facility (Figure 9, Figure 10, and Figure 11). These figures represent the major fire areas on elevation 28 ft. The areas with the hatched fencing are those assumed lost in a fire event. The bracing outside the fenced areas remains effective and provides the necessary stability.

Figure 8 LAW Building Fire Boundaries at Elevation 28 Ft

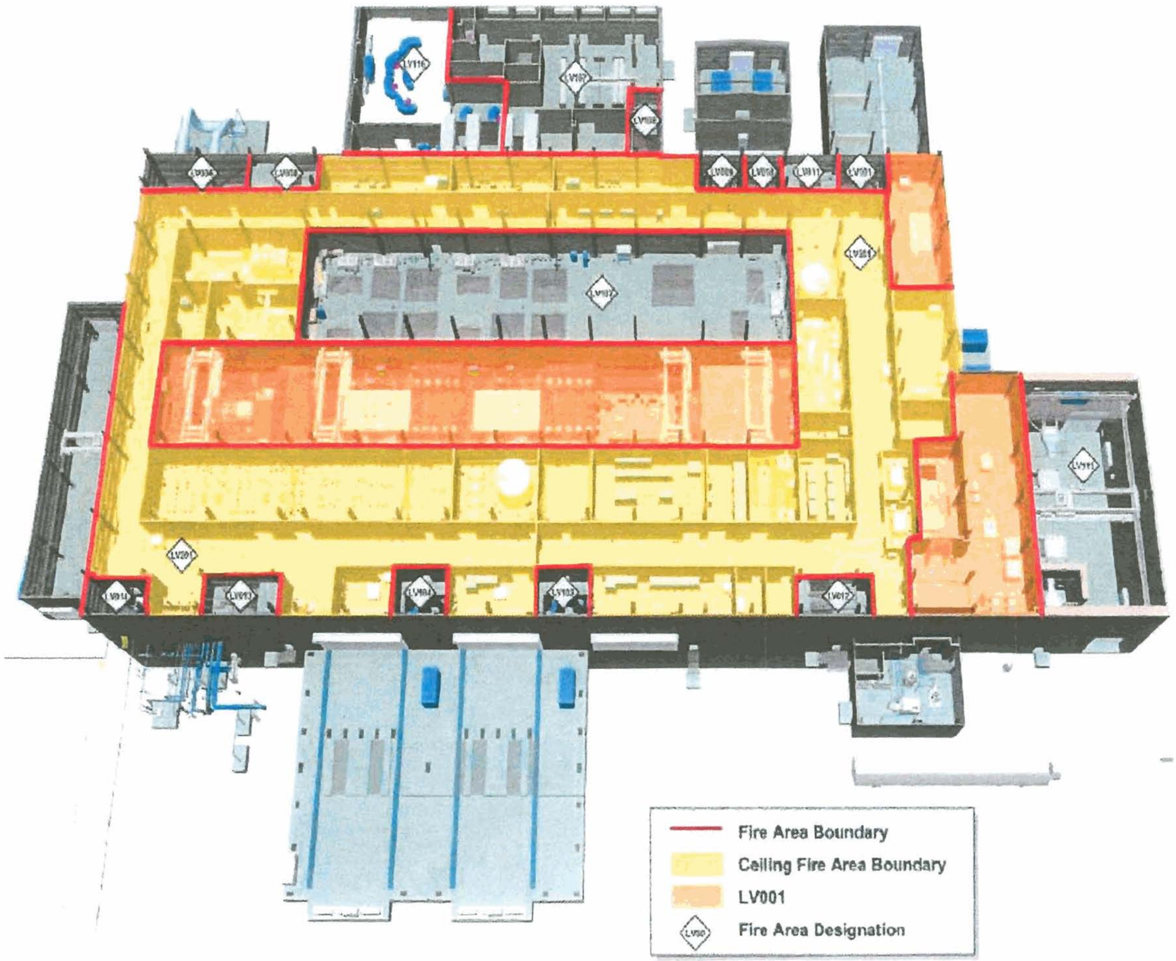


Figure 9 **Fire Area LV001 Isolation and Overall Stability with Loss of Vertical Bracing**

At elevation 28 ft a fire in zone LV201 will affect those members outside the zones shown below only. The vertical bracing in both the north-south and east-west directions in the remaining unaffected fire zones depicted below provides adequate support for lateral stability during a fire load case.

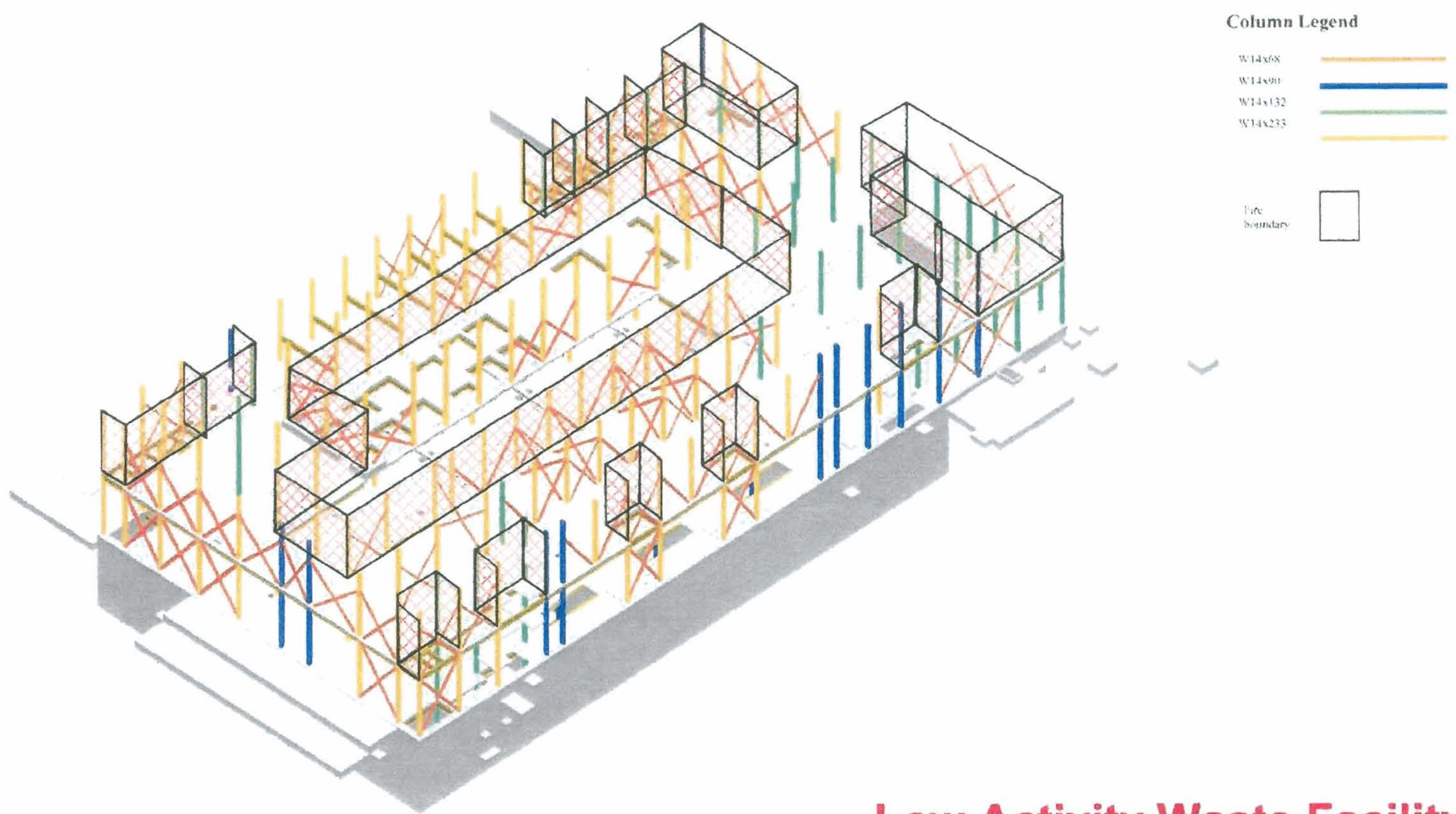


Figure 10 **Fire Area LV001 Isolation and Overall Stability with Loss of Vertical Bracing**

At elevation 28 ft a fire in zone LV001 will affect those members inside the areas shown below only. Vertical bracing shown on this fire boundary is rendered ineffective during a fire condition in this area. The vertical bracing in both the north-south and east-west directions in the remaining unaffected fire zones outside this zone provides adequate support for lateral stability during a fire load case.

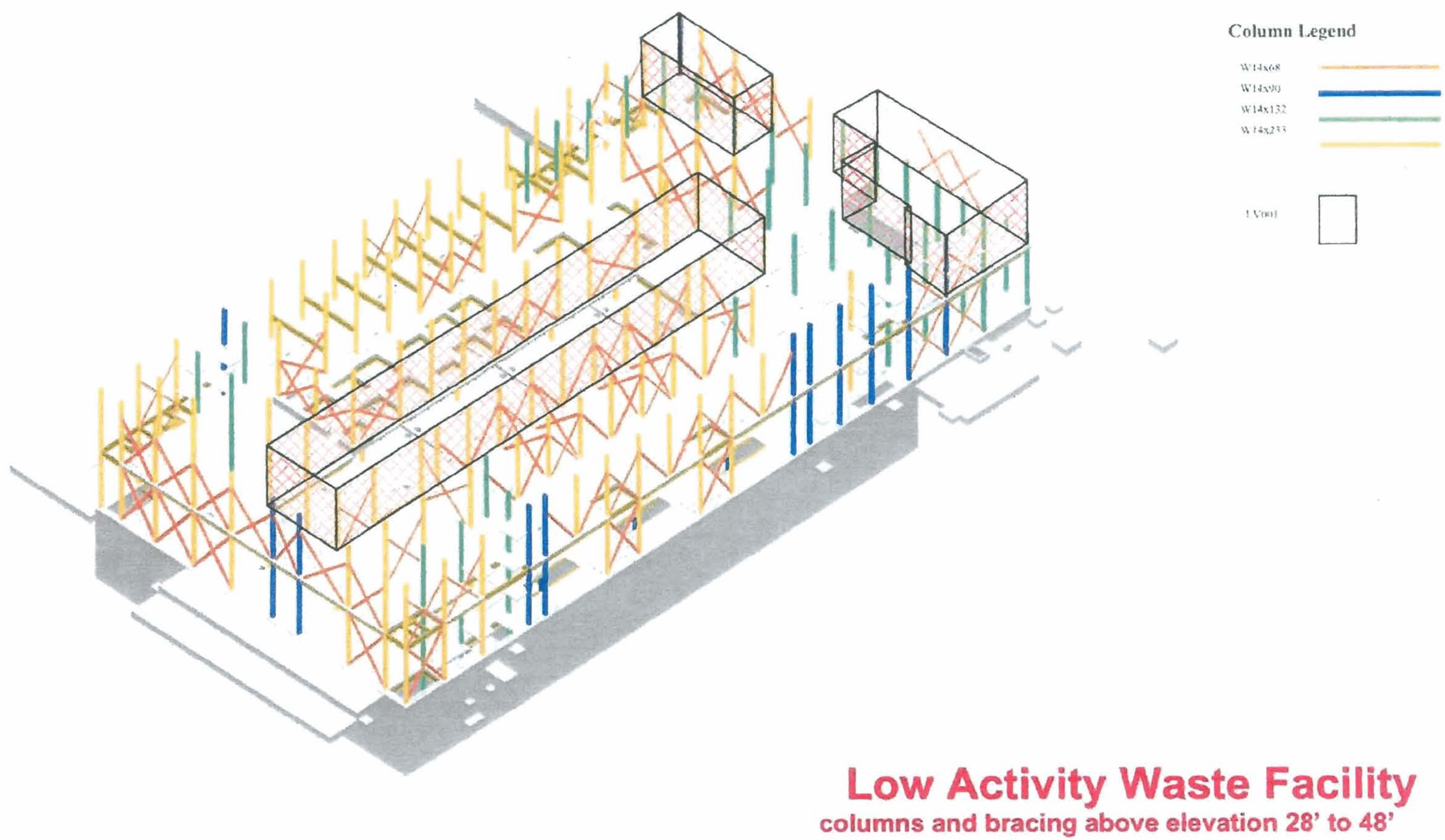


Figure 11 Fire Area LV002 Isolation and Overall Stability with Loss of Vertical Bracing

At elevation 28 ft a fire in zone LV102 will affect those members inside the zones shown below only. The vertical bracing in both the north-south and east-west directions in the remaining unaffected fire zones outside this zone provides adequate support for lateral stability during a fire load case. All other fire zones for elevation 28 ft are minor in their effect on lateral bracing and by inspection are adequate.

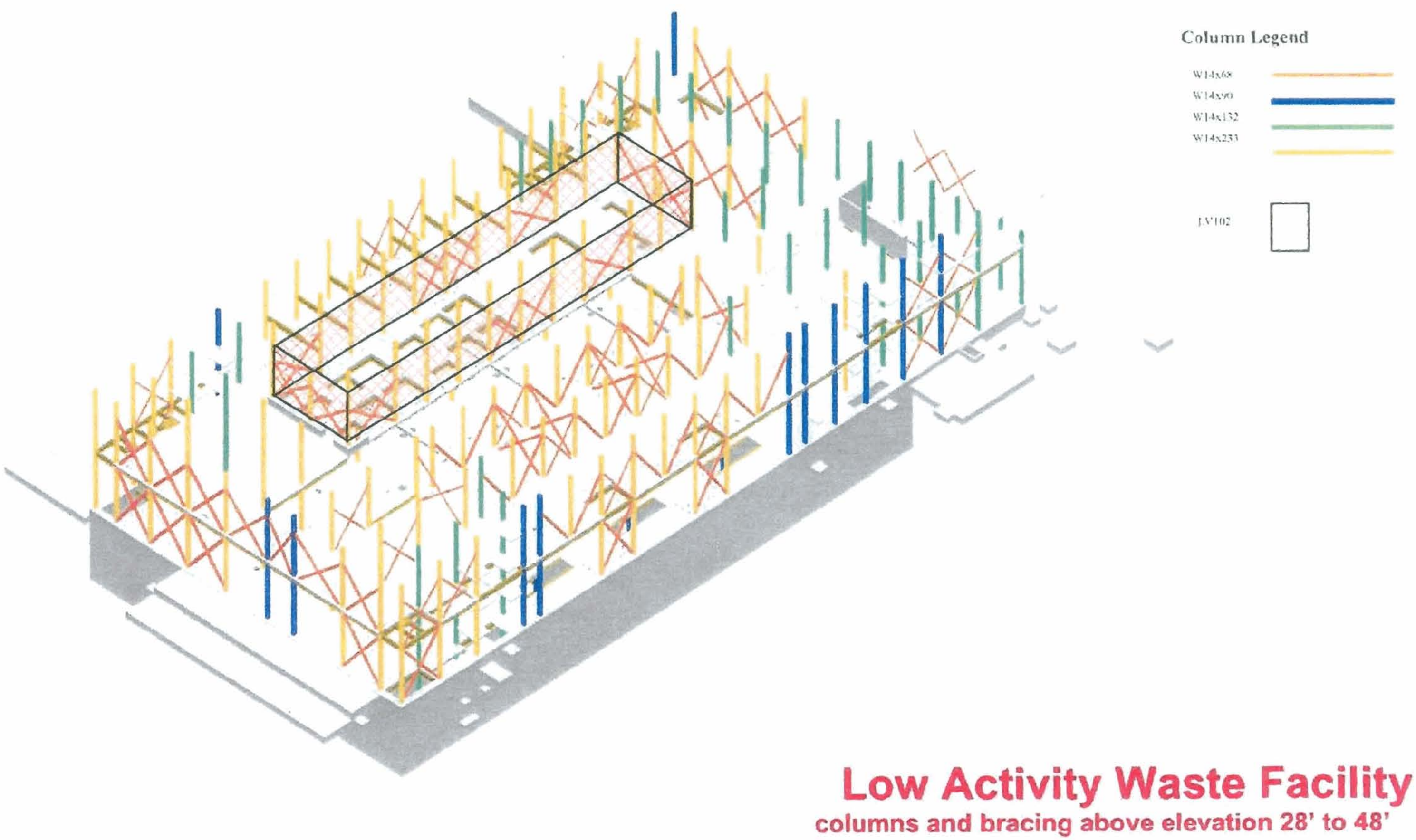
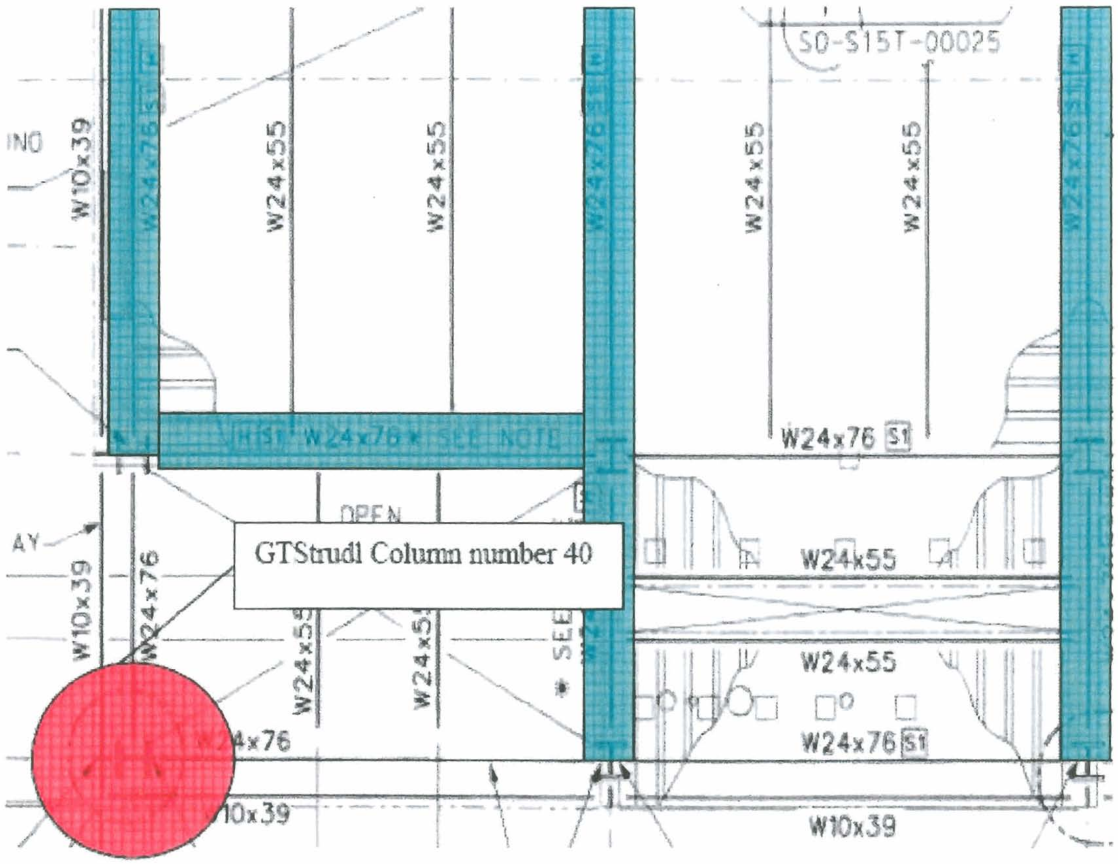


Figure 12 is the extracted model for column stability evaluation, developed to analyze one area of concern. The column being evaluated is one where loss of the NFP members would change the structural constraints. The effective length of the column would be increased. This condition is checked in 24590-LAW-SSE-S15T-00019, which shows adequate capacity remains in the LAW Facility even with NFP members rendered ineffective.

Figure 12 Extracted Model for Column Stability Evaluation

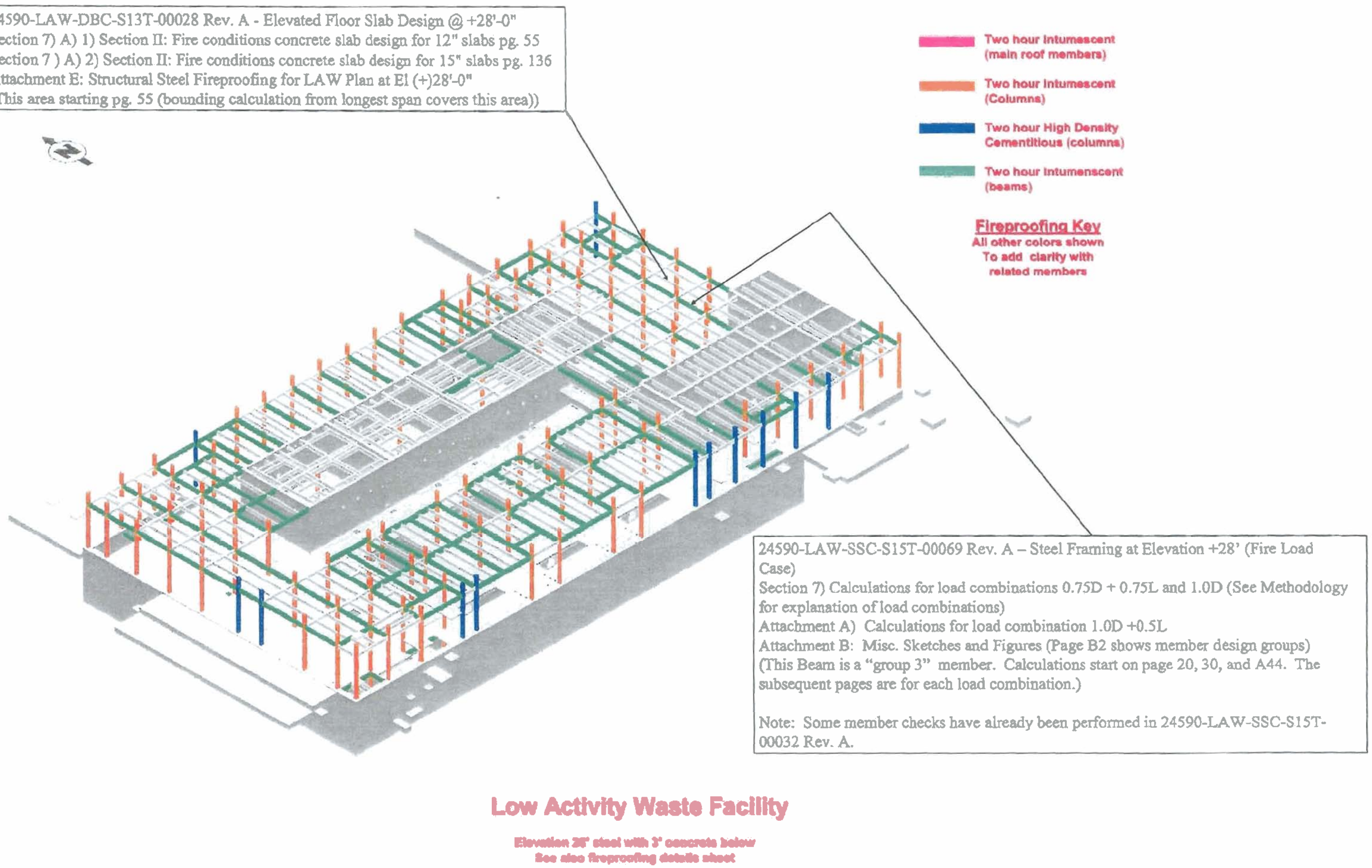


5.2 Localized Stability of Slabs to Support Fire Case Load

In the detailed calculations, the reinforced concrete slabs have been checked for their ability to span between members having 2-hr fire resistance ratings. These slabs are typically heavily reinforced both top and bottom, and the reinforcing is continuous over the span, including code-compliant splices. Figure 13 shows the framing at elevation 48 ft and the calculations both for concrete slabs and steel beams for the fire load case.

Calculation 24590-LAW-SSE-S15T-00160, which contains the figure shown below as Figure 13, documents compliance with codes and standards. The referenced calculations in Figure 13 direct the engineer to the specific location in those calculations where it is shown that the fireproofed assemblies support the fire case vertical loads. This analysis has been completed for the LAW Facility, and the calculation is issued. This calculation has also been reviewed by the DOE, PRT, and the DNFSB staff.

Figure 13 Fire Load Case Analysis



5.3 Effect of the Expansion of NFP beams on the FP beams

During a fire, NFP members will heat up faster and earlier, causing expansion and applying lateral force on the FP members. However, at the end of the design fire, the FP members have also been heated to the point that their material properties are reduced, causing deformations to both the FP and NFP members.

The following sections define the design basis fire and its associated properties used for these evaluations. The lag in heating of the FP and NFP members is addressed next. Reduction of the material properties with temperature is presented. Detailed analysis of the selected representative LAW section is provided in Appendix E.

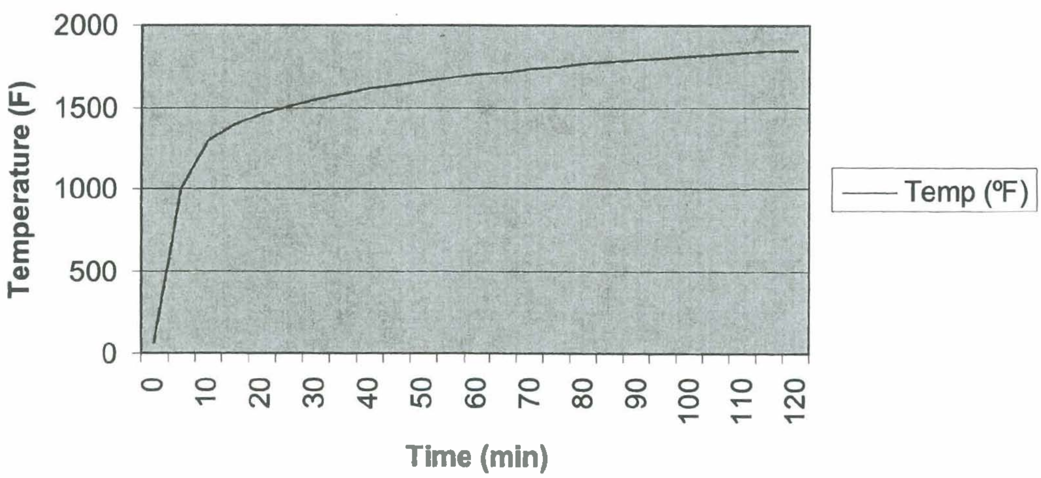
5.3.1 Design Basis Fire for Expanding Member Evaluations

The WTP is based on prescriptive compliance with codes and regulations. WTP has not completed deterministic fire modeling calculations for all facilities or areas. Therefore, fire resistant design analysis defaults to the fire postulated in ASTM E-119 (nationally recognized standard for evaluation of building materials and their construction used in 2-hour assemblies, considered conservative). Figure 14 shows the time-temperature curve for this test fire.

<u>Time-Temperature Curve</u>

The time-temperature curve is a result of the control fire tests and represents a severe fire. The characteristic points on the curve for a 2-hr overhead assembly are 1000 °F at 5 min, 1300 °F at 10 min, 1550 °F at 30 min, 1700 °F at 1 hr, and 1850 °F at 2 hr.

Figure 14 Time-Temperature Curve

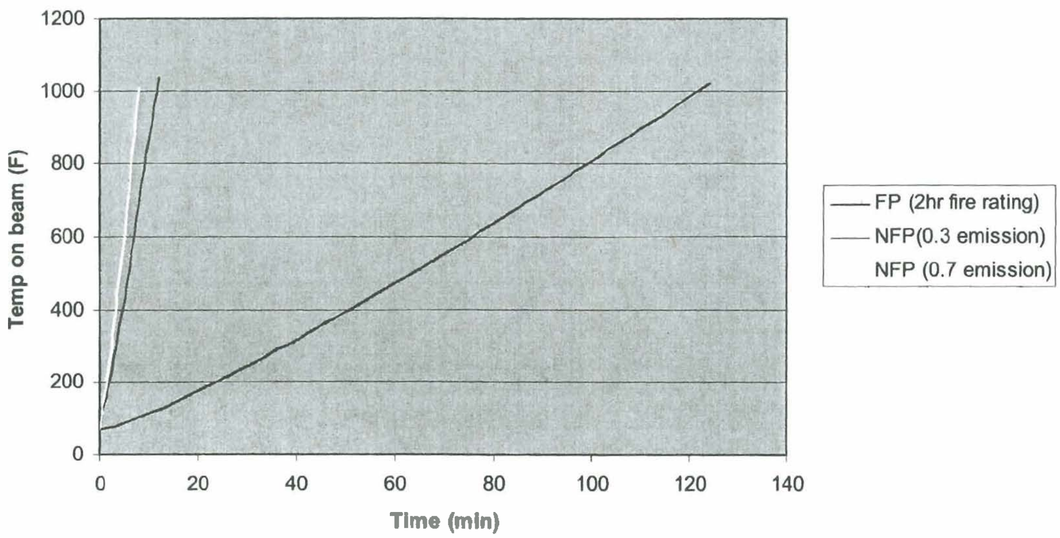


Also, the temperature profiles of bare steel and fire assemblies vary depending on amount of FP coating and member characteristics. Figure 15 illustrates the lag in temperature rise for one-member-size bare steel and with 1.5 in. of mineral fiber fire coating. The two curves for the NFP beam represent different interface assumptions and emission values with the supported structure. This graph is prepared using NUREG 1805.

Since the WTP has both FP and NFP members, the temperature characteristics of the NFP members are assessed. NUREG 1805 gives equations and tools to estimate material temperature while considering added fire coats.

The figure shows that, for a typical member size, the fireproofing significantly delays heat buildup, and the NFP members heat up early in the 2-hr event. The NFP members will become structurally incapacitated at about 1200 °F. The FP beam will reach damaging temperatures starting after 1 hr and reach the ultimate of 1000 °F at 2 hr.

Figure 15 NFP and FP Temperature Lag

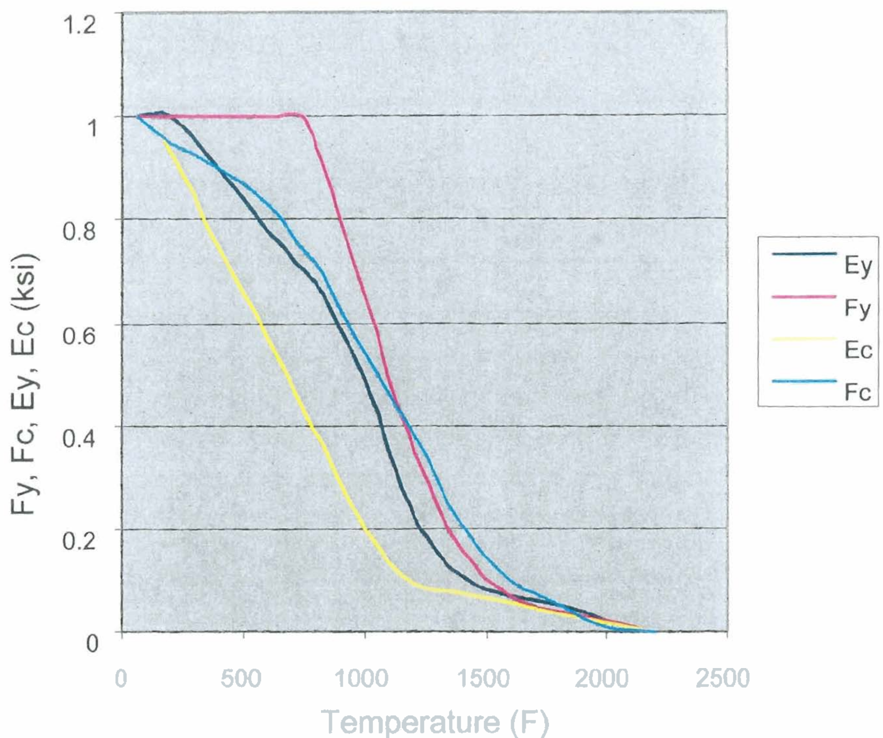


The ASTM test also set limits of the temperature of the component of the rated assembly. For steel, ASTM acceptance criteria give a peak at any location and average at multiple locations. The WTP uses the average temperature in WTP evaluations of structural response to fire. An average member temperature of 1000 °F is required for columns and beams.

WTP design requires that the rated assemblies be constructed of and supported by 2-hr assemblies. This is accomplished by designing the concrete slab to span between the FP girders or concrete walls. WTP slabs are a minimum of 12 in. thick; based on concrete thickness alone, this assembly exceeds 2-hr requirements.

AISC 360-05 provides values for critical steel and concrete properties at various temperatures (Tables A-4.2.1 and A-4.2.2). These tables are in Appendix D; Figure 16 is a graph of these tables. These properties are used in the study included in Appendix E.

Figure 16 Rate of Decline of Fy, Fc, Ey, Ec v. Temperature



The rate of decline of properties is important, specifically as it determines whether the element can still carry its load and how hard it can push on other members.

Structural steel modulus of elasticity (Es) reduces first, and the steel is able to maintain is yield strength (Fy) up to 1000 °F without significant reduction. Deflection and resulting deformation are directly related to Es, whereas member failure relates both to Es and Fy.

In the evaluation conducted, stiffness properties at the temperatures shown in Figure 16 were used. When considering more global cases, the evaluation recognized that elements of the assemblies will be at different temperatures (when FP members are a 1000 °F, NFP members are assumed to be at 2000 °F and rendered ineffective) and concrete slabs are assumed to have an average temperature of 500 °F. The 500 °F in the concrete is to take in to account expansion; the value is consistent with the Cardington and National Institute of Standards and Technology (NIST) report test data.

5.3.2 Selection of Representative Configuration (LAW) for Evaluation

The WTP facilities have many configurations and process-driven geometries, unlike office buildings, which tend to be regular in layout and construction. This being recognized, an area that captures major fire resistance design concerns has been selected based on parameters that are expected to result in fairly high forces and displacements towards a bounding case for the scenarios evaluated.

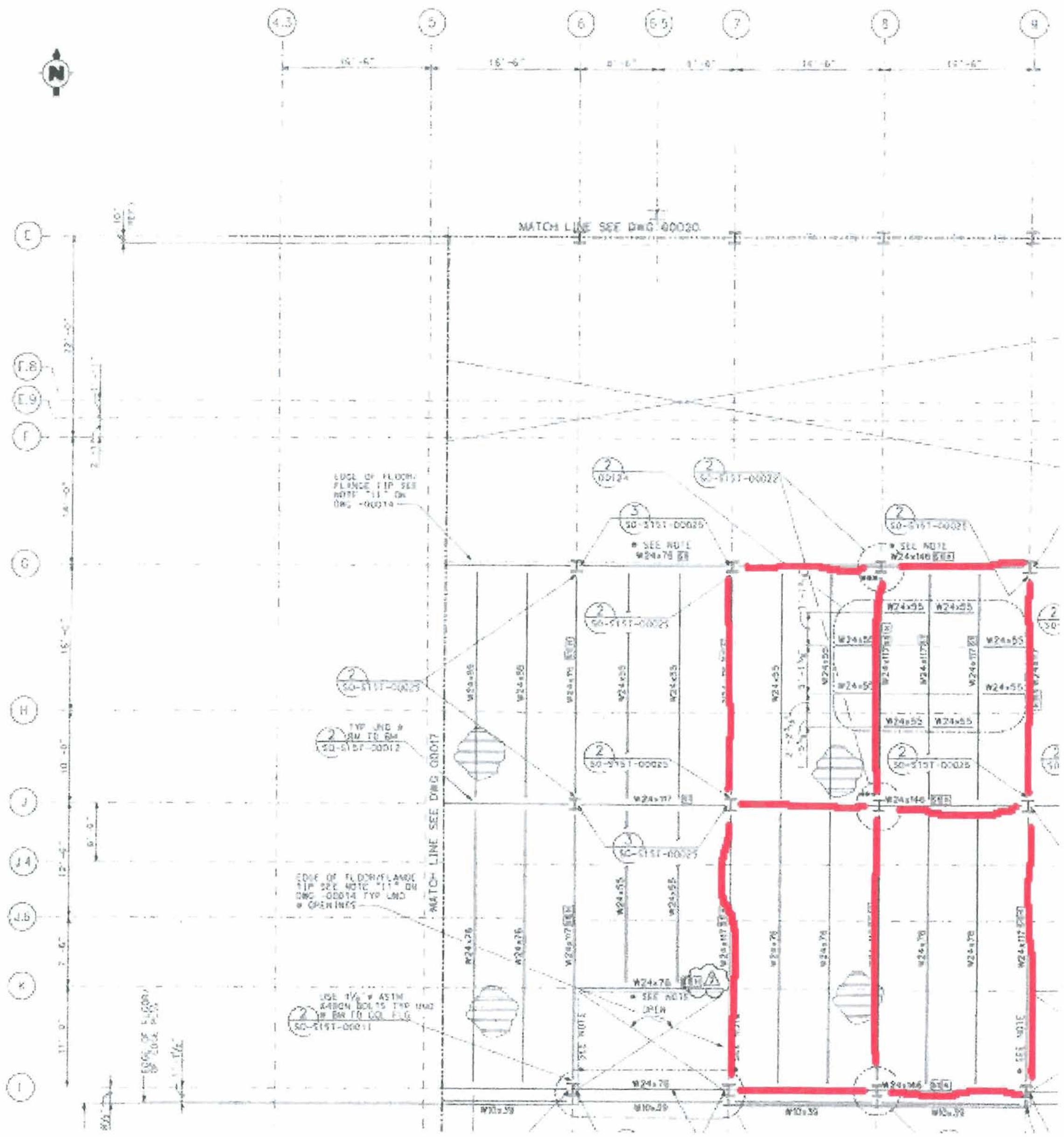
The following are considered for selecting the area:

- Representative of the typical bay spacing
- Includes both FP and NFP members

- Includes conditions where NFP member frames directly into an FP girder
- Contains longer spans, which increase thermal expansion effects
- Includes an exterior condition
- Includes a constrained edge; e.g., concrete (increases the thermal expansion effects)
- Supported by columns that are representative of typical column in the building

LAW area of framing at elevation 28 ft between column lines 7 and 9 and G and L (Figure 17) that meets the above criteria has been selected.

Figure 17 Elevation 28 Feet Evaluation Location Plan



Testing this area against the criteria, the evaluation finds the following:

- Is it representative of the typical bay spacing?
  - The bay spacing is typical for the south side of the building, and similar to the north side.
- Does it include both FP and NFP members?
  - It includes both FP and NFP beams.
- Does it include conditions where NFP member frames directly into a FP girder?
  - It has NFP beams that frame directly into FP beams.
- Does it contain longer spans, which increase thermal expansion effects?
  - It has a span of 31 ft. This compares to 29 ft on the north side of the building.
- Does it include an exterior condition?
  - It includes an edge condition along column line L.
- Does it include a constrained edge; e.g., concrete?
  - It includes a concrete constrained condition along line G.
- Is it supported by columns that are representative of typical columns in the building?
  - The floor is at elevation 28 ft, which represents the upper end of floor to floor heights.

Although, other configurations may have attributes that could drive individual deformations beyond those realized in the selected area, they would not be expected to challenge the overall stability of the structure. Considering larger fires will show more and more members being deformed (both fireproofed and non-fireproofed), but the typical bay deformations would be consistent with (if not less than) the representative area. The contribution of the adjacent areas, not in the fire, as alternate loads paths promoting overall stability is not considered in this evaluation. The restraining effects of the concrete slabs, also eliminate side sway concerns in combination with the bracing being effective in the non-fire-affected areas.

LAW, although it has a limited radiological inventory, contains longer spans typically at the lower elevations than the HLW and Pretreatment Facility which house high level waste. The spans at the lower elevations of PT and HLW are shorter, typically 18 to 20 foot range and will expand less and result in smaller deformations. PT and HLW will be reviewed to determine if there are unique configurations that challenge the representative case. If these cases are found, the same evaluation methodology will be used.

5.3.3 Selection of Evaluation Cases

In accordance with AISC 360-05, the evaluation started with individual elements and then expanded outward. Appendix E contains the detailed analysis.

The following design cases were considered:

- Three cases were modeled where only one NFP beam is heated (Cases 1a, 1b, and 1c).
- Three cases were modeled where two NFP beams, framing into a FP girder were heated (Cases 2a, 2b, and 2c).
- Two cases were modeled where only columns were heated (Cases 3a and 3b).

- One frame case was modeled where the whole frame was heated (Case 4).
- One global case was modeled where a 31-ft bay was heated such that columns were heated to 1000 °F, FP girders are heated 1000 °F, NFP beams to 2000 °F, and the 12-in. slab to 500 °F. This case was done in three steps or scenarios to iteratively approximate the conditions at different times in the fire (Case 5 Global).

Cases for the individual columns and the single frame (Cases 3a, 3b and 4) were bounded by the Global Case 5, and hence were not evaluated.

Iterative analysis using finite element method was performed due to the progressive shearing off of puddle welds (for the Q-decking connection to the beams) or Nelson studs (for beam connection to slab).

In the evaluation of Cases 1a, 1b, 1c, 2a, 2b, 2c, Case 1c bounded cases 1 and 2. In the evaluation of Cases 3a, 3b, 4, 5, Case 5 bounded cases 3, 4 and 5:

- Case 1c evaluates the maximum damage a heated NFP beam framing into a FP beam causes to the FP beam. Case 5 evaluates the damage on all of the steel members (including columns) in a heated bay.

In checking that the finite element models are conservative, the models were compared with the structural details of the general arrangements.

- How would modeling power-actuated fasteners (instead of puddle welds) to attach the Q-decking to NFP members effect our results?
- What if the stiffeners plates had been modeled?
- What if the beams as pinned-pinned rather than fixed-fixed had been modeled?

Modeling these details would have reduced the deflections and stresses on the FP members; hence, conservatively these are not modeled in the finite element analysis.

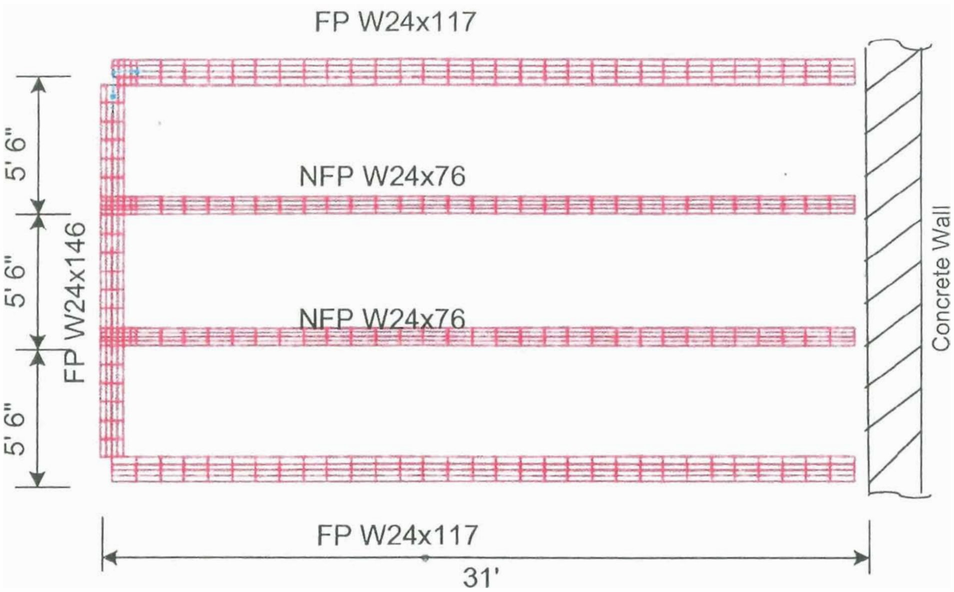
<u>Case 1c</u> (Figure 18)

Case 1c considered the following:

a) Beams frame into a non-yielding concrete wall.
b) Only one NFP beam is heated to develop the maximum force on the FP W24x146.
c) Adding additional expanding NFP beams was evaluated, but doing so reduced the deformation on the FP member.

This represents the early stage of a possible fire scenario where an NFP steel member heats up and expands before the FP member experiences any heat. This time lag in the heated NFP verses FP steel is as identified in Figure 15.

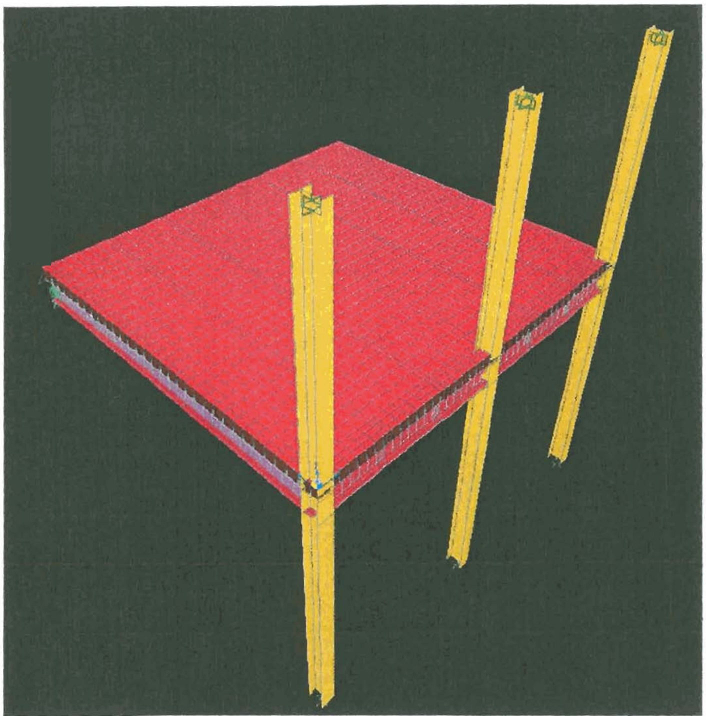
Figure 18 Plan View of Case 1c (One NFP W24x76 heated)



Case 5 (Figure 19)

Case 5 examines the effects of a typical bay as it heats up. To capture the effects on the perimeter bay beams and columns, two bays were heated and the columns below and above elevation 28 ft were modeled.

Figure 19 Isometric View of Case 5 Where a Heated Bay is Examined



5.3.4 Synopsis of Evaluations

Case 1c - One Non-Fireproofed Beam W24x76 Heated Up

A temperature of 750 °F is used since the yield strength (Fy) of steel remains unchanged up to 750 °F and the modulus of elasticity has only reduced by 30 %. A combination of these two conditions allows the NFP beam to develop the maximum force on the FP beams. Only one secondary NFP W24x76 is heated to 750 °F to produce the maximum axial load on the primary FP W24x146. Conservatively, the lower flange of the NFP beam was not allowed to buckle to produce worst lateral load on the FP beam (which could occur due to commodity supports).

A number of iterations were run. In the final run, the deflection of the FP W24x146 is 1.18 in. (Table 3) and half of the top and bottom flanges of the W24x146 have yielded, leaving an un-yielded Z-section available for major axis bending to support gravity loads. Resulting forces and deflections are noted in Table 3 (refer to Appendix E, Case 1c, page 12).

Table 3 Case 1c - One NFP W24x76 Heated to 750 °F

Member	Temperature (°F)	Elongation (in.)	Lateral Deflection (in) /	Axial_allowable (kips)	Axial_potential per SAP (kips)
NFP W24 × 76	750	less than 1.18		586	644
FP W24 × 146	**68**		**less than 1.18**		

The partially yielded FP W24x146 was checked for the gravity load of 1.2D +0.5L +T +0.2S for fire load case in accordance with AISC 360-05. The Z-section property was used due to partially yielded FP W24x146, and was found to have adequate capacity to resist gravity loads post fire with D/C=0.167<1.0 (see Appendix E, pp 14, 17-18).

Case 5 – Evaluation of a Bay Fully Heated (Including Columns)

Two bays between column lines 7 - 9 and J - L on elevation 28 ft were heated using progressively increasing temperature scenarios (Table 4). Columns were added to the model at column lines at L7, L8, and L9. The column heights are 25 ft below the slab and 19 ft above the slab. The columns were assumed pin-connected at the bottom and fixed at the top. As discussed earlier in this section, two bays were heated to capture the thermal effects on the perimeter beams.

Table 4 Case 5 Temperatures (F) Used in the Finite Element Model

Scenario	NFP W24 × 76 (2) temperature (°F)	FP W24 × 146 temperature	FP W24 × 117 temperature (°F)	FP W14 × 233 Column temperature (°F)	12 in. slab temperature (°F)
1. Maximum force from NFP beam	750	200	200	200	200
2. Maximum force from FP beam	1400	750	750	400	400
3. Columns heated to max temp	2000	1000	1000	1000	500

As in Case 1c, the steel and concrete modulus of elasticity were adjusted per AISC 360-05 to account for the temperature load. A cracked concrete modulus of elasticity was used. The thermal coefficient of expansion for steel and concrete was adjusted per AISC 360-05 for elevated temperatures.

<u>Scenario 1</u>

This scenario depicts the estimated condition early in the fire. The NFP beams are heated to 750 °F, which produces maximum axial force. FP members are at a lower temperature, accounting for the time lag between FP and NFP steel (Figure 15).

At 750 °F, the axial load on the NFP beam W24x76 is 119 kips due to buckling before developing the full expansion force; hence it deflects the FP beam W24x146 to less than the potential 1.9 in. The FP column W14x233 and FP beam W24x117 displace 0.62 in. Table 5 summarizes the resulting forces and deflections (refer to Appendix E, pg 21). The analytical model conservatively predicts a moment of 528k-ft in the column because of the connectivity of the FP W24x117, the column, and the slab. If a simple model of a simply supported column between Elevation 3 ft and 47 ft is given a deflection of 0.62 inches at Elevation 28 ft, the moment in the column is approximately 198 ft-kip (Appendix E, pp 23-24). The actual value is between these two cases.

Table 5 Case 5 Fire Scenario 1

Member	Temp (°F)	Elongation (in.)	Lateral Deflection (in.)	Axial_allowable (kips)	Axial_potential per SAP (kips)	Moment (kip-ft)
NFP W24 × 76	750	less than 1.9		119	284	
FP W24 × 146	**200**		**less than 1.9**			
FP W24 × 117	**200**	**0..62**			**363**	
FP W14 × 233	**200**		**0.62**			**528**

Scenario 2

FP beam temperatures are raised to 750 °F to produce their maximum axial force on the columns. The columns and slabs are at a lower temperature. The NFP beams have reached temperatures where their axial force and the deflection are reduced from the scenario 1. Both the FP W24x117 beam and the FP W14x233 column displace 1.55 in. since they share the same nodes. A 715 ft-kip moment on the column is produced, as shown in the Table 6 (refer to Appendix E, pg 21). In the model, the column below elevation 28 ft is heated to 400 °F, and the column above elevation 28 ft is kept at the ambient temperature of 68 °F. The worst moment at elevation 28 ft is recorded below. The analytical model conservatively predicts the moment in the column because of the connectivity of the FP W24x117, the column, and the slab. If a simple model of a simply supported column between elevations 3 ft and 47 ft is given a deflection of 1.55 inches at elevation 28 ft, the moment in the column is approximately 465 ft-kip (Appendix E, pp 23-24). The actual value is between these two cases. The maximum combined stress on the heated FP column was found to be less than 1.0 (D/C=0.979 for Case 5, scenario 3). Using the simple column model, the D/C ratio would reduce to approximately 0.53. The actual D/C is between 0.979 and 0.53.

Table 6 Case 5 Fire Scenario 2

Member	Temperature (°F)	Elongation (in)	Lateral Deflection (in)	Axial_allowable (kips)	Axial_potential per SAP (kips)	Moment (kip-ft)
~~NFP W24 ×~~ 76				119	186	
FP W24 × 146	**750**					
FP W24 × 117	**750**	**1.55**			**230**	
FP W14 × 233	**400**		**1.55**			**715**

Scenario 3

The FP beams and columns are heated to their 2-hour fire rating of 1000 °F, and the concrete slab is taken to 500 °F. FP W24x117 elongates by 2.5 in., deflecting the FP column W14x233 by the same amount. This produces a 1021 ft-kip moment on the column, as shown in Table 7 (Appendix E, pp 22 and 23). Figure 20 shows the final deflected form of the bay. As pointed out in scenario 2, the analytical model conservatively predicts the moment in the column because of the connectivity between the W24x117 beam, the column, and the slab. If the simple model of the column described above is used, the moment due to a 2.5 in. lateral deflection at 1000 °F is approximately 501 ft-kip.

It should be noted that the biggest effect on the deflection, and hence the moment and the stress on the FP column is due to expansion of the fireproofed beam W24x117 (not the expansion of NFP beams). Hence, it is concluded that the expansion of the NFP beams does not have significant impact on the load carrying capability of the FP beams or columns.

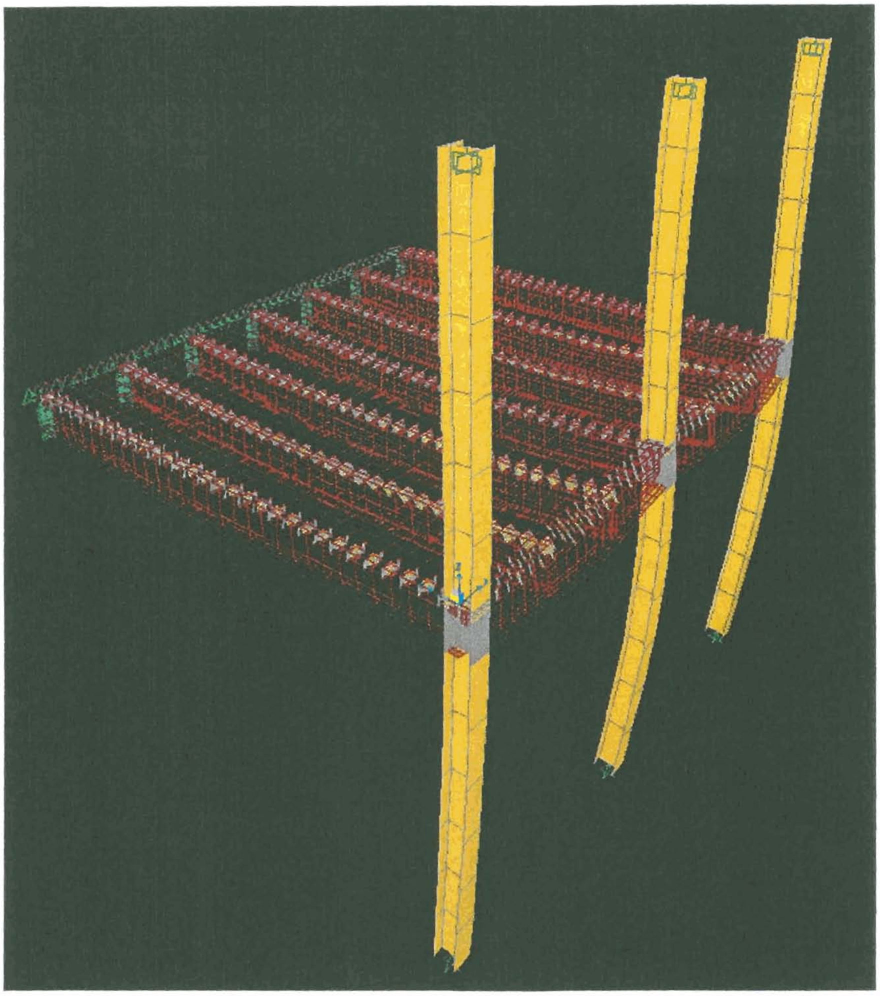
Table 7 Case 5 Fire Scenario 3

Member	Temperature (°F)	Elongation (in.)	Lateral Deflection(in)	Axial_allowable (kips)	Axial_potential per SAP (kips)	Moment (kip-ft)
NFP W24 × 76	2000			4		
FP W24 × 146	**1000**					
FP W24 × 117	**1000**	**2.5**			**401**	
FP W14 × 233	**1000**		**2.5**			**1021**

The strength capacity, accounting for P-delta effects, of the FP W14x233 column was checked for the 2.5-in. lateral deflection along with the gravity loading combination of 1.2D + 0.5L + T +0.2S. The combined stress on the heated FP column was found to be less than 1.0 (D/C=0.979). Using the simple column model described above with a 2.5-in. lateral deflection, the D/C would reduce to approximately 0.6. The actual D/C is between 0.979 and 0.6. The FP W24x146 beam was checked with the partially yielded Z-section for the gravity loads post fire that resulted in a D/C = 0.253<1.0.

Hence, all FP beams and columns have adequate capacity to support post-fire loading, complying with the requirements of AISC 360-05.

Figure 20 Deflected Shape Case 5 Scenario



6 Conclusions

Overall stability of the LAW Facility framing has been checked and is acceptable, as verified in previous Project calculations. Also, local stability and the ability of the FP members to carry gravity loads while considering the NFP member rendered ineffective has been shown and documented in the Project calculations. These two checks meet the requirements for fire design based on the prescriptive code approach required on the WTP. Existing ISM evaluations have identified steel requiring fireproofing to prevent damage to SC or SS SSCs required to maintain confinement and/or ensure nuclear safety. As the design evolves, these requirements are revised as appropriate.

Additionally, the effects of expanding members during a design fire case, using the guidance in AISC 360-05, Appendix 4, using the extreme unmitigated fire load per ASTM E-119 2-hr test fire, has been evaluated in this report. This is a conservative fire assumption for the beyond code evaluation in this report.

The evaluations show that although limited plastic deformation will occur, there remains adequate capacity to support the fire case loads as defined by AISC 360-05, with a D/C ratio of 0.253 for the FP beams and a D/C ratio between 0.6 and 0.979 for the FP columns. It should be noted that the biggest effect on the deflection, and hence the moment and the stress on the FP column, is due to the expansion of the fireproofed beam W24x117 (not the expansion of NFP beams). Hence, it is concluded that the expansion of the NFP beams does not have significant impact on the load carrying capability of the FP beams or columns.

In summary, it is concluded that the fire protection of the WTP facilities provides protection to the steel such that it will not fail or deform within the time frame required by code for safe egress and confinement. Limiting the fireproofing to the primary frame is an acceptable commercial risk and does not create conditions that overwhelm the primary structure to the point of losing its function during and after a catastrophic fire. It also concludes that permanent deformations will occur from an unmitigated catastrophic fire, regardless of whether the steel is fire protected or not. Hence, preventive and defense-in-depth measures designed in the WTP, as noted earlier, are adequate to mitigate potential damage and disruption of operation. LAW Facility fire resistant design analysis concludes that adequate structural design margin with fireproofing of the primary frame prevents building instability and/or collapse, when exposed to a 2-hr fire event.

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Appendix A

Section 4.19 of 24590-WTP-DC-ST-01-001, *Structural Design Criteria*

Appendix A Section 4.19 of 24590-WTP-DC-ST-01-001, Structural Design Criteria

Fire Resistant Design of Structures

This section addresses the criteria for the design of structural steel elements. See reference 2.4.29 for guidance in selecting fire coatings and ASCE/SEI/SFPE 29-99 Table X3.1 Construction Classification for Restrained and Unrestrained for guidance in determining thicknesses of structural steel fire coatings.

Project structures shall be designed for fire resistance as required to meet building codes, support fire barriers, support fire protection features, support the confinement structure, or to protect ITS components. Fire resistance of buildings and structures is ensured by fireproofing selected structural members and through evaluations for stability and strength of the structure during and immediately after a fire event.

For the purpose of fire resistant design, columns, girders, purlins, beams, bracing, or floor slabs that are required to support the design loads during and after a fire are defined as primary members. Members that are not required to support design loads during or after a fire are defined as secondary members. Primary members are protected by applying (e.g., sprayed or wrapped) fireproofing material, whereas secondary members are not protected by fireproofing (Figure 3) and not considered active in the stability and strength evaluations. Secondary steel members may span between primary members, concrete walls or other secondary members. They support floor slabs and/or primary members to provide vertical and lateral support for normal operating plant conditions and seismic events. Reinforced concrete walls and slabs are protected by their concrete cover and do not need additional fireproofing.

The use of non-fireproofed secondary members to carry Safety Class commodity and equipment loads is not permitted unless physical separation redundancy is provided for the safety class commodities. The ISM process through focused reviews may determine and document additional fireproofing requirements to ensure adequate protection of safety class commodities.

The primary and secondary members shall be designed to meet the loads and load combinations of Sections 5 and 6. Additional load combinations and stability evaluations are required for fire events for the primary members.

The additional load combinations (for fire events) shall be in accordance with ASCE-7 Section 2.5 (Ref. 2.1.9) and the AISC Design Guide 19 (Ref. 2.1.16). It should be noted that the load combinations for fire cases do not include seismic or wind loads. For the fire loading cases, the floor slabs and roof decks shall be designed for longer spans considering the loss of non-fireproofed structural members. Similarly, primary girders are required to carry the larger tributary areas of floor slabs or roof decks that span between fireproofed members. Secondary vertical and horizontal bracings that are not fireproofed shall not be considered active when computing forces and moments in the primary members (columns and girders). Similarly, primary members (columns and girders) shall not be considered laterally supported at those bracing points.

Building stability, individual column stability and strength shall be ensured during fire events. This stability evaluation shall account for longer unbraced column lengths due to the loss of non-fireproofed secondary beams, girders, and diagonal bracings that are postulated to no longer provide lateral support to the columns. The design demand of columns shall be evaluated for higher loads caused by the loss of fire-degraded vertical-load members (bracing that carries vertical load) that are non-fireproofed and postulated to be inactive in the structural path. It is not required to consider fire events in multiple fire areas for the strength and stability evaluations.

The failure mechanism of secondary members due to fire shall also be considered in the design of primary members. The failure of the secondary member shall not permanently impact the primary members, e.g., primary members shall be designed to resist loads from secondary member expansion, distortion or other deformation in a fire event. The primary members shall be able to retain their design capacity post fire and not require replacement. All damaged structural members will be repaired/replaced as necessary so as to restore the structure as it was originally designed.

Load Combinations for Fire Case

- **Primary Steel Girders** are fireproofed girders that frame into concrete walls, Primary columns or other fire-protected Primary girders; and along with the concrete walls, support the floor slabs during fire events. Following are the additional load combinations, enveloping ASCE-7 fire load combinations.

  For SC-I, SC-II, SC-III and SC-IV structures:

  $1.33S = D + \text{Equip} + L$

  $S = D + \text{Equip} + 0.5L$

  Note: The load term A_k is not used in the above load combinations because it is taken as zero. The load term A_k is taken as zero because there are no transient (i.e., explosive) load cases.

- **Primary Steel Columns** are fireproofed columns that support vertical loads during fire events. These members are designed and evaluated for stability accounting for the loss of lateral support provided by non-fireproofed structural members, and for the additional vertical load released by non-fireproofed vertical bracing. Following are the additional load combinations, enveloping ASCE-7 fire load combinations.

  For SC-I, SC-II, SC-III and SC-IV structures:

  $1.33S = D + \text{Equip} + L$

  $S = D + \text{Equip} + 0.5L$

  Note: The load term A_k is not used in the above load combinations because it is taken as zero. The load term A_k is taken as zero because there are no transient (i.e., explosive) load cases.

- **Floor Slabs** are concrete floor slabs that span between walls and Primary girders during fire events, and span between steel beams and girders for normal loading. Following are the additional load combinations, enveloping ASCE-7 fire load combinations.

  For SC-I, SC-II, SC-III, SC-IV structures:

  $U = 0.75\ (1.4D + 1.4\ \text{Equip} + 1.7L)$

  $U = 1.4\ (D + \text{Equip})$

  Note: The load term A_k is not used in the above load combinations because it is taken as zero. The load term A_k is taken as zero because there are no transient (i.e., explosive) load cases.

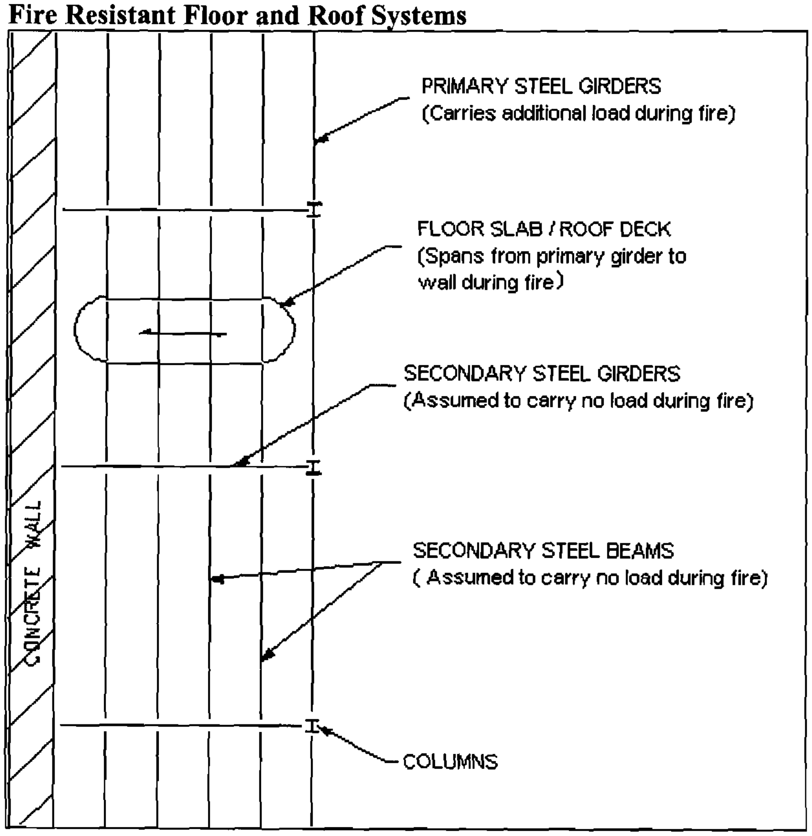
- Secondary Steel Girders and Beams are non-fireproofed girders and beams that are assumed to have zero strength during fire events. These members span between Primary girders and concrete walls, they support floor slabs and provide lateral support to columns for normal operating plant conditions and seismic events. No additional load combinations are required.

Roof Member Load Combinations for Fire Case

Roof decks are classified as fire barriers or non-fire barriers depending upon the facility's building code classification. Roof structures that are fire barriers shall be designed and supported similar to floor slabs for fire events. Roof structures that are non-fire barriers do not require fireproofing. However, these roof structures shall be designed to allow the non-fireproofed members in a fire area to be rendered ineffective without causing collapse of the remaining structure.

- **Primary Roof Girders** are fireproofed steel girders that frame into Primary columns and support roof decks during fire events, they are evaluated the same as Primary Steel Girders.

Fire Resistant Floor and Roof Systems



SC-I and SC-II Facility Design Requirement

Appendix B

Governing Codes and Standards

Appendix B Governing Codes and Standards

International Building Code

Table 601 of the International Building Code (IBC) outlines the fire-resistance requirements for fire coating and cladding structural steel on the WTP Project. Following the criteria delineated in the IBC, the Low-Activity Waste (LAW) and High-Level Waste (HLW) facilities and the Analytical Laboratory (Lab) are Type II B facilities and the structural steel for these facilities, as outlined in Table 601, does not require fire coating and cladding. The Pretreatment (PT) Facility is a Type I B facility and the structural frame requires 2-hour fire coating or cladding, except for those structural members supporting the roof (Table B-1, IBC Table 601).

DOE Orders and Standards

DOE Order 420.1A and DOE-STD-1066-97 (as interpreted by Reference 1) establishes additional fire coating or cladding requirements for DOE facilities. The additional requirements to fireproof structural steel imposed by DOE-STD-1066-97 are outlined in Section 9.2.2:

> *The development of an FHA and SAR should include consideration of conditions that may exist during normal operations and special situations (e.g., during periods of decontamination, renovation, modification, repair, and maintenance). Where required by the FHA or SAR, the structural shell surrounding critical areas and their supporting members should remain standing and continue to act as a confinement structure during anticipated fire conditions including failure of any fire suppression system not designed as a safety class item. Fire resistance of this shell should be attained by an integral part of the structure (concrete slabs, walls, beams, and columns) and not by composite assembly (membrane fireproofing). In no event should the fire resistance rating be less than 2-hours under condition of failure of any fire suppression system not designed as a safety class item. (Refer to NFPA Standard 221 and FM Data Sheet 1-22).*

The DOE-STD-1066-97 was further interpreted by ORP in Reference 1:

> *DOE interprets the above standard to also require that the failure of structural steel in non-critical areas, due to credible fire in those non-critical areas, would not cause the structural shell for the critical areas to lose confinement, or to impact important-to-safety structures, systems, and components, which must perform safety functions during a fire event. Consequences of credible fires in all areas must be considered in determining whether the shell surrounding critical areas remains functional. Specifically, consideration must be given to whether a credible fire outside the critical areas can cause sufficient damage to the unprotected structures to impair the structural shell of critical areas.*
>
> *Therefore, BNI, as the Design Authority, is requested to provide adequate fireproofing of the structural steel in the non-critical areas to preclude the above-mentioned impacts, or provide technical justification for not fire-proofing these structures.*

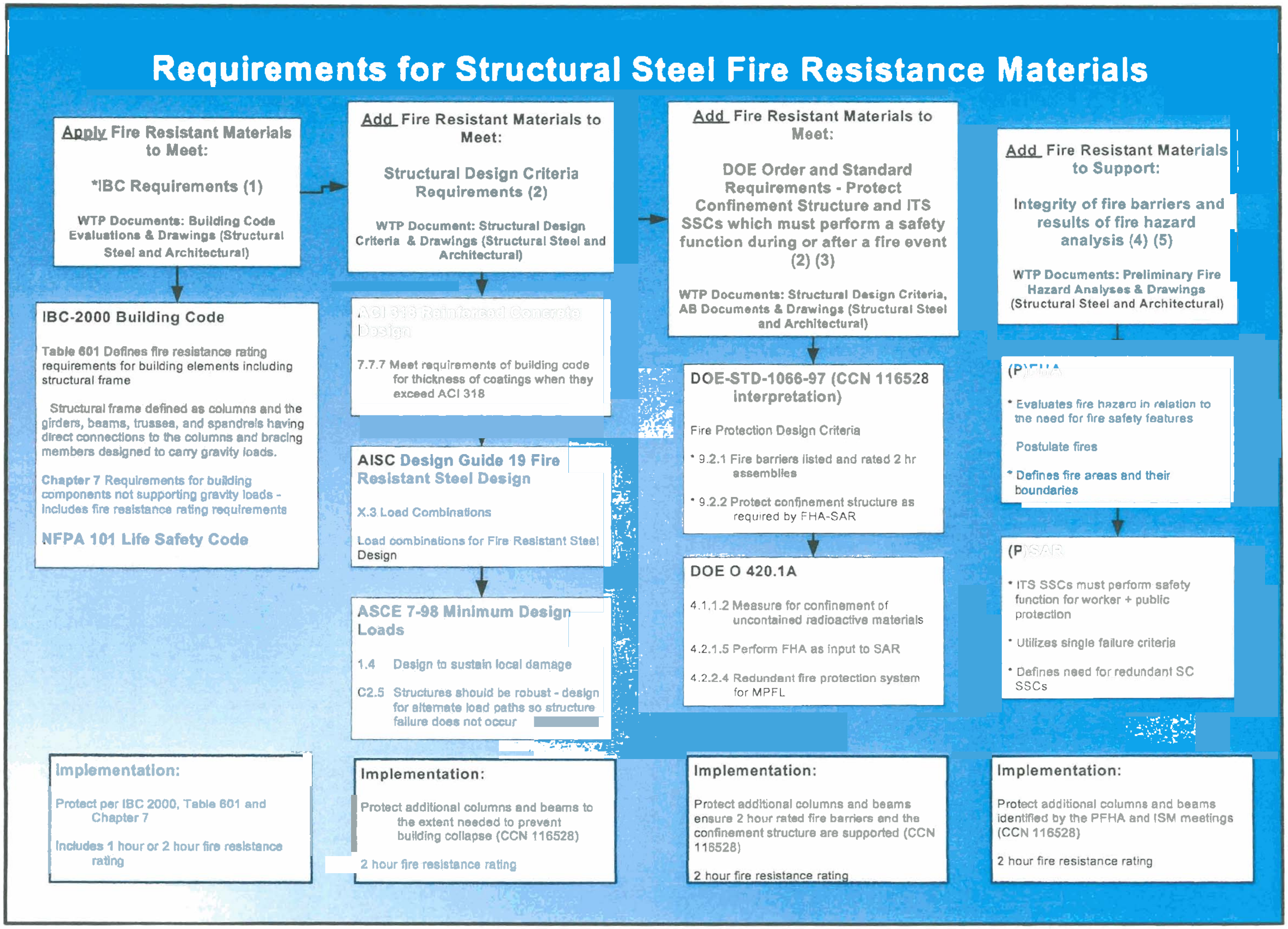

Requirements for Structural Steel Fire Resistance Materials
Apply Fire Resistant Materials to Meet:
*IBC Requirements (1)
WTP Documents: Building Code Evaluations & Drawings (Structural Steel and Architectural)
Add Fire Resistant Materials to Meet:
Structural Design Criteria Requirements (2)
WTP Document: Structural Design Criteria & Drawings (Structural Steel and Architectural)
Add Fire Resistant Materials to Meet:
DOE Order and Standard Requirements - Protect Confinement Structure and ITS SSCs which must perform a safety function during or after a fire event (2) (3)
WTP Documents: Structural Design Criteria, AB Documents & Drawings (Structural Steel and Architectural)
Add Fire Resistant Materials to Support:
Integrity of fire barriers and results of fire hazard analysis (4) (5)
WTP Documents: Preliminary Fire Hazard Analyses & Drawings (Structural Steel and Architectural)
IBC-2000 Building Code
Table 601 Defines fire resistance rating requirements for building elements including structural frame
Structural frame defined as columns and the girders, beams, trusses, and spandrels having direct connections to the columns and bracing members designed to carry gravity loads.
Chapter 7 Requirements for building components not supporting gravity loads - includes fire resistance rating requirements
NFPA 101 Life Safety Code
ACI 318 Reinforced Concrete Design
7.7.7 Meet requirements of building code for thickness of coatings when they exceed ACI 318
AISC Design Guide 19 Fire Resistant Steel Design
X.3 Load Combinations
Load combinations for Fire Resistant Steel Design
ASCE 7-98 Minimum Design Loads
1.4 Design to sustain local damage
C2.5 Structures should be robust - design for alternate load paths so structure failure does not occur
DOE-STD-1066-97 (CCN 116528 interpretation)
Fire Protection Design Criteria
* 9.2.1 Fire barriers listed and rated 2 hr assemblies
* 9.2.2 Protect confinement structure as required by FHA-SAR
DOE O 420.1A
4.1.1.2 Measure for confinement of uncontained radioactive materials
4.2.1.5 Perform FHA as input to SAR
4.2.2.4 Redundant fire protection system for MPFL
(P)FHA
* Evaluates fire hazard in relation to the need for fire safety features
Postulate fires
* Defines fire areas and their boundaries
(P)SAR
* ITS SSCs must perform safety function for worker + public protection
* Utilizes single failure criteria
* Defines need for redundant SC SSCs
Implementation:
Protect per IBC 2000, Table 601 and Chapter 7
Includes 1 hour or 2 hour fire resistance rating
Implementation:
Protect additional columns and beams to the extent needed to prevent building collapse (CCN 116528)
2 hour fire resistance rating
Implementation:
Protect additional columns and beams ensure 2 hour rated fire barriers and the confinement structure are supported (CCN 116528)
2 hour fire resistance rating
Implementation:
Protect additional columns and beams identified by the PFHA and ISM meetings (CCN 116528)
2 hour fire resistance rating

Table B-1 IBC Table 601 - Fire-Resistance Rating Requirements for Building Elements (Hours)

Building Element	TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
	A	B	A^d	B	A^d	B	HT	A^d	B
Structural Frame[a] Including columns, girders, trusses	3^b	2^b	1	0	1	0	HT	1	0
Bearing Walls Exterior[f] Interior	3 3^b	2 2^b	1 1	0 0	2 1	2 0	2 1/HT	1 1	0 0
Nonbearing walls and partitions Exterior Interior[e]	See Table 602 See Section 602								
Floor Construction Including supporting beams and joists	2	2	1	0	1	0	HT	1	0
Roof Construction Including supporting beams and joists	$1\frac{1}{2}^c$	1^c	1^c	0^c	1^c	0	HT	1^c	0

a The structural frame shall be considered to be the columns and the girders, beams, trusses, and spandrels having direct connections to the columns and bracing members designed to carry gravity loads. The members of floor or roof panels which have no connection to the columns shall be considered secondary members and not a part of the structural frame.

b Roof supports: Fire resistance rating of structural frame and bearing walls are permitted to be reduced by 1 hour where supporting a roof only.

c 1 Except in Factory-Industrial (F-1), Hazardous (H), Mercantile (M), and Moderate Hazard Storage (S-1) occupancies, fire protection of structural members shall not be required, including protection of roof framing and decking where every part of the roof construction is 20 feet or more above any floor immediately below. Fire-retardant-treated wood members shall be allowed to be used for such unprotected members.

2 In all occupancies, heavy timber shall be allowed where a 1-hour or less fire-resistance rating is required

3 In Type I and Type II construction, fire-retardant-treated wood shall be allowed in buildings not over two stories including girders and trusses as part of the roof construction.

d An approved automatic sprinkler system in accordance with Section 903.3.1.1 shall be allowed to be substituted for 1-hour fire-resistance-rated construction, provided such system is not otherwise required by other provisions of the code or used for an allowable area in accordance with Section 506.3 or an allowable height increase in accordance with Section 504.2. The 1-hour substitution for the fir resistance of exterior walls shall not be permitted

e For interior nonbearing partitions in Type IV construction, also see Section 602.4.6

f Not less than the fire-resistance rating based on fire separation distance (see Table 602.)

Project Structural Design Criteria

The Project *Structural Design Criteria* (SDC), 24590-WTP-DC-ST-01-001, is used to reduce the amount of steel requiring fire coating or cladding in the LAW and HLW facilities and the Lab. The SDC demonstrates, through the use of calculations, that individual, non-fire coated or clad structural members can lose strength and does not result in the structural collapse of the facility. Through use of this criterion, it has been demonstrated that many of the beams in the LAW and HLW facilities and the Lab are not required to be fire coated or clad to ensure the structural integrity of the facility during a fire event, while still meeting the requirements of DOE-STD-1066-97.

PFHA, PSAR and ISM

PFHAs for each facility have been developed to determine the impacts of credible fire scenarios on the WTP facilities. The evaluations of fire scenarios in Integrated Safety Management (ISM) meeting have resulted in some additional fire coating and cladding requirements being added to the PFHA and the PSAR to ensure protection of important to safety (ITS) equipment housed in the facilities.

Building Specific Evaluation of Steel Requiring Fire Coating or Cladding

Low-Activity Waste Facility (LAW)

IBC 2000 - The LAW building is designated a Type II B building for purposes of determining the fire resistance rating requirements of building elements. Per Table 601 of IBC 2000, the structural frame including columns, girders and trusses, do not require fire coating and cladding.

DOE Orders and Standards and Project Structural Design Criteria - The WTP Project has interpreted the requirements of DOE-STD-1066-97 and direction from ORP to mean that the steel in LAW must be fire fireproofed or clad such that loss of strength of non-fireproofed members (column, beam, or girder), as determined by the Project Structural Design Criteria:

- Does not result in the structural collapse of the facility during credible fire scenarios
- The structural shell surrounding critical areas has a fire resistance rating of at least 2 hours
- The project provides adequate protection of structural steel in non-critical areas to ensure that failure of steel in the non-critical areas does not affect the structural shell of adjacent critical areas. This requirement has been further interpreted by the AHJ to require 2-hr fire coating or cladding of the columns and selected girders supporting the LAW roof.

CCN 143342, "LAW Fireproofing Methodology," provides details on how structural steel was selected to be fire coated or clad based on the above criteria.

PFHA, PSAR, and ISM - ISM meetings, documented by meeting minutes CCN 150221, "LAW ISM: Fireproofing of Structural Steel," were held to evaluate the fire scenarios in the PFHA and PSAR and determine if additional fire coating or cladding would be required to ensure protection of ITS equipment. As a result of these meetings, a limited amount of additional fire coating or cladding was assigned.

High-Level Waste (HLW) Facility

IBC 2000 - Similar to the LAW building, the HLW building is designated a Type II B building for purposes of determining the fire resistance rating requirements of building elements. Per Table 601 of IBC 2000, the structural frame including columns, girders and trusses, do not require fire coating or cladding.

DOE Orders and Standards and Project Structural Design Criteria - As with LAW, additional requirements for HLW with relation to fire coating or cladding are imposed by DOE-STD-1066-97, in Section 9.2.2, as further defined by the Office of River Protection. The WTP Project has interpreted the requirements of DOE-STD-1066-97 and direction from ORP to mean that the steel in HLW must be fire coated or clad such that loss of strength of non-fireproofed members (column, beam, or girder) as determined by the Project Structural Design Criteria:

- Does not result in the structural collapse of the facility during credible fire scenarios

- The structural shell surrounding critical areas has a fire resistance rating of at least 2 hours
- The project provides adequate protection of structural steel in non-critical areas to ensure that failure of steel in the non-critical areas does not affect the structural shell of adjacent critical areas. This requirement has been further interpreted by the AHJ to require 2-hr fire coating or cladding of the columns and selected girders supporting the HLW roof.

Details on how structural steel is selected to be fire coated or clad are developed by Civil Structural and Architectural (CS&A) based on IBC 2000, DOE orders and standards and Project structural design criteria.

PFHA, PSAR, and ISM - ISM meetings, documented by meeting minutes CCN 116895, "ISM III - Identification of High-Level Waste Secondary Structural Steel Fireproofing," were held to evaluate the fire scenarios in the PFHA and PSAR and determine if additional fire coating or cladding would be required to ensure protection of ITS equipment. As a result of these meetings, a limited amount of additional fire coating or cladding was assigned

Laboratory (Lab)

IBC 2000- Similar to the LAW and HLW buildings, the Lab building is designated a Type II B building for purposes of determining the fire resistance rating requirements of building elements. Per Table 601 of IBC 2000, the structural frame including columns, girders and trusses, do not require fire coating or cladding.

DOE Orders and Standards and Project Structural Design Criteria - As with LAW and HLW, additional requirements for Lab with relation to fire coating or cladding are imposed by DOE-STD-1066-97, in Section 9.2.2, as further defined by ORP. The WTP Project has interpreted the requirements of DOE-STD-1066-97 and direction from ORP to mean that the steel in Lab must be fire coated or clad such that loss of strength of non-fireproofed members (column, beam, or girder) as determined by the Project Structural Design Criteria:

- Does not result in the structural collapse of the facility during credible fire scenarios
- The structural shell surrounding critical areas has a fire resistance rating of at least 2 hours
- The project provides adequate protection of structural steel in non-critical areas to ensure that failure of steel in the non-critical areas does not affect the structural shell of adjacent critical areas. This requirement has been further interpreted by the AHJ to require 2-hr fire coating or cladding of selected columns and girders supporting the Lab roof.

CCN 143348, "LAB Fireproofing Methodology," provides details on how structural steel was selected to be fire coated or clad based on the above criteria.

PFHA, PSAR and ISM - ISM meetings, documented by Meeting Minutes CCN 108487, "ISM III - Steel Fireproofing in the Analytical Laboratory," were held to evaluate the fire scenarios in the PFHA and PSAR and determine if additional fire coating or cladding would be required to ensure protection of ITS equipment. A limited amount of additional fire coating or cladding was assigned as a result of these meetings.

Pretreatment Facility (PTF)

IBC 2000 -The PTF has been designated as a Type I B, H 4 Occupancy. Per Table 601 of IBC 2000, the structural frame including columns, beams, girders and trusses require 2-hour fire coating or cladding and the roof and purlins require 1-hr fire protection.

DOE Orders and Standards and Project Structural Design Criteria - DOE-STD-1066-97, as interpreted by the AHJ, imposes 2-hr fire coating or cladding requirements on the columns and girders supporting the purlins that support the PTF roof.

Details on how structural steel is selected to be fire coated are developed by CS&A based on IBC 2000, DOE orders and standards and Project structural design criteria.

PFHA and PSAR - ISM meetings, documented by meeting minutes CCN 119707, "ISM Fireproofing of Structural Steel Within PTF," were held to evaluate the fire scenarios in the PFHA and PSAR and determine if additional fire coating or cladding would be required to ensure protection of ITS equipment. No additional fire coating or cladding was assigned as a result of these meetings for PTF.

Appendix C

LAW Fire Areas

Figure C-1 Floor Plan Elevation -21 Ft - Structural Framing Supporting Elevation 3 Ft

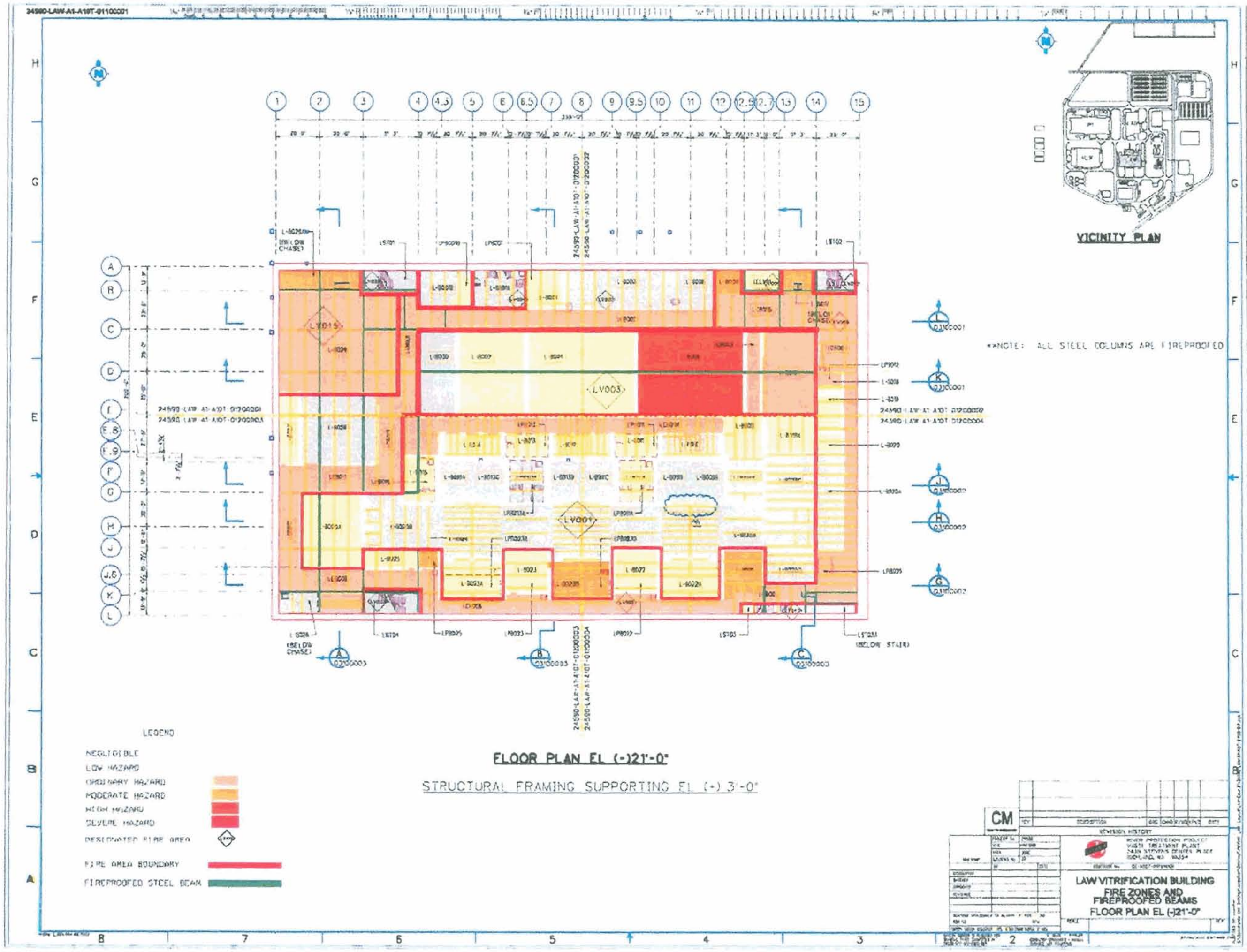


Figure C-2 Floor Plan Elevation 0 ft and Elevation 3 Ft - Structural Framing Supporting Elevation 28 Ft

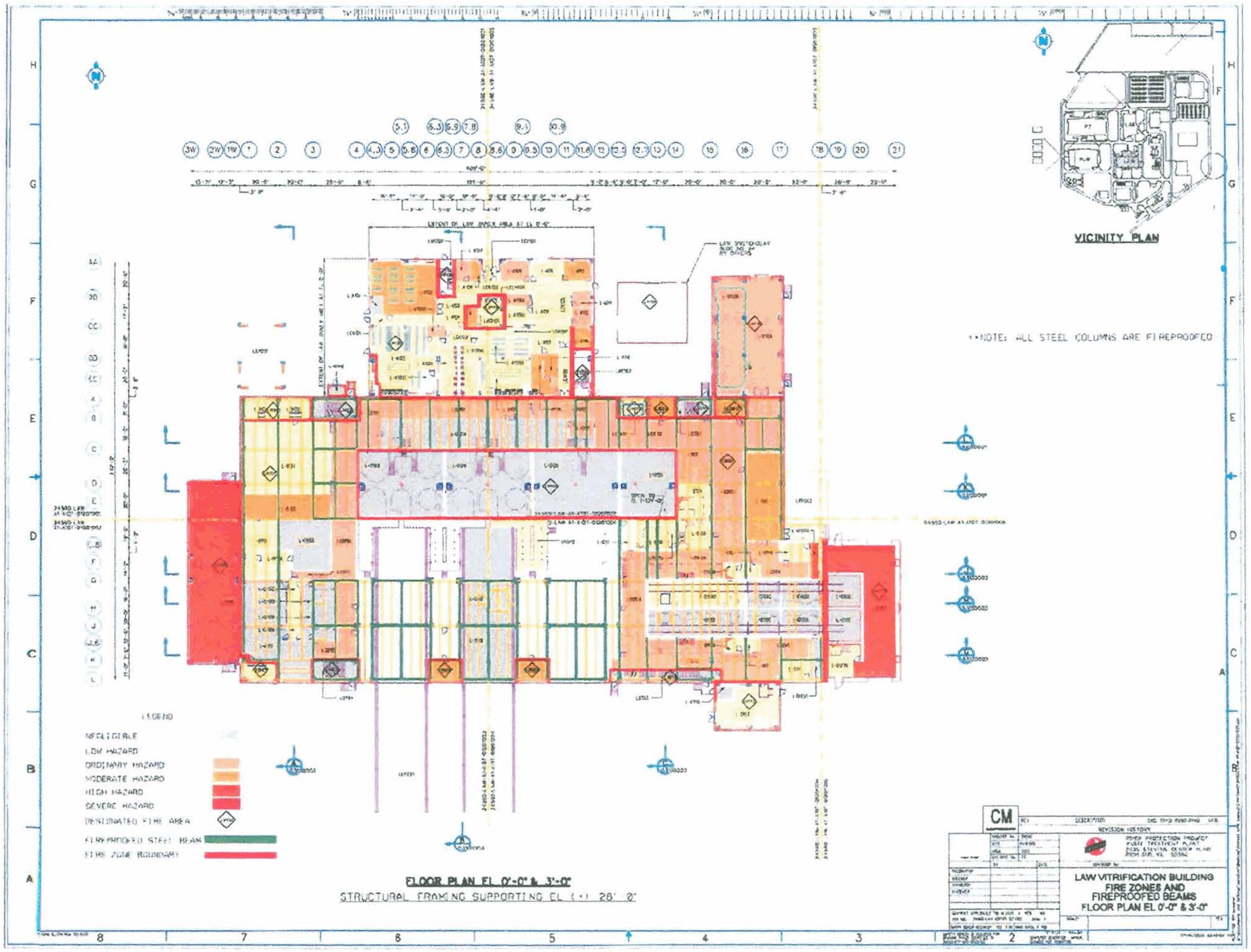


Figure C-3 Floor Plan Elevation 15 Ft and Elevation 22 Ft - Structural Framing Supporting Elevation 28 Ft

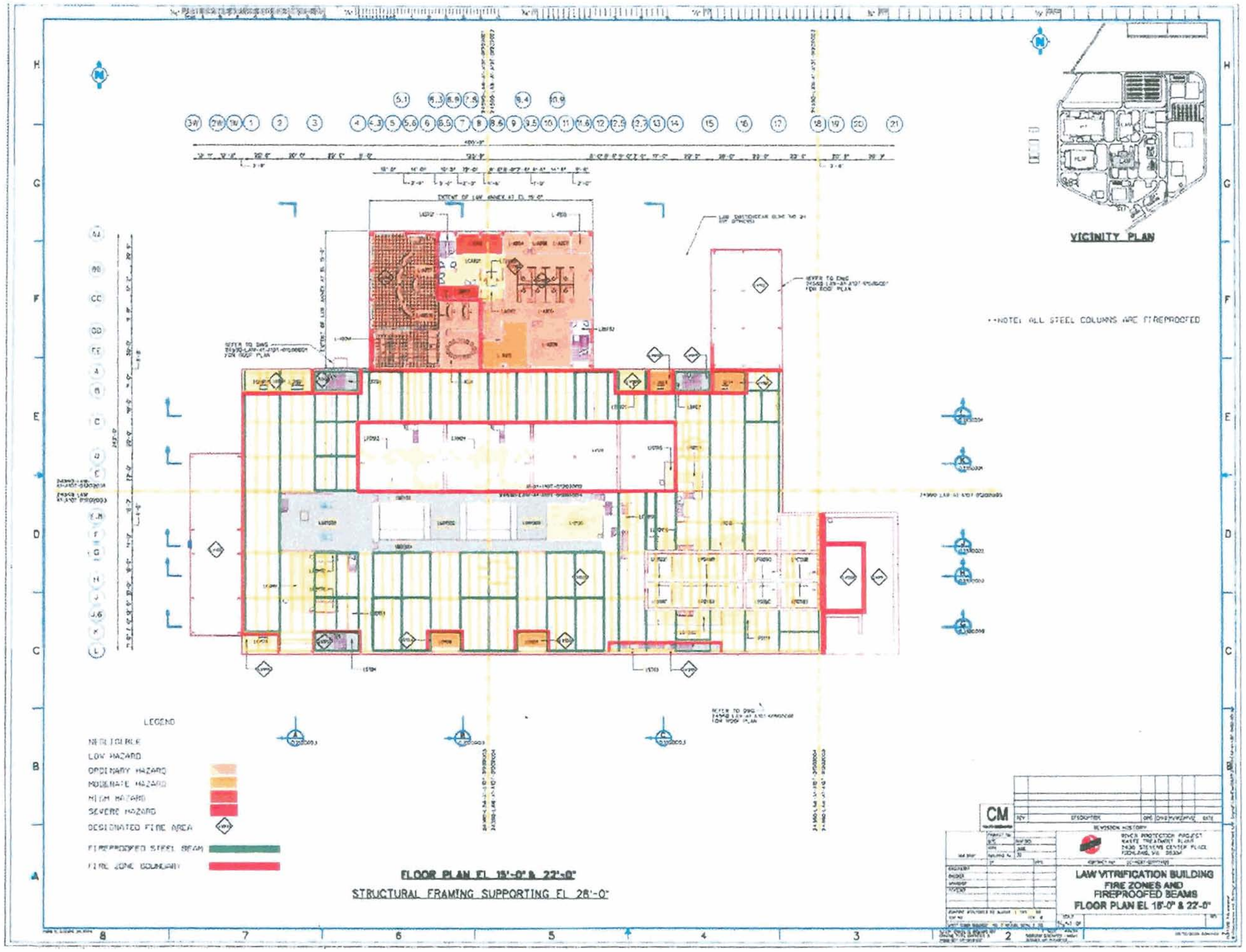


Figure C-4 Floor Plan Elevation 28 Ft - Structural Framing Supporting Elevation 48 Ft

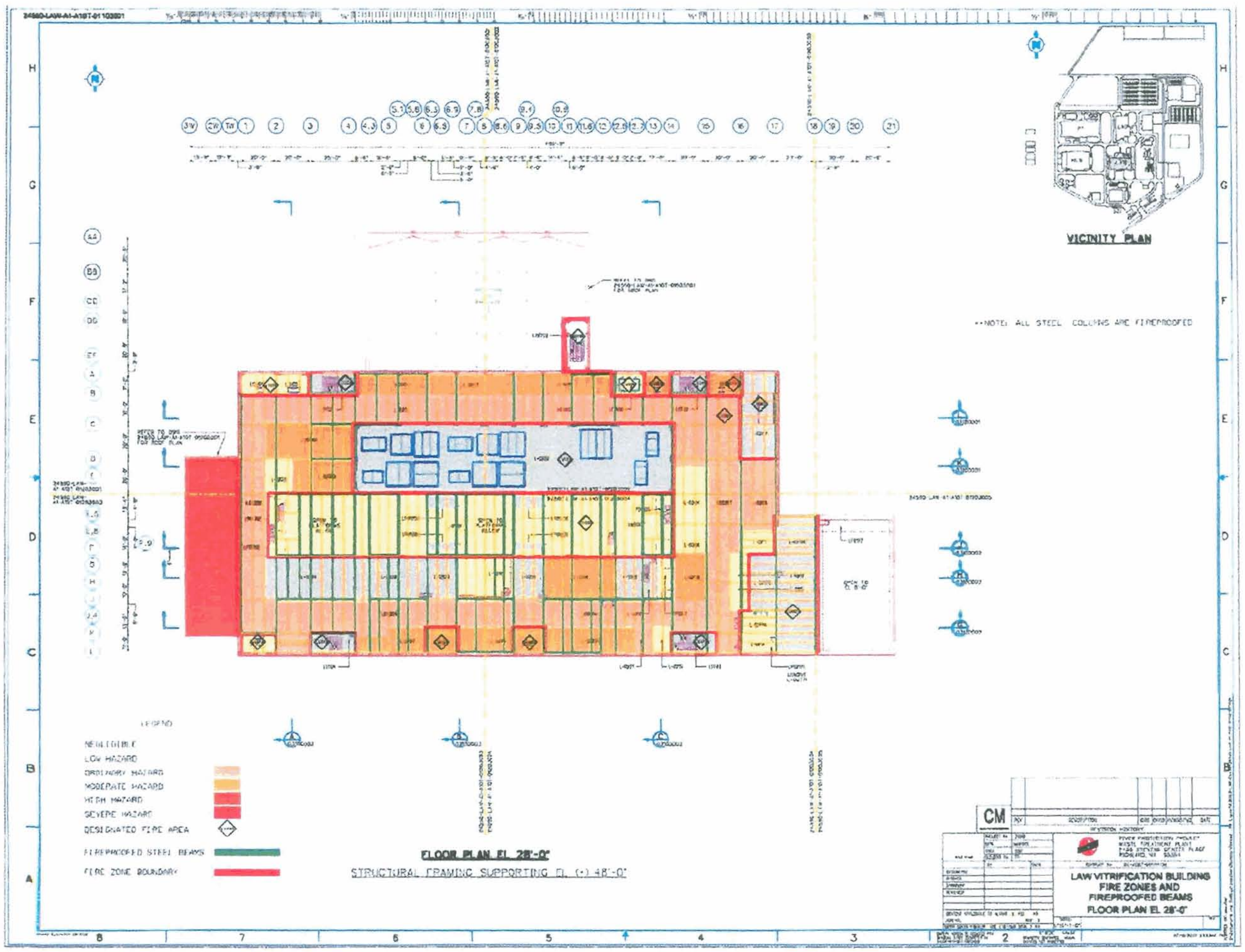
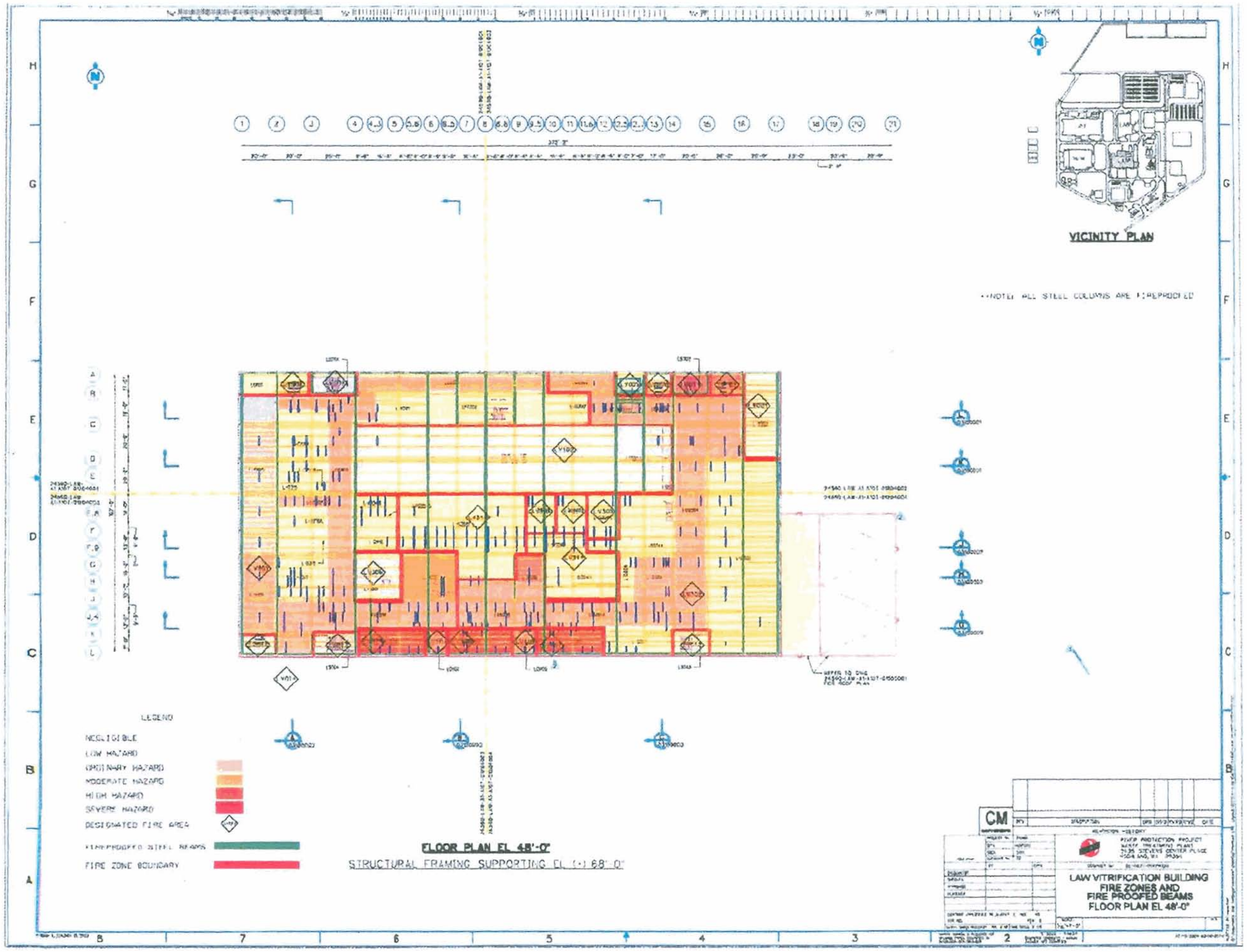


Figure C-5 Floor Plan Elevation 48 Ft - Structural Framing Supporting Elevation 58 Ft



Appendix D

AISC 360-05 Appendix 4 Application

WTP Application of Code Provisions

Code Provision

4.1.	**GENERAL PROVISIONS**
	The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.
	The appendix uses the following terms in addition to the terms in the Glossary.
	Active fire protection: Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.
	Compartmentation: The enclosure of a building space with elements that have a specific fire endurance.
	Convective heat transfer: The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.
	Design-basis fire: A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.
	Elevated temperatures: Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.
	Fire: Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.
	Fire barrier: Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.

Code Commentary

4.1. GENERAL PROVISIONS

Appendix 4 provides structural engineers with guidance in designing steel-framed building systems and components, including columns, and floor and truss assemblies, for fire conditions. Compliance with the performance objective in Section 4.1.1 can be demonstrated by either structural analysis or component qualification testing.

Thermal expansion and progressive decrease in strength and stiffness are the primary structural responses to elevated temperatures that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at elevated temperatures can lead to a more robust structural design for fire conditions.

Glossary

Terms pertinent to the design of structural components and systems for fire conditions are presented in the glossary. Terms in common with those in other fire-resistant design documents developed by the SFPE, ICC, NFPA, ASTM and similar organizations are defined in a manner consistent with those documents.

WTP Application

Code

The WTP is generally following the structural analysis approach except WTP does not have area specific design basis fires. Accordingly, WTP uses an ASTM E-119 standard fire for analysis. WTP believes this fire case is bounding and also would be an extreme abnormal occurrence.

The current Authorization Basis prohibits use of Performance Based Design (Glossary Definition) and requires use of Prescriptive Design (Glossary Definition).

Commentary

This section references "compliance with performance objectives in Section 4.1.1". This approach is not allowed under the current Authorization Basis. See CCN-097398 Applying to this restriction.

WTP evaluations and approaches are consistent with the second paragraph; this will be addressed section by section.

Code Provision

4.1.1. Performance Objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure.

Within the compartment of fire origin, *forces* and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

Code Commentary

4.1.1. Performance Objective

The performance objective underlying the provisions and guidelines in this Specification is that of life safety. Fire safety levels should depend on the building occupancy, height of building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting. Three limit states exist for elements service as fire barriers (compartment walls and floors): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three limit states are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing limit state is loss of load-bearing capacity.

Specific performance objectives for a facility are determined by the stakeholders in the building process, within the context of the above general performance objective and limit states. In some instances, applicable building codes may stipulate that steel in buildings of certain occupancies and heights be protected by fire-resistant materials or assemblies to achieve specified performance goals.

WTP Application

Section 4.1.1

Code

The current WTP fire hazard analysis has prescribed fire areas or areas and fire barriers which have been directed to be fireproofed to meet codes and regulations. Localized evaluations of primary members for LAW, PT, HLW and the LAB supporting the fire barriers have been conducted.

Commentary

WTP fire barrier designs comply with life safety and nuclear safety requirements.

WTP does consider the "elements that are not part of a separating element, the governing limit state is a loss of load bearing capacity." This is the purpose of localized analysis, where the secondary beams are rendered ineffective, but their weights were included.

Code Provision

4.1.2 Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the building code.

Code Commentary

None

WTP Application

4.1.2. Design by Engineering Analysis Comments

Anticipated performance of steel framing has been evaluated when subjected to the ASTM standard fire. Design basis fires have not been determined. The demonstration of equivalency has been restricted by the authorization basis.

Code Provision

> **4.1.3. Design by Qualification Testing**
>
> The qualification testing methods in Section 4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building codes.

Code Commentary

None

WTP Application

4.1.3. Design by Qualification Testing Comments

WTP does not use this approach.

Code Provision

> **4.1.4. Load Combinations and Required Strength**
>
> The *required strength* of the structure and its elements shall be determined from the following *gravity load combination*:
>
> $$[0.9 \text{ or } 1.2]D + T + 0.5L + 0.2S$$
>
> where
>
> D = nominal dead load
>
> L = nominal occupancy live load
>
> S = nominal show load
>
> T = nominal *forces* and deformations due to the design-basis fire defined in Section 4.2.1
>
> A lateral *notional load*, $N_i = 0.002Y_i$, as defined in Appendix 7.2, where N_i = notional *lateral load* applied at framing level i and Y_i = *gravity load* from combination A-4-1 acting on framing level i, shall be applied is combination with the loads stipulated in Equation A-4-1. Unless otherwise stipulated by the *authority having jurisdiction*, D, L and S shall be the *nominal loads* specified in ASCE 7.

Code Commentary

4.1.4. Load Combinations and Required Strength

Fire safety measures are aimed at three levels: (1) to prevent the outbreak of fires through elimination of ignition sources or hazardous practices; (2) to prevent uncontrolled fire development and flashover through early detection and suppression; and (3) to prevent loss of life or structural collapse through fire protection systems, compartmentation, exit ways, and provision of general structural integrity and other passive measures. Specific structural design provisions to check structural integrity and risk of progressive failure due to severe fires can be developed from principles of structural reliability theory (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991).

The limit state probability of failure due to fire can be written as

$$P(F) = P(F \backslash D,I)P(D \backslash I)P(I) \tag{C-A-4-1-1}$$

where $P[I]$ = probability of ignition, $P[D \backslash I]$ = probability of development of a structurally significant fire, and $P[F \backslash D,I]$ = probability of failure, given the occurrence of the two preceding events. Measures taken to reduce $P(I)$ and $P(D/I)$ are mainly nonstructural in nature. Measures taken by the structural engineer to design fire resistance into the structure impact $P(F \backslash D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ in Equation C-A-4-1-1. Analysis of reliability of structural systems for gravity dead and live load (Galambos, Ellingwood, MacGregor, and Cornell, 1982) suggests that the limit state probability of individual steel members and connections is on the order of 10-5 to 10-4/year. For redundant steel frame systems, $P(F)$ is on the order of 10-6 to 10-5. The *de minimis* risk, that is, the level below which the risk is of regulatory or legal concern and the economic or social benefits of risk reduction are small, is in the order of 10-7 to 10-6/year (pate-Cornell, 1994). If $P(I)$ is on the order of 10-4/year for typical buildings and $P(D \backslash I)$ is on the order of 10-2 for office or commercial buildings in urban areas with suppression systems or other protective measures, then $P(F \backslash D,I)$ should be approximately 0.1 to ascertain that the risk due to structural failure caused by fire is socially acceptable.

The use of first-order structural reliability analysis based on this target (conditional) limit state probability leads to the gravity load combination presented as Equation A-4-1. Load combination Equation A-4-1 is the same as Equation C2-3 that appears in Commentary C2.5 of SEI/ASCE 7 (ASCE, 2002), where the probabilistic bases for load combinations for extraordinary events is explained in detail. The factor 0.9 is applied to the dead load when the effect of the dead load is to stabilize the structure; otherwise the factor 1.2 is applied. The companion action load factors on L and S in that equation reflect the fact that the probability of a coincidence of the peak time-varying load with the occurrence of a fire is negligible (Ellingwood and Corotis, 1991).

Commentary C2.5 of ASCE (2002) contains a second equation that includes 0.2W. That equation is provided so that the stability of the system is checked. The same purpose is accomplished by requiring that the frame be checked under the effect of a small notional lateral load equal to 0.2 percent of story gravity force, acting in combination with the gravity loads. The required strength of the structural component or system designed using these load combinations is on the order of 60 percent to 70 percent of the required strength under full gravity or wind load at normal temperature.

WTP Application

4.1.4. Load Combinations and Required Strength Comments

Section 4.1.4.

Our current revision of the Structural Design Criteria provides fire load cases in accordance with the following: The additional load combinations (for fire events) shall be in accordance with ASCE-7 Section 2.5 (Ref. 2.1.9) and the AISC Design Guide 19 (Ref. 2.1.16).

The evaluation also includes the combinations in this appendix. The area selected is interior, with the exception of the columns, snow load is not applicable.

Code Provision

> **4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS**
>
> It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.
>
> **4.2.1. Design-Basis Fire**
>
> A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel *load* density based on the occupancy of the space shall be considered when determining the total fuel *load*. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.
>
> When the analysis methods in Section 4.2 are used to demonstrate an equivalency as an alternative material or method as permitted by a building code, the design-basis fire shall be determined in accordance with ASTM E119.

Code Commentary

4.2.1. Design-Basis Fire

Once a fuel load has been agreed upon for the occupancy, the designed should demonstrate the effect of various fires on the structure by assessing the temperature-time relationships for various ventilation factors. These relations may result in different structural responses, and it is useful to demonstrate the capability of the structure to withstand such exposures. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive. Based on these results, connections and edge details can be specified to provide a structure that is sufficiently robust.

WTP Application

4.2.1. Design-Basis Fire Comments

Current preliminary fire hazards analysis have not developed a compartment by compartment temperature profile. Therefore, WTP defaults to using the ASTM E-119 standard fire. Using the maximum temperature and assume the materials reach the 2hr test limits. This is extremely conservative. This assumption forces WTP to consider the fire-proofed members heating up to test temperatures, which degrades them similarly to the non fire- proofed members, just later in the fire. Based on the ASTM fire, at temperatures below 800 °F significant deformation has occurred, regardless of fireproofing.

Section 7.2 of the report provides a detailed description of the ASTM E-119 fire.

Code Provision

> **4.2.1.1. Localized Fire**
>
> Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

Code Commentary

4.2.1.1. Localized Fire

Localized fires may occur in large open spaces, such as the pedestrian area of covered malls, concourses of airport terminals, warehouses, and factories, where fuel packages are separated by large aisles or open spaces. In such cases, the radiant heat flux can be estimated by a point source approximation, requiring the heat release rate of the fire and separation distance between the center of the fuel package and the closest surface of the steelwork. The heat release rate can be determined from experimental results or may be estimated if the mass loss rate per unit floor area occupied by the fuel is known. Otherwise, a stead-state fire may be assumed.

WTP Application

Assuming the ASTM fire and also assuming a flashover type of event, regardless of the actual fire potential. This is a very conservative approach, based on the type and use of these facilities.

Code Provision

> **4.2.1.2. Post-Flashover Compartment Fires**
>
> Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel *load*, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

Code Commentary

4.2.1.2. Post-Flashover Compartment Fires

Caution should be exercised when determining temperature-time profiles for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with an open (or exposed) floor area in excess of 5,000 ft^2 (465 m^2). In such cases, it is unlikely that all combustibles will burn in the space simultaneously. Instead, burning will be most intense in, or perhaps limited to, the combustibles nearest to a ventilation source. For modest-sized compartments with low aspect ratios, the temperature history of the design fire can be determined by algebraic equations or computer models, such as those described in the SFPE *Handbook for Fire Protection Engineering* (SFPE, 2002).

WTP Application

4.2.1.2. Post-Flashover Compartment Fires Comments

The WTP approach utilizing the ASTM E-119 fire exposure is conservative because it exceeds a post flashover temperature exposure described by the AISC requirement.

Code Provision

> **4.2.1.3. Exterior Fires**
>
> The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 4.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

Code Commentary

4.2.1.3. Exterior Fires

A design guide is available for determining the exposure resulting from an exterior fire (AISI, 1979).

WTP Application

4.2.1.3. Exterior Fires Comments

Not used

Code Provision

> **4.2.1.4. Fire Duration**
>
> The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel *load* available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Section 4.2.1.2.

Code Commentary

4.2.1.4. Fire Duration

Caution should be exercised when determining the fire duration for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with a floor area in excess of 5,000 ft^2 (465 m^2). The principal difficulty lies in obtaining a realistic estimate for the mass loss rate, given that all combustibles within the space may not be burning simultaneously. Failure to recognize uneven burning will result in an overestimation of the mass burning rate and an underestimation of the fire duration by a significant margin. Note: some computation methods may implicitly determine the duration of the fire, in which case the calculation of mass loss rate is unnecessary.

Where a parametric curve is used to define a post-flashover fire, the duration is determined by means of the fuel versus ventilation provisions, not explicitly by loss of mass. This cause should not limit the use of temperature-time relationships to those where duration is calculated, as stated above, as these tend to be localized fires and external fire.

WTP Application

4.2.1.4. Fire Duration Comments

Fire is not based on combustible loading, rather the WTP Project utilizes a reasonably conservative fire duration of 2 hours and associated temperature following the ASTM E-119 standard time temperature curve.

Code Provision

4.2.1.5. Active Fire Protection Systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

Code Commentary

4.2.1.5. Active Fire Protection Systems

Due consideration should be given to the reliability and effectiveness of active fire protection systems when describing the design-basis fire. When an automatic sprinkler system is installed, the total fuel load may be reduced by up to 60 percent (Eurocode 1, 1991). The maximum reduction in the fuel load should be considered only when the automatic sprinkler system is considered to be of the highest reliability, for example, reliable and adequate water supply, supervision of control valves, regular schedule for maintenance of the automatic sprinkler system developed in accordance with NFPA (2002), or alterations of the automatic sprinkler system are considered any time alterations for the space are considered.

For spaces with automatic smoke and heat vents, computer models are available to determine the smoke temperature (SFPE, 2002). Reduction in the temperature profile as a result of smoke and heat vents should only be considered for reliable installations of smoke and heat vents. As such, a regular maintenance schedule for the vents needs to be established in accordance with NFPA (2002a).

WTP Application

4.2.1.5. Active Fire Protection Systems Comments

For purposes of structural analyses unmitigated fires (e.g., non-sprinklered) represented by an ASTM E-119 exposure are considered reasonably conservative because under an ASTM E-119 fire exposure WTP primary fireproofed structural steel members maintain sufficient capacity, after additional loads caused by non-fireproofed members, to support the facility during and after a fire plastic deformation of the primary fireproofed steel members subjected to a catastrophic fire, will occur whether the secondary members are fireproofed or not. While fire sprinklers and other defense in depth fire protection features are installed throughout the WTP facilities, these defense in depth features are not relied upon in the safety basis to support the facility during and after a fire or to prevent plastic deformation.

Code Provision

4.2.2. Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

Table A-4.2.1

Properties of Steel at Elevated Temperatures

Steel Temperature (°F)[°C]	$k_E = E_m/E$	$k_Y = F_{ym}/F_y$	$k_u = F_{um}/F_y$
68 [20]	*	*	*
200 [93]	1.00	*	*
400 [204]	0.90	*	*
600 [316]	0.78	*	*
750 [399]	0.70	1.00	1.00
800 [427]	0.67	0.94	0.94
1000 [538]	0.49	0.66	0.66
1200 [649]	0.22	0.35	0.35
1400 [760]	0.11	0.16	0.16
1600 [871]	0.07	0.07	0.07
1800 [982]	0.05	0.04	0.04
2000 [1093]	0.02	0.02	0.02
2200 [1204]	0.00	0.00	0.00

* Use ambient properties.

Code Commentary

4.2.2. Temperatures in Structural Systems under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses where the steel is assumed to be at uniform temperature to three-dimensional analyses. The uniform temperature assumption is appropriate in a "lumped heat capacity analysis" where a steel column, beam or truss element is uniformly heated along the entire length and around the entire perimeter of the exposed section and the protection system is uniform along the entire length and around the entire perimeter of the section. In cases with nonuniform heating or where different protection methods are used on different sides of the column, a one-dimensional analysis should be conducted for steel column assemblies. Two-dimensional analyses are appropriate for beams, bar joists or truss elements supporting floor or roof slabs.

Heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. This may be done in the lumped heat capacity analysis using an effective property value, determined at a temperature near the estimated mid-point of the temperature range expected to be experienced by that component over the duration of the exposure. In the one- and two-dimensional analyses, the variation in properties with temperature should be explicitly included.

The boundary conditions for the heat transfer analysis shall consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. The presence of fire resistive materials in the form of insulation, heat screens or other protective measures shall be taken into account, if appropriate.

Lumped Heat Capacity Analysis. This first-order analysis to predict the temperature rise of steel structural members can be conducted using algebraic equations iteratively. This approach assumes that the steel member has a uniform temperature, applicable to cases where the steel member is unprotected or uniformly protected (on all sides), and is exposed to fire around the entire perimeter of the assembly containing the steel member. Caution should be used when applying this method to steel beams supporting floor and roof slabs, as the approach will overestimate the temperature rise in the beam. In addition, where this analysis is used as input for the structural analysis of a fire-exposed, steel beam supporting a floor and roof slab, the thermally induced moments will not be simulated as a result of the uniform temperature assumption.

Unprotected Steel Members. The temperature rise in an unprotected steel section in a short time period shall be determined by

$$\Delta T_s = \frac{a}{c_s\left(\frac{W}{D}\right)}(T_F - T_s)\,\Delta t \qquad \text{(C-A-4-2-2)}$$

The heat transfer coefficient, a, is determined from

$$a = a_c + a_f \qquad \text{(C-A-4-2-2)}$$

where

a_c = convective heat transfer coefficient
a_f = radiative heat transfer coefficient, given as

$$a_f = \frac{5.67 \times 10^{-8}\varepsilon_F}{T_F - T_S}(T_F^4 - T_s^4)$$

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 25 W/m^2-°C. The parameter, ε_F, accounts for the emissivity of the fire and the view factor. Estimates for ε_F, are suggested in Table C-A-4-2.1.

Table C-A-4-2.1

Guidelines for Estimating ε_F

Type of Assembly	ε_F
Column, exposed on all sides	0.7
Floor beam: Imbedded in concrete floor slab, with only bottom flange of beam exposed to fire	0.5
Floor beam, with concrete slab resting on top flange of beam	
Flange width : beam depth ratio > 0.5	0.5
Flange width : beam depth ratio < 0.5	0.7
Box girder and lattice girder	0.7

For accuracy reasons, a maximum limit for the time step, Δ_t, is suggested as 5 sec.

The fire temperature needs to be determined based on the results of the design fire analysis. As alternatives, the standard time-temperature curves indicated in ASTM E119 (ASTM, 2000) for building fires or ASTM E1529 (ASTM, 2000a) for petrochemical fires may be selected.

Protected Steel Members. This method is most applicable for steel members with contour protection schemes, in other words, where the insulating or (protection) material follows the shape of the section. Application of this method for box protection methods will generally result in the temperature rise being overestimated. The approach assumes that the outside insulation temperature is approximately equal to the fire temperature. Alternatively, a more complex analysis may be conducted which determines the exterior insulation temperature from a heat transfer analysis between the assembly and the exposing fire environment.

If the thermal capacity of the insulation is much less than that for the steel, such that the following inequality is satisfied:

$$c_s W/D > 2 d_p \rho_p c_p \qquad \text{(C-A-4-2-3)}$$

Then, Equation C-A-4-2-4 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p \frac{W}{D}} (T_F - T_s) \Delta t \qquad \text{(C-A-4-2-4)}$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation C-A-4-2-3 is not satisfied), then Equation C-A-4-2-5 should be applied:

$$\Delta T_s = \frac{k_p}{d_p} \left[\frac{T_F - T_s}{c_s \frac{W}{D} + \frac{c_p \rho_p d_p}{2}} \right] \Delta t \qquad \text{(C-A-4-2-5)}$$

The maximum limit for the time step, Δ_t, should be 5 sec.

Ideally, material properties should be considered as a function of temperature. Alternatively, material properties may be evaluated at a mid-range temperature expected for that component. For protected steel members, the material properties may be evaluated at 300 °C, and for protection materials, a temperature of 500 °C may be considered.

External Steelwork. Temperature rise can be determined by applying the following equation:

$$\Delta T_s = \frac{q''}{c_s\left(\frac{W}{D}\right)}\Delta t \qquad \text{(C-A-4-2-6)}$$

where q'' is the net heat flux incident on the steel member.

Advanced Calculation Methods. The thermal response of steel members may be assessed by application of a computer model. A computer model for analyzing the thermal response of the steel members should consider the following:

- Exposure conditions established based on the definition of a design fire. The exposure conditions need to be stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux. The incident heat flux is dependent on the design fire scenario and the location of the structural assembly. The heat flux emitted by the fire or smoke can be determined from a fire hazard analysis. Exposure conditions are established based on the definition of a design fire. The exposure conditions are stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux.
- Temperature-dependent material properties.
- Temperature variation within the steel member and any protection components, especially where the exposure varies from side to side.

Nomenclature:

Am	surface area of a member per unit length, ft (m)
Ap	area of the inner surface of the fire protection material per unit length of the member, ft (m)
Ac	cross-sectional area, in.2 (m2)
D	heat perimeter, in. (m)
T	temperature, °F (°C)
V	volume of a member per unit length, in.2 (2)
W	weight (mass) per unit length, lb/ft (kg/m)
a	heat transfer coefficient, Btu/ft2·sec·°F (W/m2·°C
c	specific heat, Btu/lb·°F (J/kg·°C)
d	thickness, in. (m)
hnet,d	design value of the net heat flux per unit area, Btu/sec·ft2 (W/m2)
k	thermal conductivity, Btu/ft·sec·°F (W/m·°C)
l	length, ft (m)
t	time in fire exposure, seconds
Δt	time interval, seconds
p	density, lb/ft3 (kg/m3)

Subscripts:

a	steel
c	convection
m	member
p	fire protection material
r	radiation
s	steel
t	dependent on time
T	dependent on temperature

WTP Application

4.2.2. Temperatures in Structural Systems Under Fire Conditions Comments

Without a performance based fire, much of this section is unusable. However, one can degrade the materials based on Table 4.2.1. It is also recognized that the non-fireproofed members heat up earlier in the fire and evaluate different stages of the fire.

Code Provision

> **4.2.3. Material Strengths at Elevated Temperatures**
>
> Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a *yield strength* in excess of 65 ksi (448 MPa) or concretes with specified compression strength in excess of 8,000 psi (55 MPa).

Code Commentary

4.2.3. Material Strengths at Elevated Temperatures

The properties for steel and concrete at elevated temperatures are adopted from the ECCS *Model Code on Fire Engineering* (ECCS, 2001), with Section III.2, "Material Properties." These generic properties are consistent with those in Eurocodes 3 (Eurocode 3, 2002) and 4 (Eurocode 4, 2003), and reflect the consensus of the international fire engineering and research community. The background information for the mechanical properties of structural steel at elevated temperatures can be found in Cooke (1988) and Preston (1988).

WTP Application

4.2.3. Material Strengths at Elevated Temperatures Comments

The material degrades as the temperature increases.

Code Provision

> 4.2.4 Structural Design Requirements (no direct text)

Code Commentary

4.2.4 Structural Design Requirements

The resistance of the structural system in the design basis fire may be determined by:

(a) Structural analysis of individual elements where the effects of restraint to thermal expansion and bowing may be ignored but the reduction in strength and stiffness with increasing temperature is incorporated.
(b) Structural analysis of assemblies/subframes where the effects of restrained thermal expansion and thermal bowing are considered by incorporating geometric and material nonlinearities.
(c) Global structural analysis where restrained thermal expansion, thermal bowing, material degradation and geometric nonlinearity are considered.

WTP Application

4.2.4 Structural Design Requirements Comments

The approach selected for evaluating the expanding members is based on the three listed approaches.

This is accomplished by starting with an individual member and expanding outward case-by-case to consider a global area. This global area includes a typical bay with columns, girders and secondary members.

Code Provision

4.2.4.1 General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The *structural system* shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous *load* paths shall be provided to transfer all *forces* from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

Code Commentary

4.2.4.1 General Structural Integrity

The requirement for general structural integrity is consistent with that appearing in Section 1.4 of ASCE (2002). Structural integrity is the ability of the structural system to absorb and contain local damage or failure without developing into a progressive collapse that involves the entire structure or a disproportionately large part of it.

The Commentary C1.4 to Section 1.4 of ASCE (2002) contains guidelines for the provision of general structural integrity. Compartmentalization (subdivision of building/stories in a building) is an effective means of achieving resistance to progressive collapse as well as preventing fire spread, as a cellular arrangement of structural components that are well tied together provides stability and integrity to the structural system as well as insulation.

WTP Application

4.2.4.1. General Structural Integrity Comments

WTP has been designed by taking advantage of compartmentalization. Calculations show that adequate bracing has been provided in the non-impacted fire areas to meet the intent of this section.

Code Provision

> **4.2.4.2 Strength Requirements and Deformation Limits**
>
> Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.
>
> Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.
>
> *Connections* shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

Code Commentary

4.2.4.2 Strength Requirements and Deformation Limits

As structural elements are heated, their expansion is restrained by adjacent element and connections. Material properties degrade with increasing temperature. Load transfer can occur from hotter elements to adjacent cooler elements. Excessive deformation may be of benefit in a fire as it allows release of thermally induced stresses. Deformation is acceptable once horizontal and vertical separation as well as the overall load bearing capacity of the structural system is maintained.

WTP Application

4.2.4.2. Strength Requirements and Deformation Limits Comments

Finite element models have been developed and have evaluated the forces, stresses, and deformation for a variety of cases and temperatures. The code states that deformation (even excessive) is acceptable once the overall load bearing capacity is maintained, the calculations show this.

Deformations are controlled by using the conservative ASTM fire curve, the fireproofed members approach average temperatures of 1000 °F. This sets the maximum deflection of beams and frames.

Vertical deformation at peak temperature is evaluated. No acceptance criteria have been developed other than prevent collapse of the structure. Because all members reach at least 1000 °F, the deformation of the fireproofed girders is still going to be very significant.

Code Provision

> **4.2.4.3a. Advanced Methods of Analysis**
>
> The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.
>
> The *thermal response* shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials as per Section 4.2.2.
>
> The *mechanical response* results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicity the deterioration in the strength and *stiffness* with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.
>
> The resulting analysis shall consider all relevant *limit states*, such as excessive deflections, connection fractures, and overall or *local buckling*.

Code Commentary

4.2.4.3a. Advanced Methods of Analysis

Advanced methods are required when the overall structural system response to fire, the interaction between structural members and separating elements in fire, or the residual strength of the structural system following a fire must be considered.

WTP Application

4.2.4.3a. Advanced Methods of Analysis Comments

Code Provision

> **4.2.4.3b Simple Methods of Analysis**
>
> The methods of analysis in this section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.
>
> The support and restrain conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.
>
> 1. Tension members
>
> It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis

fire defined in Section 4.2.1.

The *design strength* of a tension member shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

2. Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The design strength of a compression member shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3.

3. Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member.

The design strength of a flexural member shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3.

4. Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the beam.

The design strength of a *composite* flexural member shall be determined using the provisions of Chapter 1, with reduced yield stresses in the steel consistent with the temperature variation described under thermal response.

Code Commentary

4.2.4.3b. Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column.

WTP Application

4.2.4.3b. Simple Methods of Analysis Comments

The design cases considered are based on this approach.

Code Provision

> **4.2.4.4. Design Strength**
>
> The design strength shall be determined as in Section B3.3. The *nominal strength*, R_n, shall be calculated using material properties, as stipulated in Section 4.2.3, at the temperature developed by the design-basis fire.

Code Commentary

4.2.4.4. Design Strength

The design strength for structural steel members and connections is calculated as ϕR_n, in which R_n = nominal strength, in which the deterioration in strength at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal strength is computed as in Chapters C, D, E, F, G, H, I, J and K of the Specification, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1 and A-4.2.2. While ECCS (2001) and Eurocode I (1991) specify partial material factors as equal to 1.0 for "accidental" limit states, the uncertainties in strength at elevated temperatures are substantial and in some cases are unknown. Accordingly, the resistance factors herein are the same as those at ordinary conditions.

WTP Application

4.2.4.4. Design Strength Comments

The material degrades as the temperature increases, which is evaluated against allowable and related code criteria.

Code Provision

> **4.3. DESIGN BY QUALIFICATION TESTING**
>
> Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE (1998), ASTM (2000) and similar documents.
>
> An unrestrained condition is one in which expansion at the support of a load carrying element is not resisted by forces external to the element and the supported ends are free to expand and rotate. A steel member bearing on a wall in a single span or at the end of multiple spans should be considered unrestrained when the wall has not been designed and detailed to resist thermal thrust.

WTP Application

4.3. Design by Qualification Testing Comments

Not used.

Appendix E

Analysis of a LAW Facility Framing using ASTM-E119 Fire Loading

Appendix E

Analysis of a LAW Facility Framing using ASTM E-119 Fire Loading

1.0 **Introduction**
In the HLW, PTF and LAW buildings, the primary beams are fire proofed (FP) and the secondary beams are non-fire proofed (NFP). Analyses have been performed and summarized in this appendix for various temperature conditions and scenarios. The objective of the analysis is to determine the effect of the NFP beams on the FP members on a floor system of primary beams and columns.

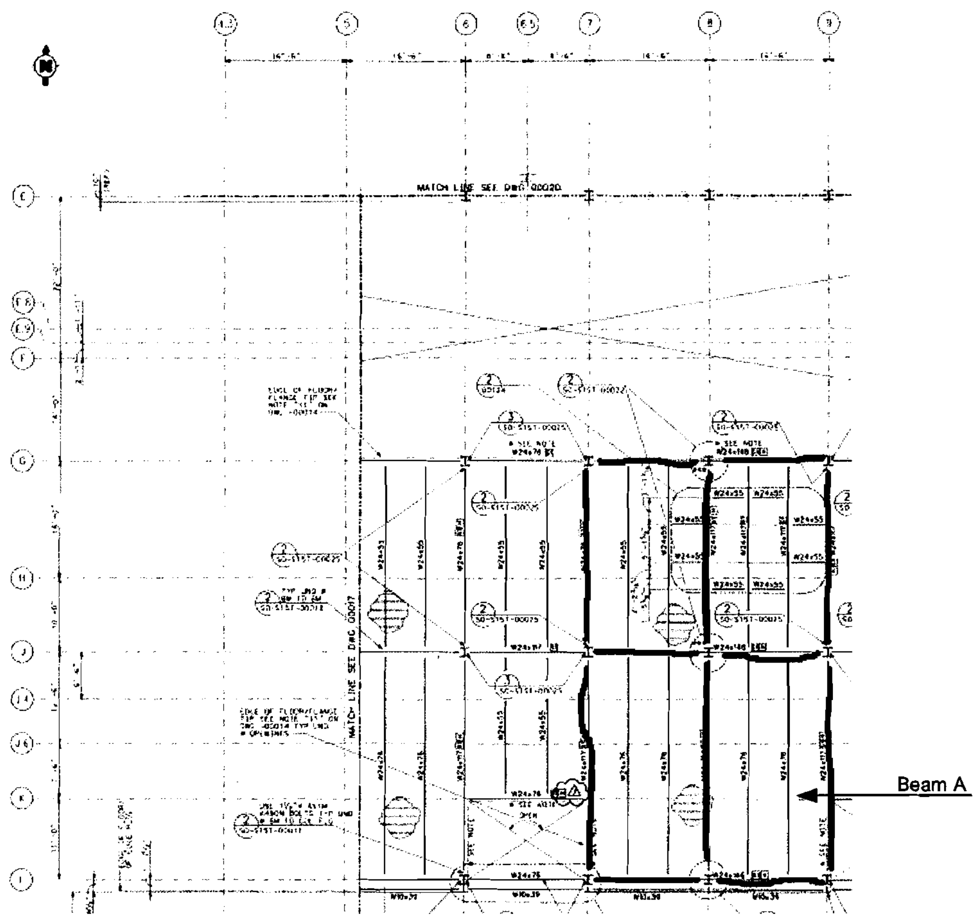
The scenarios analyzed are those where the NFP beams have the potential to damage the FP beams and columns to the extent that they are unable to perform the safety function. In evaluating the ability of a FP member, permanent deformation in the primary member is allowed. Such deformation would not be allowed for "normal design conditions" but for such extreme conditions, permanent deformation is permitted. It is necessary, however, to demonstrate that the deformed FP beams can still meet their functional requirements.

2.0 **Cases Analyzed**
The floor system selected for analysis is shown below. This section of floor was selected because it:

- is representative of the typical bay spacing
- includes both fire proofed and non-fire proofed members that frame directly into fire proofed girders
- contains longer spans which increases thermal expansion
- includes exterior conditions
- is supported by columns that are representative of typical columns in the buildings

For the evaluation in this report, a LAW area of framing at Elevation 28 feet was chosen. It is between column lines 7 and 9 and between G and L.

The fire proof beams support a 12 inch thick slab and have welded studs, 3/4 inch in diameter, 4.5 inches long and spaced at 6 inches on center. In this section of floor, there are puddle welds between the Q-decking and the top flange of the NFP beams (W24x76). The puddle welds are 5/8 inch in diameter and spaced at less than 36 inches on center.



The following cases are evaluated in this study

Case 1a

Using the floor system between column lines 8 and 9 and between J and L, heat beam A up to 1000 F in 200 F increments starting at 200 F. Assume there are no welded studs or puddle welds attaching the beam to the concrete. The FP beams have welded studs spaced 6 inches on center. No other elements are heated.

Case 1b

Same as Case 1a but the puddle welds between the Q-decking and the NFP beam are assumed to provide adequate support to the top flange. The lower flange is allowed to buckle. Heat the beam to 750 F. This is a condition where the yield strength remains unchanged by the temperature and the modulus of elasticity has only decreased to 20,300 ksi. The maximum push from the NFP beam will occur with the yield strength and modulus of elasticity in this range.

Case 1c

Same as Case 1b but the lower flange of the NFP beam is not allowed to buckle. Commodities are supported from the lower flange and will provide some lateral support and resistance to buckling.

Case 2

Use the same section of floor as Case 1 but heat both NFP beams to 750 F

Case Global

Use the floor section between column lines 7 and 9 and between J and L and heat the beams, concrete slab and columns in the following sequence. This temperature sequence is intended to replicate the conditions of a 2 hour fire.

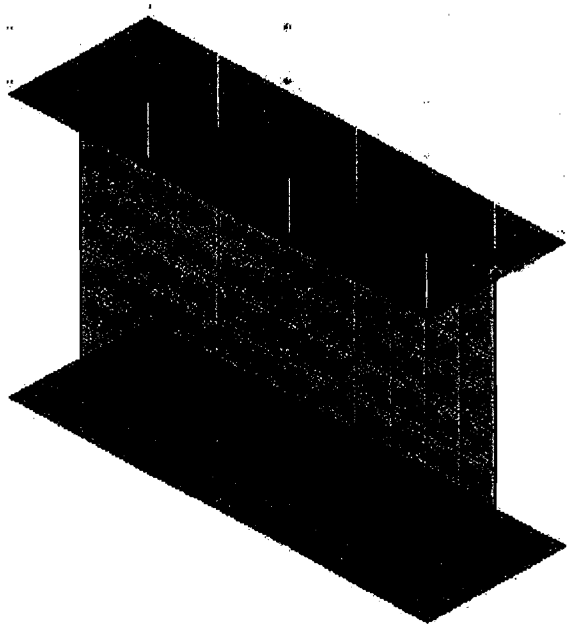
Scenario 1 Maximize the push capability of the NFP beams
- Heat both NFP beams to 750 F
- Heat the FP beams to 200 F
- Heat the column to 200 F
- Heat the slab to 200 F with a thermal gradient of 200 F over the slab depth.

Scenario 2 Maximize the push capability of the FP beams
- Heat both NFP beams to 1400 F
- Heat the FP beams to 750 F
- Heat the column to 400 F
- Heat the slab to 400 F with a thermal gradient of 400 F over the slab depth

Scenario 3 Maximize the temperature of all the components
- Heat both NFP beams to 2000 F
- Heat the FP beams to 1000 F
- Heat the column to 1000 F
- Heat the slab to 500 F with a thermal gradient of 500 F over the slab depth

3.0 Basic Finite Element Model

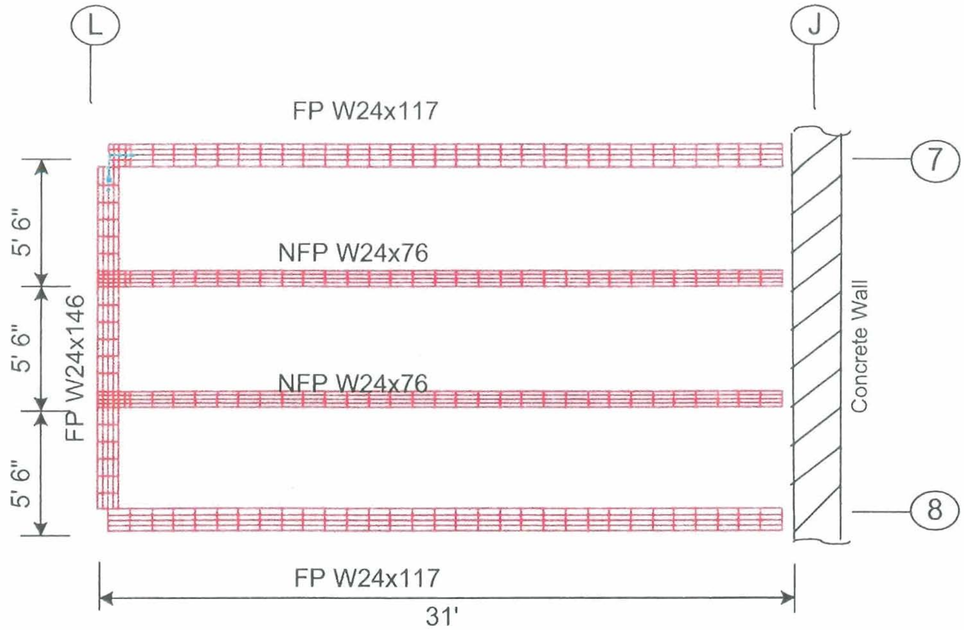
A series of FEMs were developed that modeled the various slab and beams/girders configuration with finite elements. The beams were modeled with shell elements, 4 elements for both top and bottom flanges and 8 elements for the web. The slab was also modeled with shell elements and positioned 6 inches above the top flange. The slab is 12 inches thick so the relative position of the center line of the beam and centroidal plane is maintained. The building block used to generate the beams is shown below.

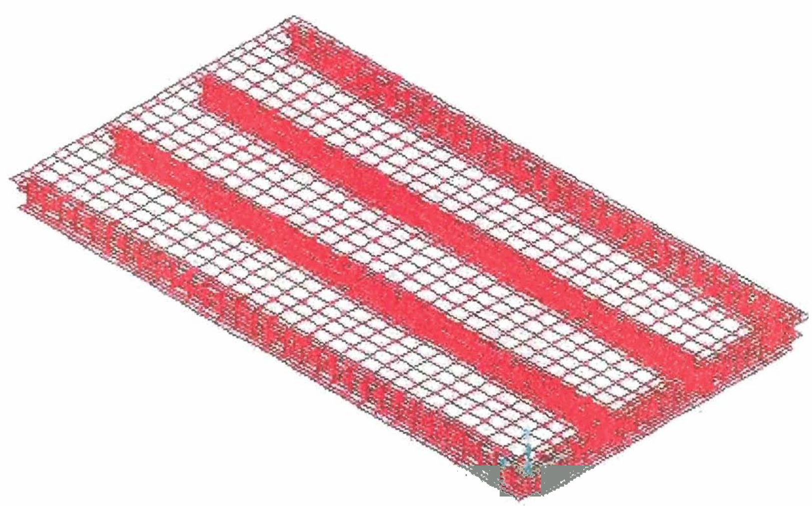


Finite Element Model of W24x76 Building Block for the Models of the Floor System

There are short beam elements along the center line of the W24x76 and along the edges of the top flange. The beam elements along the center line represent studs or puddle welds and are sized such that the stiffness is 1000 kips/in; the approximate stiffness of a 3/4" ϕ x 4.5" welded stud. The short beam elements at the edge of the flange are to provide vertical resistance so the top flange does not bend or twist in the buckling analysis. The green circles at the end of these elements indicate that the moment has been released at the ends which results in a strut.

The beams in the model of the floor system are shown below:





Isometric View of Floor System

In the model, there is a concrete wall supporting the far end of the bay to force the beam elongation into the FP girders. The near corners are supported vertically but are allowed to move in the horizontal direction.

$\alpha_{st} := 7.8 \cdot 10^{-6} \cdot \frac{in}{in \cdot deg}$

Coefficient of thermal expansion of steel for temperatures above 150 deg F per Appendix 4 of LRFD, 13th edition

$For_i = \frac{\Delta T_i \cdot \alpha \cdot Len}{\left(\frac{1}{Stiff} + \frac{Len}{Em_i \cdot Area} \right)}$

Axial force in an elastically restrained beam due to heating, spring at one end of the beam, simple support at the other end

$kip := 1000 \cdot lbf$ $\quad$ $ksi := \frac{kip}{in^2}$

Heated NFP Beams

Case 1a -- As in initial case, only Beam A is heated with no studs attaching it to the 12" concrete slab, T = 200F. The other beams and slab are not heated

From the FEM analysis, the deflection and axial force at the end of the NFP W24x76 after it is heated to 200F are:

$\Delta_{1a\_200} := 0.25 \cdot in$ $\qquad$ $Force_{200} := 539 \cdot kip$

Unrestrained elongation of the NFP beam, W24x76

$\Delta = \alpha \cdot \Delta T \cdot Len$

$\Delta T_1 := 200 \cdot deg$

$Len := 31 \cdot ft$

$\Delta_{T1} := \alpha_{st} \cdot \Delta T_1 \cdot Len$ $\qquad$ $\Delta_{T1} = 0.58\, in$

The stiffness of a single stud is approximately 1000 kips/in based on tests by Nelson Stud Welding. The stiffness at the end of the NFP W24x76 comes from the FP W24x146 in weak axis bending and the studs attached to the slab. The resistance comes from more than the one stud directly in line with the W24x76. The adjacent studs also contribute to the resistance. To determine the collective stiffness of the welded studs and the weak axis bending of the W24x146, a unit load was applied to the FEM at the intersection of the W24x76 and the W24x146 but without the W24x76 attached. The resulting stiffness was 2500 kips/in. The axial force in the W24x76 from the FEM is checked with the simple model of the single beam heated, pin supported at one end and restrained by an axial spring at the other end.

$\text{Area} := 22.4 \cdot \text{in}^2$ $\quad$ $\text{Em} := 29000 \cdot \text{ksi}$ $\quad$ $\text{Stiff} := 2500 \cdot \frac{\text{kip}}{\text{in}}$ $\quad$ $\Delta T_1 = 200 \, \text{deg}$

$$\text{For} := \frac{\Delta T_1 \cdot \alpha_{st} \cdot \text{Len}}{\left(\frac{1}{\text{Stiff}} + \frac{\text{Len}}{\text{Em} \cdot \text{Area}}\right)} \qquad \text{For} = 596.632 \, \text{kip}$$

This close approximation to the force from the FEM (539 kips) indicates the FEM is functioning adequately.

The force from the FEM is also compared to the fully restrained force.

$$\text{For}_{\text{Full}} := \frac{\Delta T_1 \cdot \alpha_{st} \cdot \text{Len}}{\frac{\text{Len}}{\text{Em} \cdot \text{Area}}} \qquad \text{For}_{\text{Full}} = 1.013 \times 10^3 \, \text{kip}$$

The fully restrained force is considerably greater than the force from the FEM, as expected.

The buckling strength of NFP W24x76 is determined in order to assess the controlling mode of behavior. Based on simply supported end conditions, the buckling strength based on the unmodified Euler buckling equation is:

$$P_{Cr\_76} = \frac{\pi^2 \cdot E \cdot I}{L^2} \qquad I_{76} := 82.5 \cdot \text{in}^4 \qquad \text{Em} = 2.9 \times 10^4 \, \text{ksi}$$

$$P_{Cr\_76} := \frac{\pi^2 \cdot \text{Em} \cdot I_{76}}{\text{Len}^2} \qquad P_{Cr\_76} = 170.634 \, \text{kip}$$

As a second condition, assume there is sufficient rotational resistance at the column ends to result in the effective length factor of 0.7 per Chapter C, Fig C-C2.3 in LRFD, 13th edition

$$k := 0.7 \qquad P_{Cr\_76\_2} := \frac{\pi^2 \cdot \text{Em} \cdot I_{76}}{(k \cdot \text{Len})^2} \qquad P_{Cr\_76\_2} = 348.233 \, \text{kip}$$

The buckling capacity is likely in this range which is less than the axial load from the FE analysis. This says that the unsupported NFP W24x76 will buckle before it is heated to 200F.

Case 1a ends at this point. There is no need to go further up the temperature scale since the W24x76 has buckled.

Case 1b -- Same as Case 1a except the puddle welds between the Q-decking and the top flange of the NFP W24x76 are included. The puddle welds are modeled using the short beam elements between the top flange of the W24x76 and the slab, the same as the model for the shear studs. The model for the shear studs is stiff but may not be as stiff as the actual puddle weld. The usefulness of the model will be judged as the analysis progresses.

In Case 1a, the NFP W24x76 was not attached to the slab and was allowed to expand into the FP W24x146 which has studs 6" on center into the concrete slab. For Case 1b, there are puddle welds between the Q-decking and the top flange of the W24x76 which are 5/8 inch in diameter spaced no greater than 36 inches and are assumed to be effective. A 12 inch spacing of the puddle welds was used as a bounding case. The Q-decking is 16 gauge which has a thickness of 0.0598 inches.

The strength of a puddle weld attached to the thin steel plate of the Q-decking is difficult to determine but a tear on each side of the weld in the 16 gauge steel behind the weld will provide a starting point estimate. The length of the tear is taken as twice the weld diameter and it is assumed to tear along both sides of the weld. The failure mode is shear. The yield strength is taken as the tensile yield strength divided by $\sqrt{3}$.

$t_Q := 0.0598 \cdot in$ Q-decking thickness

$F_{yQ} := 60 \cdot ksi$

$F_{PW} := t_Q \cdot \frac{F_{yQ}}{\sqrt{3}} \cdot \frac{5}{8} \cdot in \cdot 2 \cdot 2$ $F_{PW} = 5.179\,kip$

This is a very small value and will provide a negligible resistance to the elongation of the W24x76. The puddle welds may provide lateral support for the top flange of the W24x76 and is assumed adequate as a bounding case. The lower flange of the NFP W24x76 will roll over and buckle laterally if unsupported. For a bounding case, the lower flange is assumed to be supported by the attached commodities.

The puddle welds are likely to fail which will progress from the end of the beam farthest from the concrete wall back towards the wall. The buckling strength of the W24x76 as a function the puddle weld failure is estimated below. The calculation determines the buckling strength in one foot increments so the counter in the display of the results is also the unsupported length (Lu) of the beam in feet.

$k = 0.7$

$i := 1..30$ $\qquad$ $I_{Fl} := 82.5 \cdot in^4$

$$P_{Cr\_76R_i} := \frac{\pi^2 \cdot Em \cdot I_{Fl}}{(k \cdot i \cdot ft)^2}$$

Lu (ft) Pcr (kip)

$P_{Cr\_76R} =$ (kip)

Lu (ft)	0
0	0
1	$3.347 \cdot 10^5$
2	$8.366 \cdot 10^4$
3	$3.718 \cdot 10^4$
4	$2.092 \cdot 10^4$
5	$1.339 \cdot 10^4$
6	$9.296 \cdot 10^3$
7	$6.83 \cdot 10^3$
8	$5.229 \cdot 10^3$
9	$4.132 \cdot 10^3$
10	$3.347 \cdot 10^3$
11	$2.766 \cdot 10^3$
12	$2.324 \cdot 10^3$
13	$1.98 \cdot 10^3$
14	$1.707 \cdot 10^3$
15	$1.487 \cdot 10^3$
16	$1.307 \cdot 10^3$
17	$1.158 \cdot 10^3$
18	$1.033 \cdot 10^3$
19	927.013
20	836.629
21	758.848
22	691.429
23	632.612
24	580.993
25	535.443
26	495.047
27	459.056
28	426.852
29	397.921
30	371.835

$F_y := 58 \cdot ksi$ $\qquad$ Accounting for overstrength

$P_{Fy} := Area \cdot F_y$

$P_{Fy} = 1.299 \times 10^3\, kip$ $\qquad$ Yield strength of whole cross-section W24x76

The maximum push is likely to be near T = 750 degrees which is the temperature before the yield strength starts to decrease. Based on the FEM, the displacements and forces on the FP W24x146 due to the 750F heated NFP W24x76 are:

$\Delta_{TF} := 0.32 \cdot in$ Top flange of W24x146 displacement

$\Delta_{BF} := 0.33 \cdot in$ Bottom flange of W24x146 displacement

$\Delta_{Slab} := 0.020 \cdot in$ Slab displacement

$F_{76} := 440 \cdot kip$ Axial force in the W24x76 at 750F

$Slip := \Delta_{TF} - \Delta_{Slab}$ $Slip = 0.3\, in$

$\Delta_{750} := \alpha_{st} \cdot (750 - 68) \cdot deg \cdot Len$ Unrestrained elongation of NFP W24x76 at 750 F

$\Delta_{750} = 1.979\, in$

$E_{m750} := Em \cdot 0.7$ Modulus of elasticity at 750F

$P_{750} := \left(\alpha_{st}\right) \cdot (750 - 68) \cdot deg \cdot E_{m750} \cdot Area$ Fully restrained axial force in the 750F heated W24x76

$P_{750} = 2.419 \times 10^{3}\, kip$

The displacement of the FP W24x146 is less than the unrestrained elongation of 1.98 inches and the axial force is less than the fully restrained force of 2419 kips. Results are in the reasonable range.

The axial force F_{76} is less than the yield strength of the of the W24x76. If essentially all the puddle welds fail and do not provide adequate lateral support for the lower flange, the NFP beam will buckle and limit the push into the FP beam, W24x146

The studs on the FP beam are still attached and need to be reviewed to see if they are overloaded. The review shows there are some studs that are at their slip limit, 1/3 inch, which will be disconnected and those studs are replaced with a 30 kip load, equal and opposite at each end of the 6 inch high posts representing the studs. Rather than do the iterative analysis for this condition, it will be done in Case 2 instead.

To assess the effect of higher temperatures the single NFP W24x76 beam was heated to 1400F and all other elements were artificially maintained at 68F. At this temperature, the yield strength is reduced as is the modulus of elasticity, Case_1b_R1

$\Delta_{TF\_1400} := 0.175 \cdot in$ — Top flange of W24x146 displacement

$\Delta_{BF\_1400} := 0.183 \cdot in$ — Bottom flange of W24x146 displacement

$\Delta_{Slab\_1400} := 0.008 \cdot in$ — Slab displacement

$F_{76\_1400} := 237 \cdot kip$ — Axial force in W24x76 at 1400F

$F_{y\_1400} := 8 \cdot ksi$ — Yield strength of steel at 1400F

$F_{76y\_1400} := Area \cdot F_{y\_1400}$ — Yield strength of W24x76 at 1400F

$F_{76y\_1400} = 179.2\,kip$ — Axial force in the W24x76 at 1400F

This shows that the yield strength of the W24x76 at 1400 degrees is less than the force from the FEM with the reduced modulus of elasticity indicating the pushing capability of the W24x76 is less than in the model. The implication is the deflection of the FP W24x146 will be less than the 0.17 inches

Case 1c

Commodities will be attached and hung off of NFP beams. As indicated in Case 1b, the lower flange will buckle early if unsupported. For a bounding case, it is assumed the commodities supported from the lower flange will provide adequate lateral support to prevent premature buckling.

One secondary NFP W24x76 is heated to 750F. This is will produce the maximum load on the primary FP W24x146. If both NFP are heated, the FP will provide less resistance, therefore less axial force develops in the NFP W24x76s. After 750F the yield strength (Fy) of steel begins to degrade, reducing the amount of force NFP W24x76 can exert.

The modulus of elasticity for concrete is reduced by 1/2 to account for cracking in the concrete as was done in the design FEM of the buildings. Modulus of elasticity for the FP W24x76 steel have also been reduced per LRFD to account for the rise in temperature.

When the NFP W24x76 is heated to 750F, the puddle welds are severely overloaded. Twenty of the puddle welds that saw excessive shear were removed and the model rerun to capture more accurately the elongation of the NFP W24x76 into the FP W24x146. The axial load in the NFP W24x76 after the 20 puddle welds have been removed is 853k. This axial load is greater than the buckling capacity so the push capacity of the NFP beams is limited.

The shear in some beam elements representing the studs exceeds the stud capacity. The model is modified to better represent the actual conditions. The model was modified as follows:

1) If the shear force is greater than the stud capacity (30 kips) and the slip is greater than its capacity (1/3 inch), the stud is removed.

2) If the shear force is greater than the capacity but the slip is less than capacity, the stud is replaced by forces at the ends of the stud equal to the stud capacity.

These forces are equal in magnitude and opposite in direction and represent the internal forces between the top flange of the steel beam and the concrete. The applied force is equal to the full capacity because the load-deflection curve for welded studs is nearly flat for the final 1/2 of the slip.

Following this procedure, four studs on both sides of the W24x76 are removed and two 60k forces inserted in place of the 4th stud on each side since its slip is still less than 1/3 inch Per general arrangement, the studs are 6 inches on center on the FP W24x146. In the model, the studs are 12 inch on center so each studs capacity is 60k rather than 30k to compensate for this difference. The model is then rerun. The deflection of the FP W24x146 is 1.18", and half of the top and bottom flanges of the FP W24x146 have yielded.

The final axial load on the NFP W24x76 is 644k which is greater than its buckling capacity of 586k, meaning the NFP W24x76 will buckle before it deflects the FP W24x146 1.18inches.

$L_{u20} := 20 \cdot ft$

Unbraced length of 20 feet due to welds

$$P_{cr\_750} := \frac{\left[\left(\pi^2\right) \cdot E_{m750} \cdot I_{76}\right]}{\left(k \cdot L_{u20}\right)^2} \qquad P_{cr\_750} = 585.641\ kip$$

Buckling capacity of NFP W24x76, heated to 750F, with an unbraced length of 20 feet

Case 2 -- Use the same bay and structural configuration as in Case 1 but heat up both beams, maintaining the other structural elements at room temperature. Both beams have puddles welds between the Q-decking and their top flanges. To assess how far the W24x146 will be pushed by the W24x76, go straight to 750 degrees and W24x76s being fully effective.

$\Delta_{T2} := 0.282 \cdot in$ Top flange of W24x146 displacement at 750F

$\Delta_{B2} := 0.321 \cdot in$ Bottom flange of W24x146 displacement at 750F

$\Delta_{S2} := 0.039 \cdot in$ Slab displacement at 750F in the W24x76

$F_{A2} := 569 \cdot kip$ Axial force in W24x76 at 750F

$P_{Fy} = 1.299 \times 10^3\ kip$ Yield strength of the W24x76

This indicates that if the puddle welds can provide lateral support, an axial force of 569 kips will develop which is less than its yield strength. The lateral deflection in the FP W24x146 is only 0.3 inches which seems tolerable.

The studs on the FP W24x146 are still attached and need to be reviewed to see if they are overloaded. The review indicates there are some studs that are at their slip limit, 1/3 inch, which are then disconnected and replaced with 30 kips loads, equal and opposite at each end of the 6 inch high posts representing the studs. The disconnected studs are all the studs between the two W24x76 and two studs on either side of the W24x76 beams.

These results are from Fire_Case_2_R2 where the studs are disconnected and replaced by 30 kip shear loads

$\Delta_{T2\_R2} := 0.454 \cdot in$ Top flange of W24x146 displacement at 750F

$\Delta_{B2\_R2} := 0.33$ Bottom flange of W24x146 displacement at 750F

$\Delta_{S2\_R2} := 0.078 \cdot in$ Slab displacement at 750F

$Slip\_2R2 := \Delta_{T2\_R2} - \Delta_{S2\_R2}$ $Slip\_2R2 = 0.376\,in$

$F_{A\_R2} := 274 \cdot kip$

Slip is greater than the slip limit of 1/3 inch, therefore the shear studs attached to the 2-W24x76 are likely to shear off.

The model is revised to remove the 30 kip loads which represented the yielded shear studs. It essentially disconnects the W24x146 beam from the slab.

$\Delta_{T3} := 0.58 \cdot in$ $\Delta_{B3} := 0.58 \cdot in$ $F_{A\_R3} := 147 \cdot kip$

Check conditions at T = 1400 degrees which is after the steel has softened and reduced in strength.

$\Delta_{T2\_1400} := 0.122 \cdot in$ Top flange of W24x146 displacement

$\Delta_{B2\_1400} := 0.227 \cdot in$ Bottom flange of W24x146 displacement

$\Delta_{S2\_1400} := 0.017 \cdot in$ Slab displacement

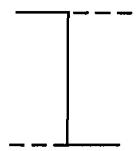
$F_{A2\_1400} := 252 \cdot kip$ Axial force in the W24x76 at 1400F

$F_{76y\_1400} = 179.2\,kip$ Yield strength of W24x76

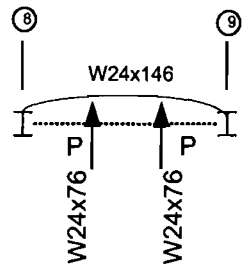
Since the yield strength of the W24x76 is less than the force from the FEM with the reduced modulus of elasticity and increase coefficient of thermal expansion, the steel member will yield and as a result will not be able to push as hard.

Check 1.2D+0.5L+T on FP girder W24x146 per Appendix 4, LRFD 13th edition.

Under gravity loading, the bottom flange will be in tension. As the NFP (one or both) W24x76 elongates due to thermal expansion, it will push against the FP W24x146 in the lateral direction and put the outer half of the top and bottom flanges of the FP girder in tension. The bottom outer flange will always be in tension when biaxially loaded hence we will assume it yields. The top inner flange will always be in compression when biaxially loaded hence we will assume it yields. The two remaining flanges will experience tension then be offset by compression (or visa-versa). To evaluate the biaxial effects, the yielded portion of the top and bottom flange areas are considered to have yielded in weak-axis bending, and a Z-shape section is available to resist the strong-axis bending.

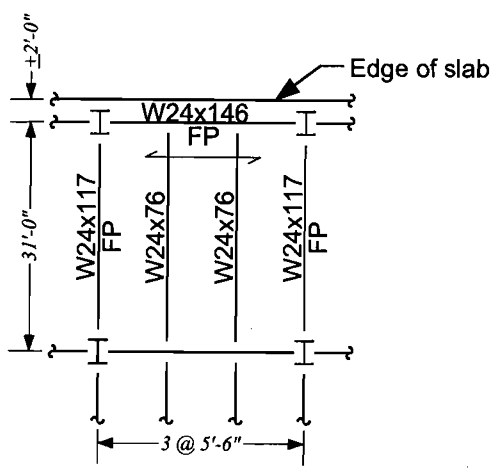


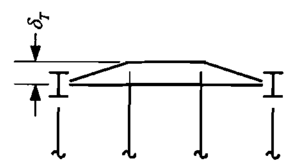
Cross section



Plan view

Layout:





δ_T = thermal expansion of NFP beams

Section and Material Properties

FP girder W24x146 properties:

$L_g := 16.5ft$ $A_g := 43in^2$ $d_g := 24.7in$ $t_{wg} := 0.65in$ $b_{fg} := 12.9in$ $t_{fg} := 1.09in$

$I_{xg} := 4580in^4$ $S_{xg} := 371in^3$ $Z_{xg} := 418 \cdot in^3$ $I_{yg} := 391in^4$ $S_{yg} := 60.5in^3$ $Z_{yg} := 93.2in^3$

$F_u := 65 \cdot ksi$

$F_c := 4 \cdot ksi$ $E_c := 3834 \cdot ksi$

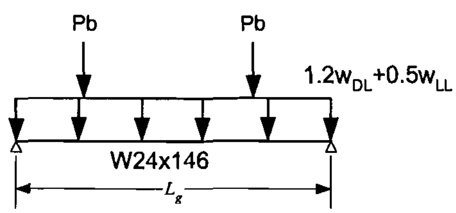
NFP beam W24x76 properties:

$L_b := 31ft$

$A_b := 22.4in^2$ $b_{fb} := 8.99in$ $t_{wb} := 0.44 \cdot in$

$I_{xb} := 2100in^4$ $I_{yb} := 82.5in^4$ $S_{xb} := 176in^3$ $S_{yb} := 18.4in^3$

Loads on FP W24x146:



Units: $psf := lbf \cdot ft^{-2}$ $plf := lbf \cdot ft^{-1}$

$DL_{girder} := 146plf$ girder self weight

$DL_{deck} := 10 \cdot psf$ metal deck

$DL_{slab} := 150 \cdot psf$ 12" slab

$DL_{beam} := 76 \cdot plf$ beam self weight

$DL_{com} := 50 \cdot psf$ commodity

$DL_{part} := 20psf$ partition

$LL := 100psf$ live load

$d_{edge} := 2 \cdot ft$ distance to slab edge

$w_{trib\_girder} := \frac{31}{2} \cdot ft + d_{edge}$ $\quad w_{trib\_girder} = 17.5\,ft$

$w_{trib\_beam} := 5.5 \cdot ft$

$w_{DL} := w_{trib\_girder} \cdot (DL_{deck} + DL_{slab} + DL_{part}) + DL_{girder}$ **Uniformly distributed dead load on girder**

$w_{DL} = 3.296 \times 10^3\,plf$

$w_{LL} := w_{trib\_girder} \cdot LL$ $\quad w_{LL} = 1.75 \times 10^3\,plf$ **Live load on girder**

$w_{beam} := DL_{beam} + DL_{com} \cdot w_{trib\_beam}$ **Load on NFP beam**

$w_{beam} = 351\,plf$

$P_b := w_{beam} \cdot \frac{L_b}{2}$ $\quad P_b = 5.44\,kip$ **Concentrated load on the FP girder from NFP beam**

$$M_{ux} := \left[\frac{(1.2 \cdot w_{DL} + 0.5 \cdot w_{LL}) \cdot L_g^{\,2}}{8}\right] + 1.2 \cdot P_b \cdot 5.5 \cdot ft$$

Maximum moment in FP girder using load combination from Ref. 1

$M_{ux} = 200.285\,kip \cdot ft$ Distance to point load Pb is 5.5 feet

Strong-axis bending:

Half of the top and bottom flanges of the FP W24x146 girder might yield in weak-axis bending due to the heated NFP beam, leaving a Z-shape composite section to resist gravity loading in strong-axis bending. If all the studs were to shear off the FP W24x146, the non-composite Z-section would need to withstand the 1.2D+0.5L+0.2S gravity loading. Per LRFD, Section 4.1.4, the deformation to the W-section due to heat T is considered then loaded with 1.2D+0.5L+0.2S to account for biaxial effects.

Z - shape section properties

$d_z := 24.7 \cdot in$

$t_{wz} := 0.65 \cdot in$

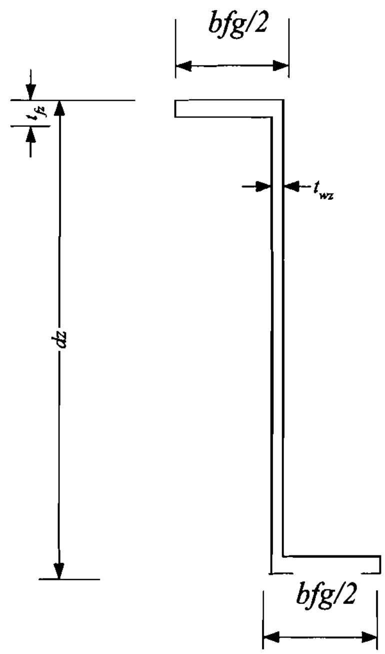
$t_{fz} := 1.09 \cdot in$

$b_{fz} := \frac{b_{fg}}{2}$

$b_{fz} = 6.45\ in$

$A_z := A_g - 2 \cdot t_{fz} \cdot b_{fz}$

$A_z = 28.939\ in^2$



$Z_{xz} := t_{fz} \cdot b_{fz} \cdot (d_z - t_{fz}) + t_{wz} \cdot \left(\frac{d_z}{2} - t_{fz}\right)^2$ $\quad Z_{xz} = 248.402\ in^3$

$Mpx := F_y \cdot Z_{xz}$

$M_{ux} < Mpx$

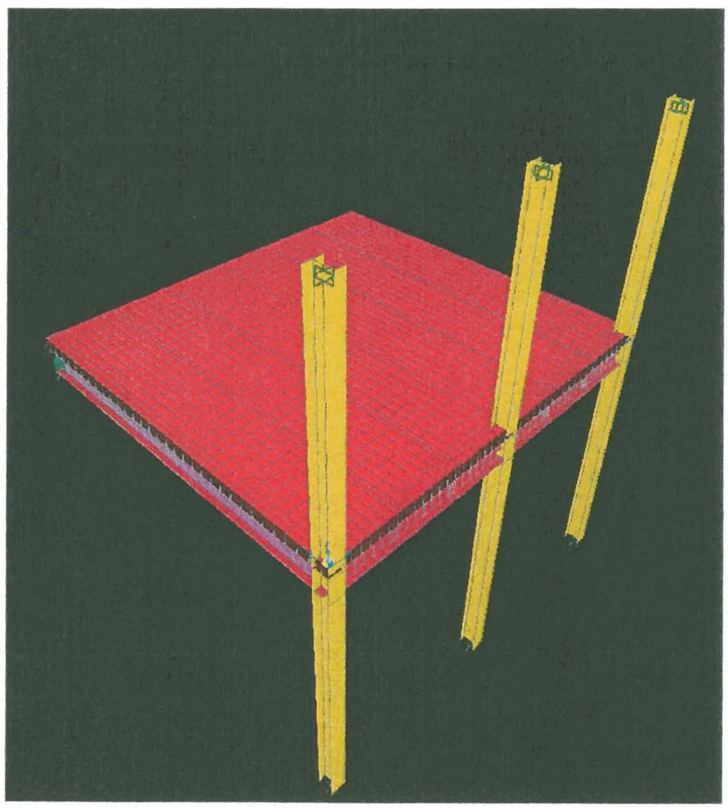
$DC_m := \frac{M_{ux}}{Mpx}$ $\quad DC_m = 0.167$

The FP W24x146 has adequate capacity to resist strong-axis bending post fire

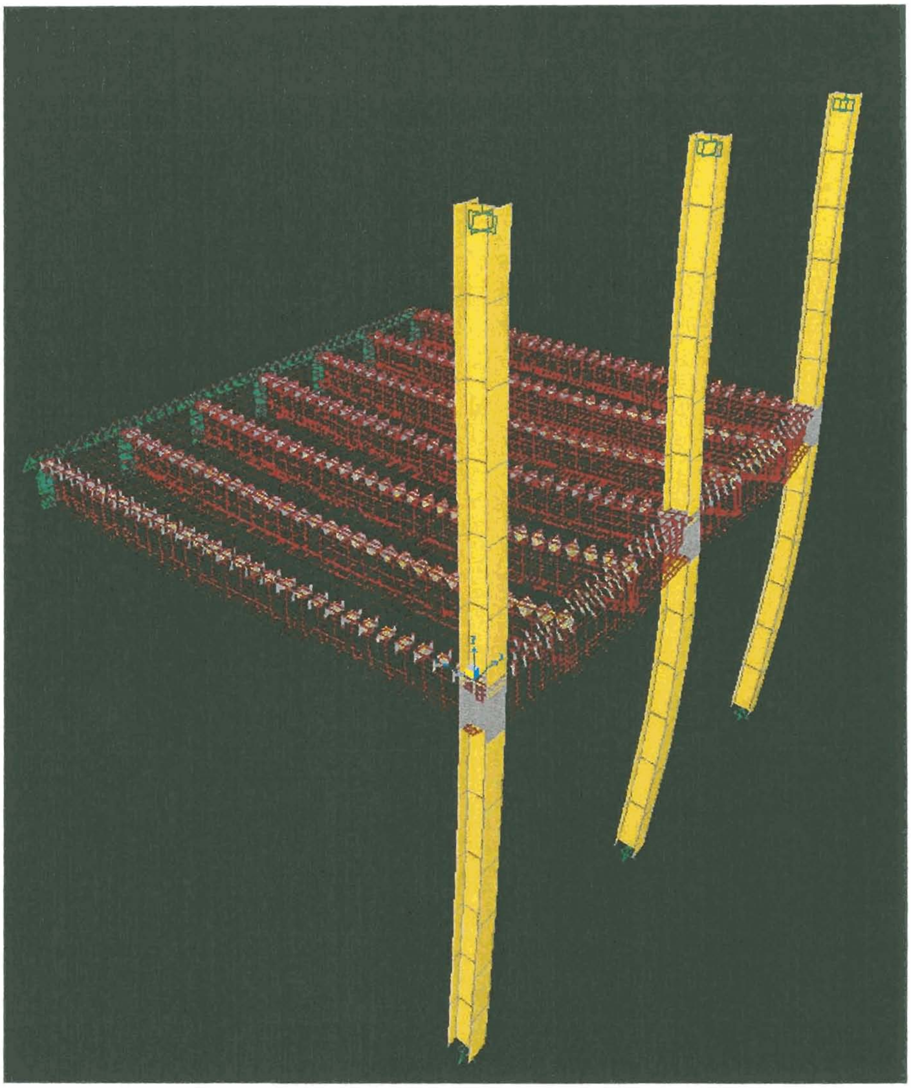
Case Global

In this case the temperature sequence is intended to replicate the conditions of a 2-hour fire. The first temperature step takes the NFP beams to 750F which produces their maximum push. Other members are at a lower temperature. The second temperature step takes the FP beam to 750F which produces their maximum push. The columns and slabs are at a lower temperature. The third step takes the FP beams and columns to 1000F and the slab to 500F. A detailed description is on Section 2.0.

Undeflected view of FEM



Deflected shape of FEM when column deflect 2.5 inches



The bay between column lines 7-9 and J-L was heated using different temperature scenarios.

As in Case 1c, the steel and concrete stiffnesses were adjusted per LRFD to account for the temperature load. The thermal coefficient of expansion for steel and concrete was also inserted per LRFD.

Scenario 1

Case1c model was used as a starting point with the 8 studs removed on the FP W24x146 and the 20 welds removed from the NFP W24x76 beams. All of the appropriate temperatures were added and the model run.

The capacity of the puddle welds is about 13k and the shear capacity of the metal deck is 5 kips. In the FEM, the shear on the NFP W24x76 welds ranges from 280k to 30 kips. Hence all of the welds on the NFP W24x76 are removed from the model and rerun.

After the puddle welds were removed and the model was reanalyzed, some of the studs on the W24x146 were overloaded. All the studs between the 2 W24x76 beams were removed. In addition, 3 studs on either side of the W24x76 were removed.

The studs on the primary FP W24x117, on column line 8, do not experience excessive shear or slip and are thus left in the model.

The slab on column lines 8/L elongates 0.62 inches
The FP W24x117 on column line(CL) 8/J-L elongates 0.62 inches into the column on CL 8/L
The NFP W24x76 on CL 7-8/L displaces 1.9 inches. The thermal coefficient of the FP W24x146 increases at 200 F, increasing its deflection.

Scenario 2

Using the final model from scenario 1, a model with all of the puddle welds on the W24x76 and most of the studs on the FP W24x146 removed, the members are then heated with scenario 2 temperatures and the model is run.

The remaining studs do not slip nor experience shear beyond its capacity. No additional studs/welds are removed.

Axial load on NFP W24x76 on CL 7-8/L =186kips (Pallow=19k therefore they have buckled)
Elongation of the slab at CL 8/L =1.55inches
Elongation of FP W24x117 on CL 8/J-L =1.55in
Axial load on the FP W24x117 on CL 8/J-L =230k
Lateral displacement of the column W14x233 on CL 8/L=1.55inches
Moment on column(CL 8/L) due to the heated W24x117 (CL 8/J-L)=715k-ft

Scenario 3

The final model from scenario 2 is used and heated with this scenario's temperatures. No studs are removed. The column below this elevation is heated to 1000F and kept at 68F above this elevation. Keeping the column above the heated bay at ambient temperature is consistant with research performed by Hong, S. et.al (see references)

Elongation of the slab at CL 8/L=2.5inches
Elongation of FP W24x117 (CL 8/J-L)=2.5in (unconstrained elongation is 2.7 inches)
Axial load on the FP W24x117(CL 8/J-L)=401kips
Deflection of column W14x233 (CL 8/L)=2.5 inches
Moment on column(CL 8/L) due to the heated W24x117 (CL 8/J-L)=1021k-ft

The strength capacity accounting for P-Δ effects of the FP W14x233 at 1000F that experiences a 2.5 inch deflection:

$M_{1000} := 1021 \cdot kip \cdot ft$ — Moment on 1000F heated column due to 1000F heated FP W24x117

$P_{gravity} := 200 \cdot kip$ — Gravity loading using 1.2DL+0.5LL+0.2S (the deformation due to T is accounted for in the reduction of a W-section to a Z-section)

$e_{1000} := 2.5 \cdot in$ — lateral displacement of column

$$M_{tot} := M_{1000} + P_{gravity} \cdot e_{1000}$$

$$M_{tot} = 1.063 \times 10^3 \, kip \cdot ft$$

$S_{x\_col} := 375 \cdot in^3 \quad Z_{x\_col} := 436 \cdot in^3 \quad r_y := 4.1 \cdot in$ — column W14x233 section properties

$A_{col} := 68.5 \cdot in^2$

$F_{y\_1000} := F_y \cdot 0.66$ — Yield strength at 1000F

$E_{m1000} := Em \cdot 0.49$ — Modulus of elasticity at 1000F

$\phi := 0.9$

$$F_b := \frac{(\phi \cdot F_{y\_1000} \cdot Z_{x\_col})}{S_{x\_col}}$$

Flexural buckling stress, Fcr, per LRFD Chapter E

$k_{col} := 1.0 \qquad L_{col} := 21 \cdot ft$

$$\frac{k_{col} \cdot L_{col}}{r_y} = 61.463 \quad < \quad 4.71 \cdot \left[\frac{(E_{m1000})}{F_{y\_1000}}\right]^{0.5} = 90.747$$

$$F_e := \frac{(\pi^2)\cdot(E_{m1000})}{\left[\left[\frac{k_{col}\cdot(L_{col})}{r_y}\right]^2\right]} \qquad F_e = 37.124\,\mathrm{ksi}$$

$$F_{cr} := \left[0.658^{\left(\frac{F_{y\_1000}}{F_e}\right)}\right]\cdot F_{y\_1000} \qquad F_{cr} = 24.862\,\mathrm{ksi}$$

$$F_a := F_{cr}\cdot\phi \qquad F_a = 22.376\,\mathrm{ksi}$$

$$\left[\frac{\frac{P_{gravity}}{A_{col}}}{F_a} + \frac{\left(\frac{M_{tot}}{S_{x\_col}}\right)}{F_b}\right] = 0.979$$

The moment in the column from the elastic model is unrealistically high due to the connectivity between the FP beam W24x117 and the FP column W14x233. In the model, the flanges and the web of the FP W24x117, and the slab are attached to the column. The actual connection per general arrangements is a shear plate welded (5/16 inch fillet weld all around) to the column and attached to the beam with 7 - 7/8 inch A325 bolts. The bolts will slip in this connection before the moment from the elastic analysis can develop. The connectivity in the model overestimates the moment in the column due to the beam/slab rotation.

An estimate of the moment in the column, due to the lateral deflection from the elongation of the W24x117, can be obtained by considering the column pinned connected at the floor above (El 47 ft) and below (El 3 ft) the floor being heated (El 28 ft) and inducing a 2.5 inch deflection at El 28 ft.

The moment in the column is given below where L1 and L2 are the lengths of the column segments above and below the floor being heated. L1 is heated, L2 is kept at ambient temperature of 68F:

$$I_{14x233} := 3010\cdot in^4 \qquad L_1 := 25\cdot ft \qquad L_2 := 19\cdot ft$$

$$M_{xx} = \left[\frac{3\cdot(L_1 + L_2)\cdot E_1\cdot E_2\cdot I\cdot\Delta}{\left(L_1^2\cdot L_2\cdot E_2 + L_1\cdot L_2^2\cdot E_1\right)}\right]$$

$e_{200} := 0.62 \cdot in$

Column deflection (Case 5, scenario 1)

$e_{400} := 1.55 \cdot in$ $\quad E_{m400} := Em \cdot 0.9$

Column deflection and modulus of elasticity (Case 5, scenario 2)

$$M_{200xx} := \frac{3 \cdot (L_1 + L_2) \cdot Em \cdot Em \cdot I_{14x233} \cdot e_{200}}{\left(L_1^{\ 2} \cdot L_2 \cdot Em + L_1 \cdot L_2^{\ 2} \cdot Em\right)}$$

Moment in column at Elev 28, heated to 200F, with 0.62inch deflection (Case 5, scenario 1)

$$M_{200xx} = 197.806\, kip \cdot ft$$

$$M_{400xx} := \frac{3 \cdot (L_1 + L_2) \cdot E_{m400} \cdot Em \cdot I_{14x233} \cdot e_{400}}{\left(L_1^{\ 2} \cdot L_2 \cdot Em + L_1 \cdot L_2^{\ 2} \cdot E_{m400}\right)}$$

Moment in column at Elev 28, heated to 400F, with 1.55inch deflection (Case 5, scenario 2)

$$M_{400xx} = 465.15\, kip \cdot ft$$

$$M_{1000xx} := \frac{3 \cdot (L_1 + L_2) \cdot E_{m1000} \cdot Em \cdot I_{14x233} \cdot e_{1000}}{\left(L_1^{\ 2} \cdot L_2 \cdot Em + L_1 \cdot L_2^{\ 2} \cdot E_{m1000}\right)}$$

Moment in column at Elev 28, heated to 1000F, with 2.5inch deflection (Case 5, scenario 3)

$$M_{1000xx} = 501.206\, kip \cdot ft$$

These moments would reduce the combined stress interaction ratio to approximately
0.31 in Case 5, scenario 1
0.53 in Case 5, scenario 2
0.57 in Case 5, scenario 3

Strong axis bending (Z-section) when heated to 1000F

Since the studs of the primary FP W24x146 have sheared off, the non-composite section need to withstand gravity loads

$Z_{xz} = 248.402 \text{ in}^3$

$M_{px\_1000} := F_{y\_1000} \cdot Z_{xz}$

$$DC_{m\_1000} := \frac{M_{ux}}{M_{px\_1000}}$$

$DC_{m\_1000} = 0.253$

The FP W24x146 has adequate capacity to resist strong-axis bending post fire

5.0 Conclusion

The analyses progressed from heating a single beam to heating an entire bay. As the conditions became more realistic, the response of the structure to the fire became less severe. Once the full bay and associated column were considered, all pertinent features were active and larger multi-bay modes do not appear to be necessary.

In the example floor system, a fireproofed member, W24x146, sustained permanent deformation but maintained its ability to support gravity loads. A column also experienced lateral deformation due to another member expanding from the heat. Post fire, the column can still support the required gravity loads.

The full bay model also pointed out that if the fire is large enough to heat up the FP components, the effect is the same regardless of fireproofing.

6.0 References

AISC. 2006. *ASD/LRFD Steel Construction Manual*, 13th edition. American Institute of Steel Construction, Chicago, IL.

Hong, S., Varma, A. H., Agarwal, A., Prasad, K. *Behaviour of Steel Building Structures Under Realistic Fire Loading.* Structures 2008: Crossing Borders, 2008 ASCE, Section 4.4