# Post-Fire Bridge Assessment Guidance and Procedures

**August 29, 2023**

# Table of Contents

**List of Tables and Figures**



**List of Appendices**

# 1.0 - Introduction

## 1.1 - Purpose

This document provides guidance and procedures for professional engineers responsible for post-fire bridge assessment. This information is applicable to bridges on Forest Service Roads (FSRs) that have been subjected to heat and flames from fires caused by wildfire, arson, vehicle accidents, and other causes.

During a wildfire, an average surface fire on the forest floor might have flames reaching 1 metre in height, with temperatures reaching 800 degrees Celsius or more. Under extreme conditions a wildfire can produce greater than 10,000 kilowatts per metre of fire front, which could mean flame heights of 50 m or more and flame temperatures exceeding 1200 degrees Celsius (ref. 9). Fire conditions such as these can cause damage to bridges. Fire impacts to bridges may be easily observable and quantifiable, or they may be difficult to detect and evaluate.

## 1.2 - Document History & Description of Literature Reviewed

A working draft of this document was originally researched, written, reviewed and published on the ministry's public website in 2018. The working draft was based on a literature review of pertinent technical documents publicly available on the internet at that time. Some of the reviewed documents focused on steel and concrete highway bridge/overpass fires that typically involved burning fuels from tanker vehicles. One document focused on wood covered bridge fires. No documents specific to bridges damaged in wildfires were discovered in the internet search, however the information provided in the reviewed documents was considered to be relevant to fire damaged FSR bridges whatever the cause of the fires. In 2023 the ministry made minor text updates and formatting changes to the working draft, updated weblinks, and made minor changes to the title (including the deletion of "Working Draft" from it.)

## 1.3 - Document Layout

Section 2.0 of this document provides basic technical guidance relating to the effects of fire on various bridge components and possible repair options.

Section 3.0 of this document provides procedures that should be followed when assessing an FSR bridge that has been involved in a fire event.

The appendices to this document provide relatively detailed information. Technical engineering information and guidance considered essential knowledge for a professional engineer involved with fire damaged FSR bridge assessment is provided in Appendices 1 to 4. This information has been excerpted from reference documents and consolidated to present detailed technical engineering information particularly germane to FSR bridges. Appendices 5 to 7 provide additional relevant information.

The information contained in each appendix is briefly described below:

- Appendix 1 "Bridge Behaviour and Capacity During and After a Fire" contains essential detailed information related to general bridge behaviour during and after a fire. It provides guidance in relation to detailed inspection and testing, damage types, steel behaviour, concrete behaviour, and bridge capacity determination.

- Appendix 2 “General Concrete Information” and Appendix 3 “Colour Change Indicators of Concrete Temperature and Deterioration” provide additional valuable information specific to fire damaged concrete bridge components.
- Appendix 4 “Wood Component Information” provides technical information about fire damaged wood bridge components.
- Appendix 5 “Celsius/Fahrenheit Comparison Table” presents a Celsius to Fahrenheit temperature conversion chart, for reference.
- Appendix 6 “Fire Damaged FSR Bridge Photos” contains photographs of some example FSR bridges that have been damaged by fire.
- Appendix 7 “List of Reference Documents” is a list of references which provides a brief description of, and a web link to, pertinent documents that could be accessed as required by professional engineers when they are involved with a particular bridge assessment.

# 2.0 – Post-Fire Bridge Assessment Guidance

## 2.1 - General Fire Effects on Bridge Components

The extent of fire damage depends on the intensity and duration of the fire, the geometry of the structure, the type of material, and the type of load on the members. Bridge components individually subjected to high temperatures due to direct proximity to flames are obvious members that may be significantly damaged, however components distant from direct flame contact and high temperature may also be damaged or distorted due to issues such as unusually large, or differential, thermal expansion of connected components.

The remainder of section 2.0 provides a summary of basic technical information that will assist professional engineers in inspecting and assessing fire damage that has been sustained by a bridge. Additional detailed and essential technical information is provided in the appendices.

## 2.2 - Fire Effects on Structural Steel

The information in this section applies to the structural steel types normally used in FSR bridges, which have a yield strength less than or equal to 350 MPa. These steels are not heat-treated during the original steel material creation process. Steels that are created using a heat-treatment process, such as high-strength bolts, are affected differently by high temperatures during fire events. A subsequent section addresses high-strength ASTM A325 structural bolts that are commonly used on FSR bridges.

As non-heat-treated steel is exposed to increasing temperature in a fire, its strength decreases steadily. By the time the steel reaches 600 degrees Celsius, its strength will be reduced by more than 50%. If the steel does not reach temperatures higher than 600 degrees Celsius, it will regain its strength when the steel cools. Studies have shown that there is little change in post-fire strength properties when heating is kept below about 700 degrees Celsius, however, distortion may occur at a significantly lower temperature.

The presence of distortion can indirectly indicate the post-fire properties of steel. Members heated beyond 700 degrees Celsius will usually show large deflection or localized distortion when heated in a structural system.

The following summary general recommendations apply to the non-heat-treated steel material itself (not considering deformation):

- Steel heated to temperatures at or below 700 degrees Celsius can be considered to have no reduction in strength following fire exposure.
- Steel subjected to temperatures exceeding 700 degrees Celsius but less than 1000 degrees Celsius can be conservatively estimated to have a 10% reduction in both tensile and yield strength. If this is a cause for concern after load rating the post-fire bridge, testing may be indicated to determine a more refined estimate of steel strength.
- If there is evidence that the steel temperature may have exceeded 1000 degrees Celsius, such as evidence of flaking or surface degradation, the fitness for service of the steel requires further evaluation. Note that this is likely an uncommon scenario in most FSR bridge fires.

## 2.3 - Fire Effects on Reinforced Concrete

In general, there is a slow degradation in compressive concrete strength up to 400 degrees Celsius followed by a more pronounced reduction at higher temperatures. The post-fire strength of concrete does not recover like the post-fire strength of steel when the material cools down.

The thermal conductivity of steel is about 30 times greater than concrete, therefore, concrete tends to heat up internally much slower when exposed to the same surface temperatures. This effect is beneficial to prevent or delay strength loss in concrete beams and bridge decks.

Concrete deterioration will generally be limited to surface areas, with the depth of deterioration being dependent on the fire temperature and duration.

Spalling will be either visually obvious or detectable using non-destructive techniques following a fire event. Spalling will not be a hidden problem, therefore strength consequences can be addressed in terms of section loss and effects on the development capacity of the reinforcing steel.

The response of mild steel reinforcement in concrete is similar to non-heat-treated structural steels. Minimal post-fire reduction in strength is expected as long as the temperature does not exceed 700 degrees Celsius. Since there will also be severe concrete damage at this temperature it is unlikely that there will be any "hidden" reinforcing bar strength loss that needs to be considered in post-fire strength evaluation.

This document does not discuss fire damaged prestressed concrete since it is rarely used for FSR bridges. If a prestressed concrete FSR bridge is fire affected, the evaluating professional engineer should refer to the publications listed in the reference appendix.

If concrete strength loss is suspected, destructive testing is the preferred approach to accurately quantify this effect. This typically involves coring cylinders from the affected members and performing standard compression tests. This can determine if the concrete meets its required strength capacity. Initial subjective determination of strength loss of concrete should be undertaken with a hand-held hammer in order to determine areas of deteriorated and softened concrete. Mechanical field testing of concrete hardness may also be worthwhile.

It is sometimes possible to determine the temperature to which concrete was heated by its color. Concrete which has been heated and then cooled and is not discolored probably was not heated above about 300 degrees Celsius. If the concrete has become pink, it may have been heated to a temperature between 300 degrees Celsius and 600 degrees Celsius. Concrete heated above 600 degrees Celsius and then cooled tends to become a whitish-gray, and above 900 degrees Celsius some concrete turns to a buff (pale yellow-brown) color.

## 2.4 - Fire Effects on Wood

The flash point for wood (temperature at which wood will burst into flame) is 300 degrees Celsius (ref. 9).

As wood burns, marked zones of degradation become apparent (if a specimen is thick enough). Wood exposed to temperatures in excess of 300 degrees Celsius will form a residual char layer. Any charred layer of a wood member has no residual load capacity. A layer of wood below the char layer is subject to some thermal degradation because of exposure to elevated temperature. This wood layer has been estimated to be approximately 30mm thick, based on limited research.

The ignition and flammability characteristics of water-borne preservative (e.g., CCA, which is frequently used for ministry bridge component preservation) treated wood are considered to be similar to those for untreated wood except that water-borne preservative treated wood is known to smoulder longer than untreated wood which results in greater total damage to the treated components.

Oil-borne preservative (e.g., creosote, which was historically frequently used for ministry bridge component preservation) treated wood can be much more flammable than untreated wood (ref. 5).

## 2.5 - Fire Effects on Other Components

### Bearings

It is common to see bridges that have bearing damage and possible contact damage with abutment walls following fire events. The thermal expansion (longitudinal and transverse) can be much higher than anticipated by normal bridge temperature changes. Expansion at the bridge ends reflects the integration of the thermal expansion occurring at every section of the bridge. It is therefore possible to see bearing and joint problems away from the locations directly affected by fire. Therefore, all expansion joints and bearings, regardless of location, require thorough inspection and evaluation following a significant fire event. Much of the bridge expansion present at peak temperature will recover as the bridge cools. Bearings may have suffered damage at the peak expansion point that may not be apparent after the bridge contracts. This is a particular concern for elastomeric or other non-mechanical bearing types.

See ministry guide: Protocol for the Inspection of Bridge Bearings

### Welds

Expansion and contraction of steel during heating and cooling during a fire event may introduce cracks in some weldments and introduce higher residual stresses. Distortion and high forces due to thermal expansion can cause cracking or distress of welds.

Weld fatigue life can be divided into two phases, a crack initiation phase and a crack propagation phase. Most of the crack life is spent in the initiation phase where no finite size cracks are present at a detail. Fire events cause very large amounts of thermal expansion in some steel members. This results in high forces in members that are interconnected. In some cases, these forces can overstress fillet welds and introduce localized cracking. If such cracks are present, it shortens the initiation phase and can lead to premature fatigue failure. It is therefore important to perform a "hands-on" inspection of fillet welds in steel bridges that have had substantial exposure to fire.

### Bolted Connections

ASTM A325 structural bolts are heat-treated steel products that depend on controlled heating, and cooling, to obtain their high strength properties during manufacturing. Lacking more precise information, it is recommended that sample bolts from critical connections in primary members that may have experienced temperatures exceeding 200 degrees Celsius should be tested.

### Coatings

Bridge coatings can experience different degrees of damage during fire events, depending on the fire intensity and the coating type. When temperatures exceed 400 degrees Celsius coatings will typically be severely damaged or destroyed.

## 2.6 - Melting/Ignition Point of Common Bridge Component Materials

Table 1 provides approximate melting/ignition temperatures for commonly used bridge materials. This information may be useful for inspecting professional engineers because it can verify that certain parts of a bridge reached or exceeded specific temperatures if the components are observed to have melted or burned. When considering the table below, professional engineers should bear in mind that components may be damaged to the point of no repair even if the materials they are composed of have not melted or burned up.

For reference when reviewing the table below, it is helpful to remember that forest fires can reach temperatures between 800 and 1200 degrees Celsius (as mentioned earlier in section 1.1).

In 2017 wildfires, ministry engineering personnel observed roadway signs, in some locations, which had burnt wooden posts with un-melted aluminum signs (see photo 6 in appendix 6). Based on the table below, this observation would indicate that the signpost and sign reached temperatures between 300 and 660 degrees Celsius.

In other locations in the 2017 wildfires, ministry personnel observed "completely melted signs that were just puddles of aluminum on the ground next to the steel telespar post." At these locations, the signs and posts must have reached temperatures between 660 and 1500 degrees Celsius, based on the table below.

Indicators of component temperatures during a fire event, such as the two examples above, may be helpful for a professional engineer when they are evaluating the capacity of components that may not have clear visible indicators of the temperatures that were reached during the fire.

**Table 1: Melting Point for Common Bridge Materials**

| Material | Approximate Melting / Ignition Point (degrees Celsius) |
|---|---|
| Rubber (e.g., bearings) | 100 to 200 |
| Plastic (e.g., rebar chairs, "Big O" culverts) | 100 to 200 |
| Wood | 300 (ignition) |
| Zinc (e.g., galvanized coatings) | 420 |
| Aluminum (e.g., delineators) | 660 |
| Structural Steel | 1500 |

## 2.7 – Repair Options for Bridge Components

This section provides suggestions and references related to some repair options for bridge components.

### Steel Repairs

References 11 and 12 provide detailed information related to heat-straightening of deformed steel bridge components. In some cases, the addition of new steel members for strengthening may be more economical and simpler than heat-straightening existing deformed components.

### Concrete Repairs

Reference 13 provides detailed information about concrete repairs specific to FSR bridges. Other references listed in Appendix 7 provide detailed examples and recommendations in relation to concrete repairs for fire damaged bridge components.

**Fibre-Reinforced Polymer (FRP) Repairs**

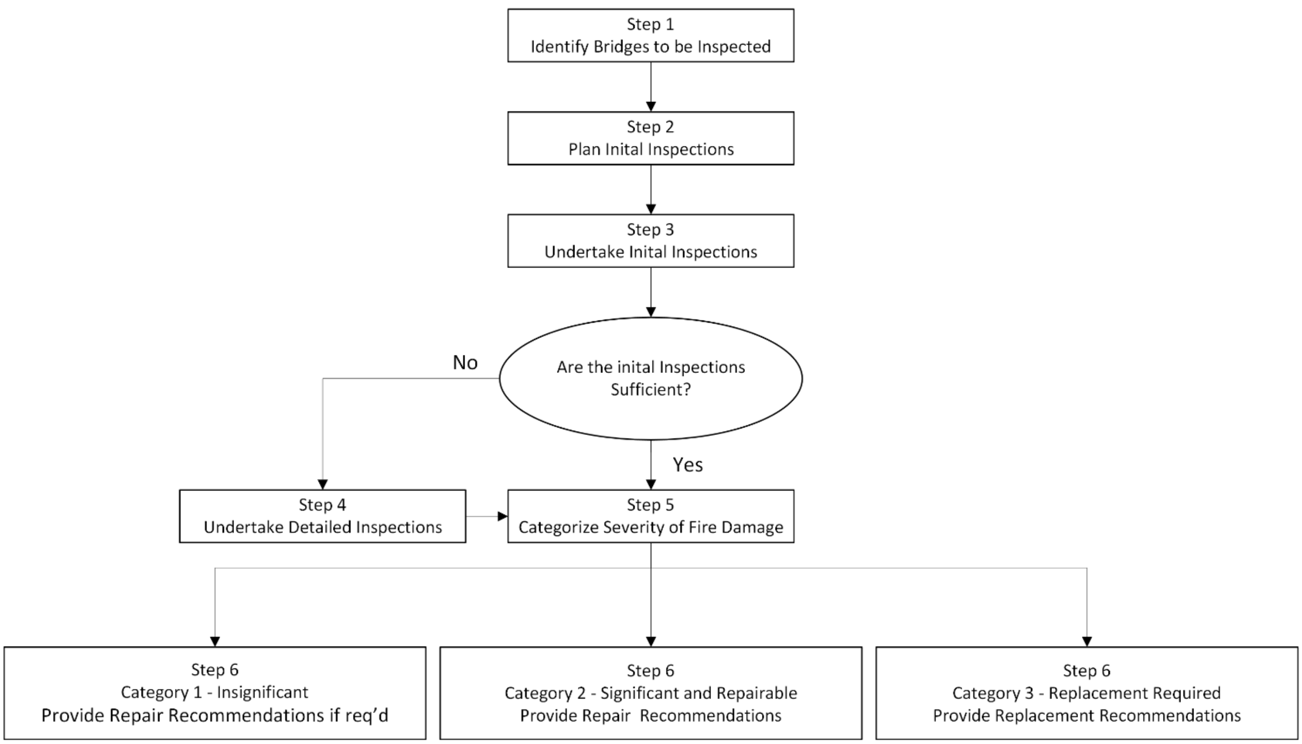
In some cases, FRP repairs may be worth consideration, however fire damaged components may prove to have special challenges in this regard. Special investigation of FRP repair manuals (not referenced in this document) would need to be undertaken.

# 3.0 - Post-Fire Bridge Assessment Procedures

This section describes a six-step procedure that can be followed when a professional engineer is required to assess a fire affected bridge. A flowchart (Figure 1) is provided which visually indicates the sequencing of the steps.

Each fire affected bridge requires an initial on-site inspection to be undertaken (described as Step 3 below). In a situation where an initial inspection is insufficient for a professional engineer to make full recommendations regarding the bridge, a subsequent detailed inspection will be required (Step 4). If the initial inspection provides sufficient information for the professional engineer, Step 4 will be unnecessary.

**Figure 1: Procedures Flowchart**




## 3.1 - Step 1 (Identify Bridges to be Inspected)

Specific bridges affected by fire may be brought forward by ministry staff or others for evaluation.

In the case of wildfires, communication with ministry staff in the relevant Natural Resource District office and /or British Columbia Timber Sales (BCTS) office is generally expected to provide adequate information in order to identify a list of FSR bridges that may have been impacted. Additional wildfire impacted structures may be discovered when doing the field work required for the initially determined list of bridges.

## 3.2 - Step 2 (Plan Initial Inspections)

### Bridge File Review

Previous inspections and record information should be reviewed prior to the inspections in order to understand the pre-fire condition of the structures.

### Tools and Equipment

Standard routine bridge inspection tools and equipment should be arranged. Additionally, tools and materials for cleaning fire affected bridge components to allow inspection should be prepared.

### Personnel

At minimum, the inspection team should consist of one professional engineer and one assistant.

## 3.3 - Step 3 (Undertake Initial Inspections)

### Safety

Follow standard FSR bridge inspection safety requirements.

Additionally, be aware of reduced strength of fire damaged components to support an inspector, and risks of falling debris that may be loosely attached to the structure after the fire which may be prone to falling due to wind, inspector activities, and other causes.

In active wildfire zones, which may still be in effect long after a wildfire is at its peak, access approval is required from the BC Wildfire Service for individuals inspecting fire damaged bridges.

### Use of Routine Bridge Inspection Forms

The bridge inspections should be undertaken fully utilizing the appropriate ministry "routine" bridge inspection forms. The ministry routine bridge inspection forms are available in Section 14 of the ministry's Bridge Standards Manual (BSM).

The ministry has several electronic versions of the routine bridge inspection forms that may alternatively be used.

Notes, measurements, photos, sketches, etc. should also be documented, specific to the fire damage.

### General Focus of the Inspection

The main focus of the inspection is to evaluate structural damage done directly by the fire, however all other bridge concerns also need to be identified through the use of the standard routine inspection form, since they have considerable influence on the final recommendations for repair or replacement of a structure or of components in the structure.

Particularly in the case of wildfires, the inspector should assess whether the likelihood of occurrence of scour and debris flows has increased post-wildfire, as is common (ref. 8), and should evaluate whether this poses a significant threat to the bridge structure in the near future.

In order to adequately inspect various bridge components, special cleaning techniques may be required to remove soot, char, deteriorated coatings, and other debris.

### Checklist of Items for Special Attention

The standard ministry routine bridge inspection forms contain complete lists of items to be inspected. Table 2 provides an ancillary checklist of typical items that require particular attention when undertaking a post-fire inspection.

**Table 2: Checklist of Items for Special Attention in a Post-Fire Inspection**

| **Steel** | | |
|---|---|---|
| **Item #** | **Description** | **Comments** |
| **1** | Paint or Galvanized Coatings | |
| **2** | Non-skid Epoxy Deck Coatings | |
| **3** | Guardrails | |
| **4** | Welds | |
| **5** | Bolts | |
| **6** | All-Steel-Portable Bridges | |
| **7** | Steel I-Girders | |
| **8** | Bracing and Diaphragms | |
| **9** | Sheet Steel Abutments and Retaining Walls | |
| **10** | Piles and Posts | |
| **Concrete** | | |
| **Item #** | **Description** | **Comments** |
| **1** | Approach Barriers | |
| **2** | Deck | |
| **3** | Slab Girders | |
| **4** | Pre-Stressed Girders (rare) | |
| **5** | Compo-Girders | |
| **6** | Epoxy Coated Rebar | |
| **7** | Plastic Rebar Chairs | |
| **8** | Substructures | |
| **Wood** | | |
| **Item #** | **Description** | **Comments** |
| **1** | Approach Barriers | |
| **2** | Curbs | |
| **3** | Deck | |
| **4** | Superstructure | |
| **5** | Substructure and Retaining Walls | |
| **Miscellaneous** | | |
| **Item #** | **Description** | **Comments** |
| **1** | Elastomeric Bridge Bearings | |
| **2** | Signs (including Delineators) | |
| **3** | Geogrid and Geotextiles | |
| **4** | Utilities on Bridges | |
| **5** | Increased Scour Susceptibility | |

## 3.4 - Step 4 (Undertake Detailed Inspections if Required)

If the initial inspection concludes that specialized equipment and/or personnel are required in order to adequately assess and quantify the damage that a bridge has sustained, a more detailed inspection, which may include taking samples of components for testing, needs to be undertaken. This section summarizes recommendations related to such a detailed inspection.

If a substantial amount of time will elapse between an initial and a detailed inspection, the professional engineer undertaking the initial inspection may need to provide immediate interim recommendations related to load rating, posting, or closing a bridge in order to ensure public safety in the time period between the initial inspection and the final engineering recommendations that will be provided following the detailed inspection and assessment.

**Equipment Required**

If routine bridge inspection equipment is not adequate to undertake the required level of inspection following a fire, additional equipment will be required in order to undertake a detailed inspection. This could include access equipment such as a snooper truck, long ladders, fall protection systems, among other equipment. It could also include specialized sampling or testing equipment for activities such as concrete coring, structural steel sample collection, bolt extraction, weld inspection, or evaluating the density, hardness, or interior make-up of various bridge components.

**Personnel Required**

A professional engineer should lead all detailed inspections, with help from at least one assistant. In some cases, individuals with specialized training will be required for the operation of specialized access equipment (e.g., snooper truck), or for specialized fall protection activities (e.g., rope work). Specialized individuals and/or companies may also be required for sampling and testing activities, such as concrete coring, non-destructive concrete testing, and steel testing or sampling.

**Subsequent Lab Testing if Required**

Possible lab testing may include steel yield and tensile strength, steel hardness testing, steel metallurgical analysis, concrete core strength testing, concrete core petrographic analysis, and other testing.

## 3.5 - Step 5 (Categorize Severity of Fire Damage)

Upon completion of a satisfactory inspection (either initial or detailed), the severity of fire damage sustained by a bridge or a bridge component can be categorized as follows:

**Category 1 - <u>Insignificant</u>**
There has been little or no damage sustained. No repairs, or only minor repairs, are required due to the fire.

**Category 2 - <u>Significant & Repairable</u>**
Damage sustained is moderate/significant but can be economically and practically repaired.

**Category 3 - <u>Replacement Required</u>**
The bridge, or a component of the bridge, is damaged beyond the point where it can be economically or practically repaired therefore the bridge, or a specific bridge component, requires replacement.

**Distortion of Steel Members**

Category 1 steel members will appear to have only slight deformations, not easily detected by visual observations (ref. 15). Category 2 steel members will have noticeable visual deformations that can be economically repaired. Category 3 steel component members will have noticeable visual deformations that cannot be economically repaired, resulting in the need for member or entire bridge replacement.

A professional engineer may consider acceptable deformation values for steel members that are specified in various codes for welded steel structures in order to obtain an impression of fully acceptable minor distortions in new structures that have not been involved in a fire. The tolerances allowed for fire damaged steel components will typically be significantly higher than these values, with the ultimate acceptable values needing to be determined on a case by case basis by the evaluating professional engineer considering the exact component, the location of the deformation, the structural system, the required load capacity of the member, and other factors.

### Summary Table of Material Damage Categorization Examples

Table 3 provides some theoretical examples of categorization of damage for various common bridge materials. The example estimated temperatures, example appearance, and corresponding repair/replacement concepts do not describe many of the other possibilities/combinations of temperatures, post-fire appearances, and repair concepts that are possible. An inspecting professional engineer must use judgement related to evaluation of visible fire effects, and should review the appendices of this document, including any relevant reference documents, in order to perform a satisfactory evaluation for a specific situation.

**Table 3: Summary of Material Damage Categorization Examples**

| Material Type and Damage Category | Example Max. Component Temp. (degrees Celsius) | Example Post-Fire Appearance | Possible Repair/Replacement Concept |
|---|---|---|---|
| **Structural Steel (Fy<=350 MPa)** | | | |
| Category 1 | < 600 | No distortion, flaking, pitting, or other visible damage | Ok as is |
| Category 2 | < 600 | Moderate repairable distortion; no flaking, pitting, etc. | Heat straightening, or reinforcing with new members |
| Category 3 | > 600 | Severe distortion, flaking, pitting, etc. | Replacement |
| **Concrete** | | | |
| Category 1 | < 200 | Minor spalling or cracking; no discolouration | Ok as is |
| Category 2 | > 200 | Moderate spalling, cracking or discolouration | Concrete repair may be practical |
| Category 3 | > 400 | Moderate to severe spalling, cracking and discolouration | Replacement may be required, if repair is not practical |
| **Wood** | | | |
| Category 1 | < 300 | No charring | Ok as is |
| Category 2 | > 300 | Minor exterior charring | Reduction in load rating may be acceptable |
| Category 3 | > 300 | Severe exterior charring and substantial loss of section | Replacement |
| **Elastomeric Bearings** | | | |
| Category 1 | < 100 | Minimal or no distortion or damage | Ok as is |
| Category 2 | N/A | N/A | Repair of elastomeric bearings is likely not practical |
| Category 3 | > 100 | Severe distortion or damage | Replacement |
| **A325 Bolted Connections** | | | |
| Category 1 | < 200 | No distortion or damage of bolts or structural steel components | Ok as is |
| Category 2 | N/A | N/A | Repair of bolts is not practical |
| Category 3 | > 200 | May have minimal visible bolt deterioration, but damage to other components is visible | Requires strength testing of sample bolts to determine necessity of replacement |

## 3.6 - Step 6 (Provide Repair or Replacement Recommendations)

The bridge inspection and assessment should conclude with final recommendations for the bridge. If no detailed inspection is required, the initial inspection report should include a completed routine bridge inspection form plus any additional information required to elaborate upon the unusual fire damage circumstances. If a detailed inspection is required, a routine bridge inspection form should be completed, plus a substantial professional report should also be produced describing the situation and the special inspection procedures undertaken. Whether the final inspection is the initial inspection or a subsequent detailed inspection, it should contain detailed recommendations for repair and replacement of bridge components and load rating and posting for the structure.

# Appendices

# Appendix 1 - Bridge Behaviour and Capacity During and After a Fire

The information in this appendix has been excerpted from **Reference 4b**. Excerpts were chosen that were deemed to be particularly helpful for inspection and assessment of FSR bridges. Minor modifications to the original layout and content have been made in order to suit the particular requirements of this document.

## General Bridge Behaviour in Fire Events

Knowing the high temperature structural response of bridges is useful for predicting behavior during and after fire events. The maximum bridge deflection at high temperatures is determined by thermal expansion effects, reduced material strength, the reduced modulus of materials at high temperatures, and the effects of creep. When the structure cools after the fire event, a substantial amount of this deflection recovers. It is even possible in some cases to have some positive residual camber if localized yielding occurs during the fire. If the deflection recovers, the geometry of the bridge is still suitable for its intended traffic use. Any effect on load rating needs to be determined based on a survey of localized damage and post-fire material properties.

The most important engineering problem to evaluate is the post-fire strength and serviceability of the bridge structure. Any permanent deflections will be obvious and their impact can be assessed without the need for high temperature modeling.

There is substantial information available that can be used to predict post-fire material properties based on the temperature reached during a fire event.

The amount of expansion that occurs at high temperatures is dependent on how the material is constrained. Free expansion occurs when the material has no restricting boundary conditions and no stress is developed. If free expansion is constrained due to boundary conditions or surrounding cooler material, stress develops in the material instead of strain.

If a member is constrained to prevent expansion, the strain will be zero and the change in temperature will create stress. Members in a bridge system will be subject to constraint from the surrounding members they are attached to. This constraint creates stress that can cause distortion, buckling, or crushing of concrete.

Large deflections do not occur immediately, it takes many minutes for deformation to develop. The bridge materials heat slowly under fire exposure and the mechanical properties decrease proportionally. Eventually the materials do not have sufficient strength to support the gravity loads on the bridge.

For bridges without structural members above the deck level, fires contained on bridge decks do not cause any significant heating of the bridge members below the deck. Most of the heat is directed upward and the deck concrete has relatively low thermal conductivity. This protects the underlying members from heat.

Although it may seem somewhat obvious, the vertical clearance of the bridge over the fire has an effect on the likelihood of damage and deflection. As a general guideline, girders with continuous direct flame impingement will reach much higher temperatures compared with girders experiencing no flame impingement or intermittent flame impingement.

Bridges that are free to expand at the ends will experience larger vertical deflection compared to those with end restraint. However, bridges with end restraint develop high axial stress in the bridge that leads to buckling and/or cracking distortion. In some cases, a restrained bridge can actually deflect upward initially due to the differential expansion of the deck and girder flanges.

## Temperature Related Mechanical Properties of Steel

The yield strength of steel is greatly reduced as steel is heated in a fire. Some data collected by Kodur is shown in Figure 6 along with the predictive models from the ASCE and the Eurocode. The y-axis shows the yield strength at high temperature normalized by the yield strength at room temperature. There is a wide scatter in the data that has been reported by different researchers for different steel grades.

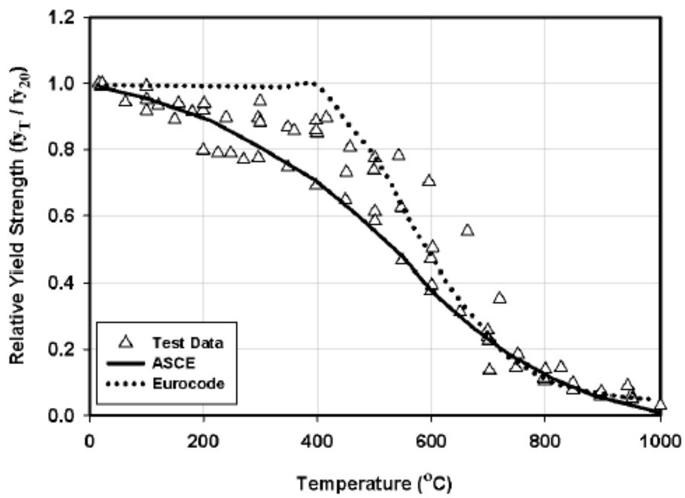



*Figure 6 Reduction in yield strength of structural steel at high temperature.*

The elastic modulus of structural steel decreases with temperature similar to the yield stress.

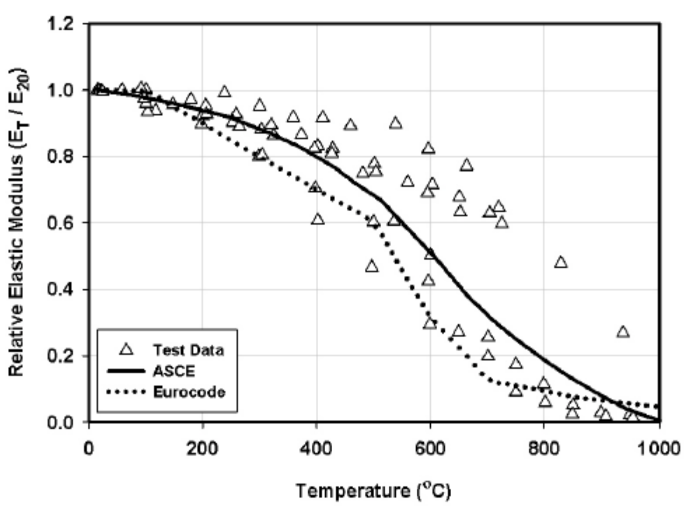



*Figure 7 Reduction in elastic modulus of structural steel at high temperatures*

In addition to changes in yield stress and modulus, the shape of the stress-strain curve changes significantly at high temperatures.

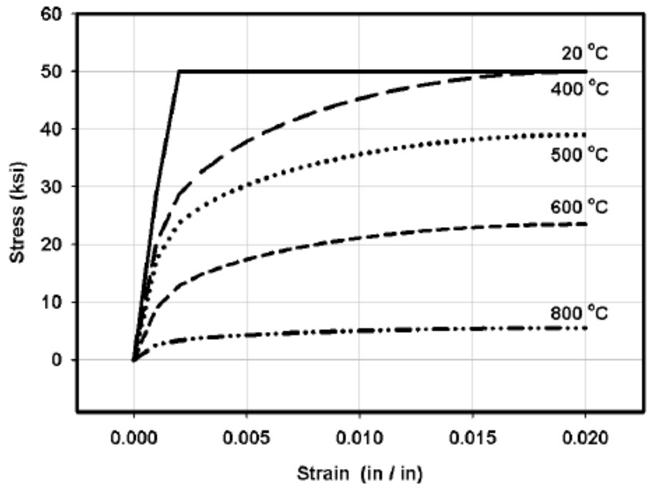



*Figure 8 Change in shape of the stress-strain curve for Grade 50 steel based on the Eurocode model.*

## Post-Fire Steel Properties for Non Heat-Treated Steel

Assuming a structure does not collapse in a fire event, the residual properties of steel must be understood to evaluate the post-fire safety of the structure. It is possible for steel to sustain damage in the fire event that will alter its residual properties. However, damage is only expected for cases with extreme fire exposure. The extent of damage depends on the intensity and duration of the fire, the geometry of the structure, the type of material, and the type of load on the members. The best way to assess structural integrity after a fire is to measure the material properties. However this requires destructive testing that may not be warranted or practical in most cases. This is because common structural steel grades generally do not show much, if any, post-fire strength reduction unless they reach very high temperatures approaching the melting point.

Studies have shown that there is little change in the post-fire strength properties when heating is kept below about 700°C. The British Steel study showed that even after extreme heating to 1000°C, there was at most a 10% reduction in post-fire strength for A36 type material. BS 5950 recommends re-use of steel as long as the distortion is within tolerable limits. BS 5950 also recommends that 90% of the nominal yield strength should be used for post-fire structural evaluation.

Statistically, the average yield strength of undamaged structural plate is about 8% higher than the nominal strength used for design. Therefore, even if there is a slight post-fire strength reduction, most steel members will still exceed their nominal design strength. The bridge may still meet its required load rating with a slightly reduced yield strength, depending on what limit states govern the design. The degree of structural redundancy should also be considered. The modeling results show that all girders are not equally heated during large fire events. Engineering judgment should be applied to each specific fire event and the 10% BS 1950 recommendation should not be applied without due consideration of all factors involved. The strength reduction should only be invoked for the very large fire events where heating above 700°C is clearly suspected for a majority of members comprising the redundant load path.

The presence of distortion is another piece of evidence that can indirectly indicate the post-fire properties of steel. The AISC report indicates that members heated beyond 700°C will usually show large deflection or localized distortion when heated in a structural system (12). Such distortion may require remediation or repair that lessens the need to evaluate strength. The models run in the NCHRP 12-85 research showed substantial web buckling when the steel temperature exceeded about 600°C. There was also a tendency to have lateral buckling of the bottom flange when the end boundary conditions restrained longitudinal expansion of the bridge. This supports the observations of the previous studies related to distortion. Distortion may be even more significant in bridge fires compared to buildings. Welded plate girders used in bridge members typically have non-compact webs compared to compact webs in most rolled beam members. The girder depth also tends to be greater in bridges. These effects are expected to magnify web distortion during fire. Fabricators are well aware of how heat application can induce web distortion in bridge members, even when the heating is held below 700°C. This is exasperated by the presence of composite concrete decks since the deck remains much cooler that the web and bottom flanges during bridge fires. This temperature gradient imposes significant restraint to web expansion, therefore local web buckling is expected in many heating scenarios.

Extreme overheating, beyond the rolling temperature, can be expected to reduce steel properties to a larger degree. At these temperatures, carbon and other strengthening alloys can be "baked" out of the steel, thereby reducing the available alloy content. There will be evidence of pitting and flaking on the steel surface if this degree of extreme heating has occurred. Fortunately, this may be a relatively impractical situation for most bridge fire scenarios. Steel will lose almost all of its strength at these temperatures. Severe bridge fires typically engulf members or significant portions of members. Large distortions and permanent vertical deflection or collapse can be expected before extreme temperatures are reached in most cases. Extreme temperatures can be locally introduced by oxy-fuel torches or other extreme heat sources but these temperatures can be considered atypical for bridge fire events.

Based on the information generated in the NCHRP 12-85 project and that available in the literature, the following recommendations are made for evaluating the strength of structural steel with Fy <= 345 MPa:

- Steel heated to temperatures at or below 700°C can be considered to have no reduction in strength following fire exposure.
- Steels subjected to temperatures exceeding 700°C but less than 1000°C can be conservatively estimated to have a 10% reduction in both tensile and yield strength. If this is a cause for concern after load rating the post-fire bridge, testing may be indicated to determine a more refined estimate of steel strength.
- If there is evidence that the steel temperature may have exceeded 1000°C, such as evidence of flaking or surface degradation along with the presence of abnormally high temperature heating, the fitness for service of the steel requires further evaluation. Note that this is an extremely unlikely scenario in bridge fires and should only be considered for abnormal circumstances.

Following the same logic used for strength, if the temperature is less than 700°C, no significant change in microstructure is expected and any effect on toughness is expected to be minimal. There may even be an improvement in some circumstances.

On the downside, expansion and contraction of steel during heating and cooling may introduce cracks in some weldments and introduce higher residual stresses. Because of the lack of control on heating and cooling during a fire event, engineers should not count on any beneficial effects induced by the fire. Distortion and high forces due to thermal expansion can cause cracking or distress of welds.

Some bridge members, such as those classified as fracture critical, may present a higher concern for CVN toughness evaluation after reaching high temperatures in a fire event. In these cases, destructive CVN testing may be the only option available to determine fitness for service.

## Post-Fire Steel Properties for Heat-Treated Steel

Heat-treated steels require extra caution in the post-fire strength evaluation process. ASTM A709 grades HPS 50W, 50W, HPS 70W, and HPS 100W have higher hardenability compared to low-alloy structural steels. This indicates that the mechanical properties are more dependent on the heating and cooling rate history. Grades HPS 70W and HPS 100W rely on some form of heat treatment in the manufacturing process to develop mechanical properties. Quenching, where the steel is heated to a critical temperature and rapidly cooled, provides a boost in strength and a noticeable reduction in ductility and toughness. Quenching is usually followed by tempering, a process where the steel is heated to a lower temperature and allowed to slow cool. Tempering restores ductility and toughness while preserving some of the strength gain provided through quenching. Other forms of thermo-mechanical processing also rely on precise control of heating and deformation to achieve properties.

Grades HPS 50W and 50W are not classified as heat-treated steels since no heat treatment is used in the manufacturing process. However, the chemistry is very similar to heat-treated grades (70W, and HPS 70W) that add heat treatment to boost properties. From a strength perspective, grades 50W and HPS 50W can be expected to have little or no strength loss or mechanical property loss following fire exposure. They can be expected to perform similar to low alloy steels. However, rapid cooling during the fire process, such as from direct exposure to water from fire hoses, may have a quenching effect on the steel resulting in strength elevation. In some circumstances this could be considered good, but there may be an accompanying loss of ductility and toughness. In general, HPS 50W and 50W steels should be evaluated as low alloy steels for strength and toughness. If there is evidence that the steel was subjected to high cooling rates from heavy application of water, further evaluation of toughness may be warranted. As a general guideline, a direct, concentrated stream from a fire hose where the water is converted to steam on contact may be considered as heavy water application. Diffuse spray or side spray from water directed away from the girders should not be considered heavy application.

In general, the AWS D1.5 Bridge Welding Code allows heat forming operations for quenched and tempered steels as long as the heating temperature falls below the tempering temperature used in manufacturing. AWS conservatively limits the maximum temperature during heat forming operations to 600°C for the HPS grades. As long as the temperature stays below this level, no significant change in material properties is expected. However, if there is suspicion that the temperature may have exceeded this level, more thorough evaluation of the steel is indicated. Higher temperatures, exceeding 700°C, have the potential to significantly reduce the strength properties that were developed through heat treatment.

Based on the information gathered in the 12-85 project and that available in the literature, the following recommendations are made for evaluating the strength of heat-treated structural steel with
$F_y > 345$MPa:

- Heat-treated steel heated to temperatures at or below 600°C can be considered to have no reduction in strength following fire exposure.
- Heat-treated steels subjected to temperatures exceeding 700°C are expected to have a significant loss of strength and require further evaluation.
- Heat-treated steels that reach temperatures between 600°C and 700°C may experience strength loss or toughness degradation and should be considered for further evaluation.
- If there is evidence that the steel temperature may have exceeded 1000°C, such as evidence of flaking or surface degradation along with the presence of abnormally high temperature heating, the fitness for service of the steel definitely requires further evaluation. Note that this is an extremely unlikely scenario in bridge fires and should only be considered for abnormal circumstances.

High strength bolts should be classified as heat-treated products for strength evaluation. Grade A325 and A490 structural bolts are heat treated during manufacture to obtain their mechanical properties. Heating above 600°C may alter the tensile strength and ductility of the bolts. Any bolted connections designed for strength should be subjected to further evaluation if it is suspected that the girder temperature exceeded 600°F at the connection location. Unlike base plates, bolt strength is relatively easy to evaluate by removing a sampling of bolts from the connection and testing the bolts. Possible effects on slip-critical connections are considered as a serviceability limit state.

## Mechanical Properties of Concrete

Similar to steel, the properties of concrete change significantly at high temperatures. However, concrete properties are much more variable depending on the material composition, moisture content, and other material differences.

The thermal conductivity of steel is about 30 times greater than concrete. Therefore, concrete tends to heat up internally much slower than steel when exposed to the same surface temperatures. This effect is very beneficial to prevent or delay strength loss in concrete beams and bridge decks.

The strength of concrete cylinder tests at elevated temperature is the most widely tested and reported property in the literature. In general, there is a slow degradation in compressive strength up to 400ºC followed by a more pronounced reduction at higher temperatures.

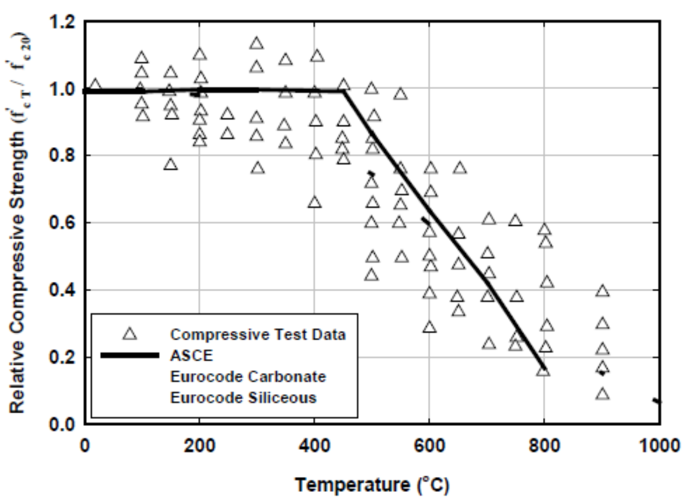



*Figure 13 High temperature compressive strength of normal strength concrete.*

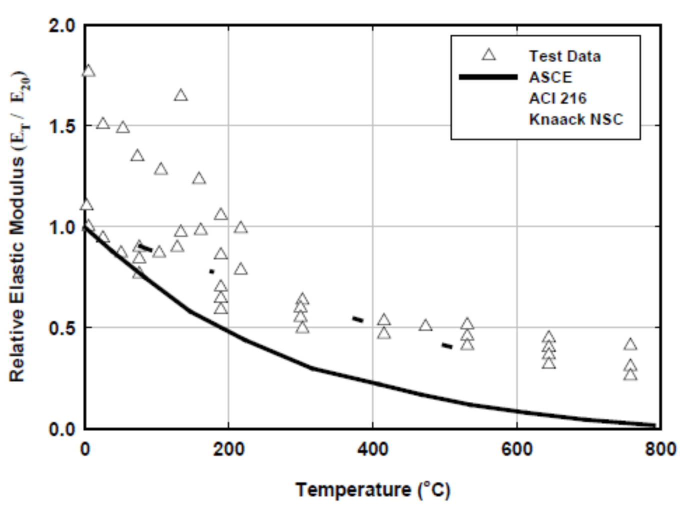



*Figure 15 Variation of elastic modulus for normal strength concretes.*

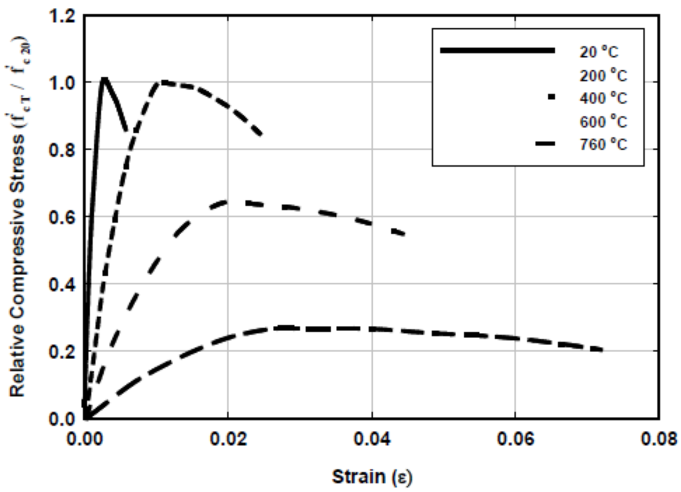



*Figure 16  ASCE temperature-dependent stress-strain models.*

## Post-Fire Concrete Properties

The post-fire strength of concrete does not recover like the post fire strength of steel when the material cools down.

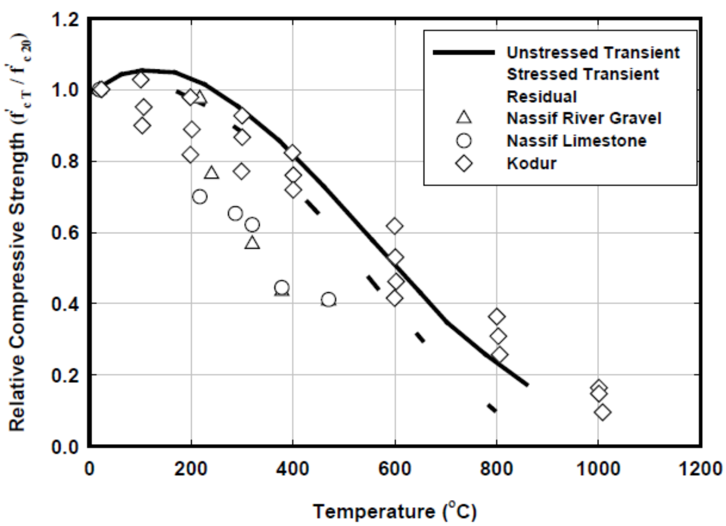



*Figure 24  Post fire compressive strength of normal strength concrete.*

The tensile strength of concrete is not usually relied upon for strength design, however it is important for understanding shear behavior. There is limited research on the tensile strength of concrete at elevated temperatures. Reduced tensile strength can lead to increased cracking in members that are allowed to go into tension in service.

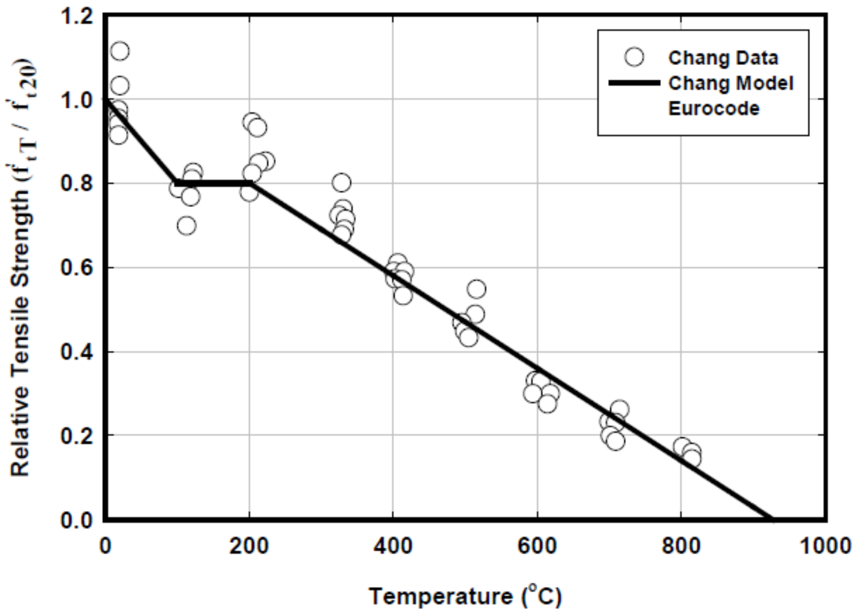



*Figure 25 Post-fire tensile strength of concrete as a function of temperature.*

There are many proposed models for spalling available in the literature. While these models are useful for prediction of the onset of spalling, they have little relevance for post-fire evaluation of structures. In general, spalling will be either visually obvious or detectable using NDE techniques following a fire event. This will not be a hidden problem, therefore the strength consequences can be addressed in terms of section loss and effects on the development capacity of the reinforcing steel.

The response of mild steel reinforcement in concrete is similar to structural steels. No post-fire reduction in strength is expected as long as the temperature does not exceed 700°C. Since there will also be severe concrete damage at this temperature it is unlikely that there will be any "hidden" reinforcing bar strength loss that needs to be considered in post-fire strength evaluation.

Prestressing strands, however, can experience a notable loss of strength following fire exposure. There can also be a significant relaxation of prestress force following the fire event. The loss of post-fire strength does not begin until strands have been heated to temperatures approaching 400°C. High strength strands rely on heat treatment and mechanical working during manufacturing to achieve their strength. Heating above 400°C begins to undo the effect of these processing methods. Figure 27 shows the post-fire (residual) strength prediction models in the Eurocode compared to models developed by MacLean and Hertz. As long as the maximum temperature reached during the fire is accurately known, the post-fire strength can accurately be estimated from this figure.

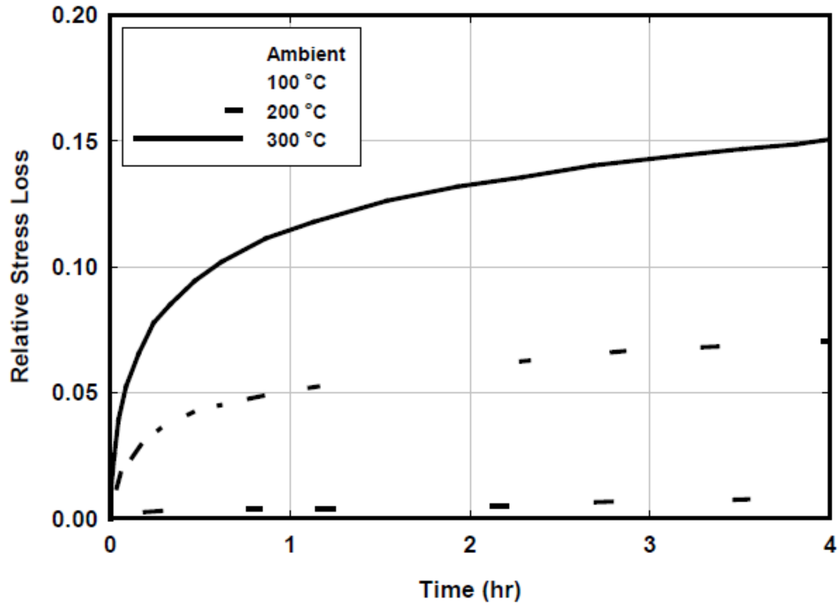



*Figure 26 Stress loss over time for strands heated to various temperatures after being pretensioned to 70% of the tensile strength (0.7 $f_{pu}$). At temperatures above 300°C the strands failed.*

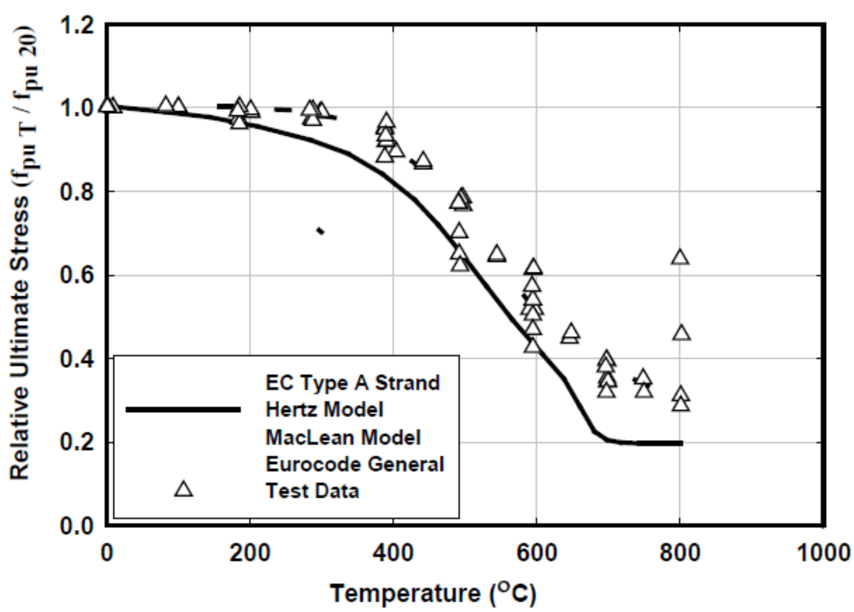



*Figure 27 Summary of residual strength test data reported by MacLean (2007).*

Estimation of temperature in concrete members is more important than steel members since the expected concrete damage is permanent. The prestress loss and the post fire strength of strands is also very dependent on temperature. Because of the large through-thickness temperature gradients in concrete members, these effects will be different depending on internal location. Direct evaluation of internal concrete strength requires destructive evaluation of concrete cylinder cores. This can be difficult to perform in prestressed and reinforced concrete members with high reinforcement density at critical locations.

The low thermal conductivity of concrete results in substantially lower temperatures internally as the distance from the surface increases.

These high surface temperatures predict a large loss of compressive and tensile strength of the concrete indicating severe surface damage. Of more critical concern is the internal temperature and damage at the location of the prestressing strands. The relatively low thermal conductivity of concrete limits the internal temperature to about half the surface temperature at a depth 2" above the bottom flange surface. Table 3 shows that the internal temperature continues to decrease at deeper depths.

*Table 3. Approximate internal temperature observed in the middle third of concrete girder bridge span exposed to a tanker fire at Location A.*

| Girder | Surface Temperature (°C) | Internal Temperature at 2" Cover (°C) | Internal Temperature at 4" Cover (°C) | Internal Temperature at 6" Cover(°C) |
|---|---|---|---|---|
| 1 | 830 | 425 | 200 | 180 |
| 4 | 1050 | 525 | 270 | 180 |

## Post-Fire Strength Evaluation

Post-fire inspection of bridges should begin with an overall mapping to identify the regions that have potential heat damage. These regions should be relatively obvious to engineers. The location of the fire is known and there will be extensive visual evidence of soot, paint distress, concrete discoloration, concrete spalling, and possibly localized distortion of web plates and secondary bracing members. In the most severe cases, there may be evidence of large vertical structural deflection. Correlations can be made between the appearance of the bridge members and documentation of the flame height and location during the fire event. Development of an overall damage map is the first step to developing a plan to proceed with evaluation of the bridge.

No specific guidance on visually mapping fire damage is provided in this guide since appearances can be different depending on the fuel source and coating materials present on a specific bridge. In general, the overall damage mapping should define regions of the structure into at least three categories relative to fire exposure: 1) regions with no direct contact with flames; 2) regions with continuous flame contact; and 3) regions with intermittent or lapping flame contact. The purpose of this mapping is to focus the post-fire inspection on the regions with the highest potential for material damage. Regardless of fire exposure, physical damage may exist in other parts of the structure due to thermal expansion and distortion of members. Therefore, a detailed structural inspection of the entire structure is indicated, regardless of the damage map. The presence of physical cracking and distortional damage can be readily assessed by detailed visual inspection and the consequences of such damage can readily be considered by engineers.

### Destructive Testing of Steel

Destructive testing is the most accurate way to determine the residual properties of steel after fire. Performing tension tests according to ASTM E6 procedures provides an accurate assessment of the post-fire strength and ductility of steel. However, this requires removing samples of material from the web and flange plates, often in critical structural areas. The cost of testing therefore may be large since structural repairs may be required following testing. Sometimes it may be possible to test splice plates, stiffeners, or other secondary members without causing damage that is costly to repair. Consideration needs to be given if the secondary member material is different than the primary member material of concern. It is also possible to test sub-size specimens that can be machined from a 4 in core drilled from the material. Sub-size specimens can introduce a size effect that must be considered when comparing results to the standard size specimens used for structural steel testing.

**Observations and Non-Destructive Testing of Concrete**

Degradation of the concrete cross section due to delamination and/or spalling can be easily detected by a detailed post-fire inspection. Spalling where the concrete has fallen off is easy to visually detect. Sounding the concrete with a hammer can detect delaminations that indicate the early stages of spalling. Data is available in the literature relating the color of concrete to post-fire compression capacity. A pinkish hue generally serves as an indication of altered material properties. Tests such as rebound hammer may also be useful to evaluate concrete strength.

Cracking in tension areas of prestressed concrete members provides a strong indication that there is a loss of effective prestressing force. Open cracks will be accompanied by measurable vertical deflection and are a clear sign that the prestressing force has been compromised. Tight cracks with no apparent vertical deflection are more difficult to evaluate since they may have been formed by thermal expansion at high temperature. In this case, visual crack observations should be corroborated with other assessment means such as temperature estimation.

**Destructive Testing of Concrete**

If concrete strength loss is suspected, destructive testing is the preferred approach to accurately quantify this effect. This typically involves coring cylinders from the affected members and performing standard compression tests. This can determine if the concrete meets it's required strength capacity. The available locations for coring may be limited in prestressed members in regions with close strand and reinforcement spacing. For design, concrete strength is measured at the 28 day point in the curing process and it is well known that strength continues to increase with age. It is typical for tests performed on older age concrete to show strength significantly exceeding f'c . This is an asset for post-fire strength evaluation. Even with a slight reduction in capacity relative to the pre-fire strength, the concrete strength may still exceed the nominal design requirements.

Full scale tests were performed on concrete box beams removed from a bridge in Connecticut following a large fuel spill fire event. The beams had extensive surface spalling and the strength capacity was in question. Physical tests performed at the FHWA Turner-Fairbank Highway Research Center determined that the post-fire capacity still exceeded the design capacity. Results from relatively undamaged beams were compared to heavily exposed beams and no significant difference could be found. At least for this case, this supports a conclusion that concrete members, particularly those that have surface damage, may in fact still have adequate strength for continued service.

**System Strength Evaluation**

Steel members recover most of their strength when they cool following a fire event. This also applies to concrete reinforcing steel and strands. The compression strength of concrete does not recover following heating and may be slightly decreased below the high temperature strength. However the relatively short duration of most bridge fires and the thermal properties of concrete usually results in minimal internal material damage for concrete members. The tension capacity of steel generally will have the most effect on strength, therefore structures can be expected to have higher strength after the fire than during the fire event. Structures that appear to have only minor deflections and damage will usually be safe to allow access for inspection.

The modeling study shows that there is a period of stable, increasing vertical deflection, before girder-type bridges can reach a state where complete collapse is possible. Deflections, therefore, serve as a warning for collapse.

Other bridge types, such as trusses and cable-suspended structures may not exhibit deflections prior to collapse. It depends on the forces present in the members that are affected by the fire. Pure compression members, such as compression chords in trusses or steel column bents are vulnerable to reduced buckling capacity at high temperature. The consequence of such buckling depends on the redundancy provided by the bridge system. Wood structural members are also vulnerable to failure without significant deflection. The conclusion that there will be deflection before collapse only applies to typical steel and concrete beam bridge superstructure types.

**Load Rating Procedures**

Load rating procedures for the post-fire structure are the same as for the undamaged structure. The only differences relate to any detected reduction in material properties or prestress force. Any collision damage or physically apparent distortional damage also needs to be considered. As previously discussed, post-fire material properties can be either measured directly or estimated from knowledge of the likely temperatures reached during the fire event.

**Composite Action**

Steel girder bridges can be expected to have higher forces on the composite deck connection compared to concrete girder bridges because steel has a higher coefficient of expansion and can reach higher temperatures during the fire. The modeling study shows that there is considerable shear introduced at the deck-girder interface due to differential thermal expansion during the fire event. This shear may be sufficient to degrade the stiffness of the connection. When stressed to capacity, shear stud connections can degrade through a complex combination of events such as stud yielding, stud weld failure, or localized bearing failure of the concrete around the stud. The post fire capacity of the connection may or may not be affected for ultimate strength. However, there may be a loss of stiffness of the connection that may affect the bridge response to live loads. There is insufficient data to make any conclusions about whether this is or is not a problem for bridges.

Post-fire inspection may reveal physical evidence that the beam to deck connection has slipped during the fire event. This may show up as cracking along the beam deck interface and haunch. It may also be evident at locations where transverse diaphragms or floor beams connect the beams. If there is no evidence of distortion, it can be reasonably assumed that there is no degradation of strength at ultimate capacity.

As a first step, visual examination of the structure should be performed to detect any evidence of slip at the beam to deck interface. If such evidence exists, further evaluation may be indicated. A first step may be simple observation of behavior underneath the structure when live loads are on the bridge. Sometimes movement, either vertical or horizontal can be detected as a truck crosses the bridge.

**Member Distortion**

Local member distortion is a common problem in steel bridges. The differential fire heating typically causes large temperature gradients in the structure that can cause large web distortion. This distortion was clearly evident in the NCHRP 12-85 modeling study and was a challenge that had to be addressed to obtain solution stability in the models. Such distortion will be obvious in a post-fire inspection of the structure.

Web distortion is currently a controversial topic. Levels of distortion typically exist in fabricated beams due to welding distortion, residual stresses, and heat cambering operations. Dimension tolerance limits are currently provided in the AWS D1.5 Bridge Welding Code, but they are largely based on aesthetics. There is no clear consensus on what effect these distortions have on strength. AASHTO now allows consideration of tension field action when evaluating shear strength and the capacity based on current codes may be greater than that calculated in the original design. This provides some relief when calculating the shear capacity of sections with web distortion. It is highly likely that large web distortions will be present following a severe fire event, particularly for welded plate girders. At the present time, the impact of these distortions must be determined on a case-by-case basis by an engineer. A field instrumentation evaluation can be very useful to determine any impact of web distortion on the structural response of the bridge.

Distortion of transverse stiffeners, cross frames, diaphragms, and other secondary members needs to be assessed as required. It may be possible to ignore some deformation of transverse stiffeners if they are still capable of performing their web stiffening and connection functions. The welds near any such deformations need to be carefully inspected. Axial force members such as those used in cross frames or diaphragms often show effects of buckling or distortion following a severe fire event. The distortion typically occurs at peak heating when differential thermal expansion is at a maximum and the member strength is reduced. Some members may undergo a force reversal between the heating and cooling phases of a fire event. A member that is normally in tension may go into compression at high temperatures and revert back to tension when the structure cools. There are many possible scenarios that may be encountered by engineers. Distortion in cross frame members may not have much effect on the response of the bridge system. Many bridges were designed using girder-line analysis procedures that do not consider cross frames to be effective in wheel load distribution. The primary purpose of many cross frames is to provide stability to girders during bridge erection. The effect of cross frame distortion should be evaluated on a case by case basis by an engineer considering that some distortions may not affect bridge performance.

**Bearings & Expansion Joints**

It is common to see bridges that have bearing damage and possible contact damage with abutment walls following fire events. The thermal expansion (longitudinal and transverse) can be much higher than anticipated by normal bridge temperature changes. Expansion at the bridge ends reflects the integration of the thermal expansion occurring at every section of the bridge. It is therefore possible to see bearing and joint problems away from the locations directly affected by the fire. Therefore, all expansion joints and bearings, regardless of location, require thorough inspection and evaluation following a significant fire event. Much of the bridge expansion present at peak temperature will recover as the bridge cools. Bearings may have suffered damage at the peak expansion point that may not be apparent after the bridge contracts. This is a particular concern for elastomeric or other non-mechanical bearing types.

Expansion joints require thorough inspection, again irrespective of their location relative to the fire. The large forces present at peak expansion may have caused compression damage to the joints and their attachment connections to the concrete. Bridges that experience any significant vertical deflection at high temperatures that recover as the bridge cools may cause tensile damage to the joints.

**Connections**

The structural connections in bridges are typically designed as slip-critical to prevent any slip under the SERVICE II load combination. They are also designed as bearing-type connections relative to the STRENGTH I limit state. Luecke (21) reports that the high temperature tensile strength of structural bolts begins to reduce at about 200°C. Additional data on bolt strength is available in Appendix E of the NCHRP 12-85 Research Report. No data is presented about the post-fire residual strength. For strength evaluation, A325 and A490 structural bolts are heat-treated products that depend on controlled heating to obtain their properties during manufacturing. Lacking more precise information, it is recommended that bolts from any connections in primary members that may have experienced temperatures exceeding 200°C should be tested. This can easily be done by removing a few bolts from the connection and testing them in a Skidmore device or in direct tension. Direct tension testing may be easier if the threads are distorted from the initial tensioning. If the bolts are found to have insufficient strength, it is relatively easy to replace the bolts incrementally to restore the connection capacity.

It is a bit more difficult to determine the slip capacity for SERVICE II. Bolts are pretensioned to a large portion of their tension capacity during installation to provide frictional clamping force. Depending on the temperature and duration of the fire event, creep may be expected to allow relaxation of the initial bolt tension. This will reduce the clamping pressure and therefore the slip capacity of the connection. Direct evaluation of bolt tension is sometimes possible using ultrasonic methods but this approach requires calibration information for the bolts being tested. The data for high strength strands indicates that creep is possible at around 200°C. While no such data exists for high strength bolts, it is reasonable to assume that bolts will also begin to lose clamping force at 200°C. Similar to strands, tensioned high strength bolts are heat-treated products and operate under high tension stress.

Fatigue life can be divided into two phases, a crack initiation phase and a crack propagation phase. Most of the life is spent in the initiation phase where no finite size cracks are present at the detail. Fire events cause very large thermal expansion of some steel members. This results in very high forces in members that are interconnected. In some cases, these forces can overstress fillet welds and introduce localized cracking. If such cracks are present, this shortens the initiation phase and can lead to premature fatigue failure. It is therefore important to perform a "hands-on" inspection of fillet welds in steel bridges that have substantial exposure to fire. The same procedures used for fracture critical inspection should be followed. The overstressing effects of thermal expansion may extend beyond the heating area of the fire. Therefore, any parts of the bridge that may be affected by thermal expansion should be inspected. This includes regions in compression that are not normally evaluated for fatigue.

Fire exposure can also potentially change the fracture toughness of steel. Some researchers have reported a slight decrease in ductility for fire exposed steels. Brandt has reported CVN test results showing that fire damaged steels still often exceed the AASHTO fracture toughness requirements for new steels (13). CVN tests were also performed on the I-78 beams in the FHWA study. Again, all of the material tested still exceeded the AASHTO CVN requirements for new bridges. Research on heat straightening of steel members does not show any special concerns for CVN toughness in grade 36, 50, 50W, or 50S steel. Therefore, for typical girder bridges with redundant (non-fracture critical) members there is no evidence to suggest that fire will impair fracture toughness.

Special concerns, however, may be required for fracture critical members exposed to fire. AASHTO has higher CVN toughness requirements for fracture critical members because they are critical to the system strength of the bridge. No cases were found in the literature where fracture critical bridge members were exposed to fire. If CVN toughness is a concern, core samples can be taken and CVN tests performed to evaluate the post-fire toughness of fracture critical members.

**Bolted Connections**

The serviceability of connections needs to be evaluated to determine the slip capacity for SERVICE II. Bolts are pretensioned to a large portion of their tension capacity during installation to provide frictional clamping force. Depending on the temperature and duration of the fire event, creep may be expected to allow relaxation of the bolt tension. This will reduce the clamping pressure and the slip capacity of the connection. Direct evaluation of bolt tension is difficult. The data shows that creep deformation begins to occur at temperatures around 200°C. If it is estimated that this temperature was exceeded for any length of time, it is reasonable to suspect some reduction in bolt pre-tension. The solution to this problem is relatively simple, replace the bolts in the connection. However, the complexity of this operation depends on the surface preparation of the faying surface. For surfaces prepared by abrasive blasting, the bolts can be replaced one by one without loosening the entire connection. However, for connections that utilize class B coatings on the faying surface, coating damage may compromise the coefficient of friction assumed for the connection. In this case, the entire connection should be disassembled and re-constructed. This significantly adds to the cost of connection repair since shoring is required to support the bridge weight when the connection is disassembled.

**Coatings**

Bridge coatings can experience different degrees of damage during fire events, depending on the fire intensity. When temperatures exceed about 600°C, coatings will be severely damaged or destroyed. The required actions are obvious in such cases. Less obvious are cases where the coatings have experienced some heating but are largely intact. Lacking research, the best approach in these cases is to contact the coating manufacturer for recommendations.

**Epoxy Coated Reinforcing Steel**

Internal concrete deck temperatures are much lower than exterior deck surfaces that have been exposed to fire, and in most cases do not significantly exceed the temperature associated with loss of material strength (around 400 °C). However, these temperatures are high enough to cause concern for the epoxy coatings of some reinforcement, and GFRP composite bars that rely on epoxy resins. The glass transition temperature of these materials can be as low as 110 °C.

**Effect on Corrosion Resistance in Concrete Products**

It is difficult to assess the effect of fire on the long term performance of concrete members. One factor is that fire can introduce cracking in bridge decks and members. Another factor is that the permeability of concrete may be impaired. The long-term consequences of subtle damage may take many years after the fire to become evident. No information was found in this study concerning the long-term performance of fire exposed concrete members. One approach may be to only monitor performance through the normal inspection cycle for the bridge. This approach doesn't help if the long term consequences of the fire need to be determined sooner for bridge management or liability concerns.

Considerable information exists in the literature about evaluating the durability of bridge decks. Any methods applicable to normal decks can be equally applied to fire exposed decks and members. Every State DOT has experience with evaluating deck durability and a comprehensive discussion of this topic is beyond the scope of this Guide. A detailed cracking survey should be performed following a significant fire event. Results should be interpreted using the same analysis that is used on non- fire exposed decks. Crack width can be evaluated concerning the expected effect on permeability. If concrete material changes are suspected, the permeability can be evaluated through destructive testing.

One unknown factor is the internal condition of epoxy coatings on the reinforcing steel. There is one known test of a slab with epoxy coated reinforcement and high temperature pull-out tests available in the literature. A deck specimen was tested in a furnace to determine the effect of the epoxy on fire rating. From a high temperature strength and fire rating perspective information is available on the effect of epoxy coatings. No information, however, is available on any possible impairment of the barrier protection capacity of the coating after the concrete has cooled. The best approach is to develop an estimate of the highest internal slab temperature at the location of the coated bars. This can be compared to the glass transition temperature for the epoxy product. Manufactures may be able to provide information related to the coating integrity when this temperature is exceeded. More research is needed in this area.

# Appendix 2 – General Concrete Information

The information in this appendix has been excerpted from **Reference 3**. Excerpts were chosen that were deemed to be particularly helpful for inspection and assessment of FSR bridges. Minor modifications to the original layout and content have been made in order to suit the particular requirements of this document.

**a) Spalls**

Spalls are the removal or loss of concrete from the girder and may range in extent from being minor to severe. With minor spalls the aggregate within the concrete is exposed, whereas moderate spalls are deeper and the prestressing strands or reinforcing steel is exposed. With severe damage, spalls will be deep and the inner structure of the girder is not only exposed but may also be damaged. The location, size and severity of each spall on all the affected girders shall be identified.

**b) Cracks**

A crack is defined as a separation of parts. Cracks are identified according to their width as follows:

- Hairline – less than 0.1 mm
- Narrow – ≥ 0.1 mm and < 0.3 mm.
- Medium – ≥ 0.3 mm and < 1.0 mm
- Wide – ≥ 1.0 mm

The width, location and length of all cracks shall be noted.

**c) Hardness Testing**

Limits of areas damaged by fire can be determined using an impact hammer. An average impact hammer reading should first be obtained in the un-damaged areas for each type of unit. Readings in the damaged areas will generally be substantially lower than those in the undamaged areas. By taking a large number of readings throughout the areas suspected of damage, the severely damaged areas can be identified.

**d) Concrete Cores**

To identify the extent of damage, a sufficient number of cores, as determined by the Engineer, shall be retrieved from concrete areas distressed by heat to perform testing for compressive strength and petrographic analysis. The areas to be tested can be determined by Hardness Testing.

**e) Reinforcing Steel**

Normal reinforcing steel loses yield strength with increase in temperature; the loss is about 50% at 600°C. The modulus of elasticity also reduces significantly at elevated temperatures. However, the original properties are recovered on cooling provided the maximum temperature has not exceeded the austenitic transformation temperature of 720°C for a period long enough for significant grain size coarsening.

**f) Prestressing Strands**

If the concrete temperature exceeded 1200°C, there will be severe damage and it may be necessary to retrieve a sample of the prestressing strand in the damaged area to determine the material and strength properties and compare them with the requirements of ASTM A416.

# Appendix 3 - Colour Change Indicators of Concrete Temperature and Deterioration

The information in this appendix has been excerpted from **Reference 1**. Excerpts were chosen that were deemed to be particularly helpful for inspection and assessment of FSR bridges. Minor modifications to the original layout and content have been made in order to suit the particular requirements of this document.

*Ref. 1 refers to a PCI publication as follows:*

PCI – "Design for Fire Resistance of Precast Prestressed Concrete"2
Chapter 9.4 – Post Fire Examination
It is sometimes possible to determine the temperatures to which concrete was heated by its color. Concrete which has been heated and then cooled and is not discolored probably was not heated above about 600°F. If the concrete has become pink, it may have been heated to a temperature between 600°F and 1100°F. Concrete heated above 1100°F and then cooled tends to become a whitish-gray, and above 1700°F some concretes turn to a buff color.

*Ref. 1 continues as follows with an example inspection of a particular fire damaged concrete bridge:*

Visual inspections of the damage in Span 8 made note of concrete color variations on the soffit of the bottom flanges that corresponded to changes in concrete condition states. Figure 3 contains a map that was produced to display the boundaries of four fire induced color regions. The color regions were described as Extreme-White, Ash-White, White-Gray, and Soot.

EXTREME-WHITE
The Extreme White bottom flange color appeared to represent exposure to the most intense heat directly over the fire source. The boundaries of this region were difficult to see but the region was clearly visible in photos of span 8. This region represented approximately 10% of the Span 8 area and included Girders 8D, 8E, 8F, 8G, and 8H. An elliptical shape directly above the tanker was placed to approximate the size and position of this region. Concrete on the soffit of the bottom flanges crumbled when tapped with a rock hammer. Prestressing strands on the east and west edges of the bottom flange were easily exposed. Concrete spalls were noted on the soffit and both sides of the webs and both sides of the top flanges. Heat deformation was also noted in mild steel in the top flange. The nylon reinforcing chairs in the bottom flanges burned during the fire leaving deep pockets of charred nylon in the flange soffit.

ASH-WHITE
The Ash-White bottom flange region encompassed approximately 30% of the Span 8 area and included all fifteen (15) girders. The North and South boundaries were relatively easy to see from the ground. Their position was measured from the center of Pier 9. Concrete on the soffit of the bottom flanges crumbled or spalled when tapped with a rock hammer.
Prestressing strands on the edges of the bottom flange were easily exposed. Concrete spalls were noted in the webs and the top flanges. Web spalls occurred only on the side facing the heat source, due to shadowing effects from the adjacent girders. The density and extents of spalling on the webs appeared to increase with distance from the heat source. This is probably another demonstration of shadowing effects that occurred near the heat source. The girder webs furthest away from the fire were actually more exposed to the radiant heat than were the webs nearest to the fire. Web spalling and top flange spalling did not occur outside of this color region. The nylon reinforcing chairs in the girders had melted with some indication of being burned during the fire. Melted nylon formed stalactites and pockets of charred nylon were visible.

WHITE-GRAY

The White/Gray bottom flange region encompasses approximately 30% of the Span 8 area and included all fifteen (15) girders. This region was characterized by obvious delamination sounds in the bottom flanges when struck with a rock hammer. Concrete spalling could be produced with forceful strikes using a rock hammer. The concrete did not crumble and the spalled pieces were 6 inches to 12 inches in size. Prestressing strands on the edges of the bottom flange were difficult to expose using a rock hammer. The web and top flanges were covered with soot but appeared to have sound concrete. The nylon reinforcing chairs in the girders had melted during the fire leaving long nylon stalactites.

SOOT

The Soot colored region encompasses approximately 20% of the Span 8 area beginning at about 6 feet from Pier 8 and extending to Pier 9. The region outside of the White/Gray boundary was dark gray in color. The concrete appeared sound but delamination sounds were evident on the edges of the bottom flange when struck with a rock hammer. The nylon reinforcing chairs in the girders melted during the fire leaving marks on the bottom side of the bottom flange, stalactites were not seen in this area.

# Appendix 4 - Wood Component Information

The information in this appendix has been excerpted from **Reference 5**. Excerpts were chosen that were deemed to be particularly helpful for inspection and assessment of FSR bridges. Minor modifications to the original layout and content have been made in order to suit the particular requirements of this document.

### a) Treatment considerations

The ignition and flammability of water-borne preservative treated wood (eg. CCA, which is frequently used for ministry bridge component preservation) is considered to be similar to untreated wood. Oil-borne preservative treated wood (eg. creosote, which was historically frequently used for ministry bridge component preservation) can be much more flammable than untreated wood (ref. 5).

### b) Burn Characteristics

The flash point for wood (temperature at which wood will burst into flame) is 300 degrees C (ref. 9).

As wood burns, marked zones of degradation become apparent (if a specimen is thick enough). Wood exposed to temperatures in excess of 300 degrees C will form a residual char layer. Any charred layer of a wood member has no residual load capacity. A layer of wood below the char layer is subject to some thermal degradation because of exposure to elevated temperature. This wood layer has been estimated to be approximately 30mm thick, based on limited research. The normal wood (located beneath the zone of elevated temperatures) remains unaltered from the fire. The timber member cross-section sketch below shows these layers (Ref. 5).

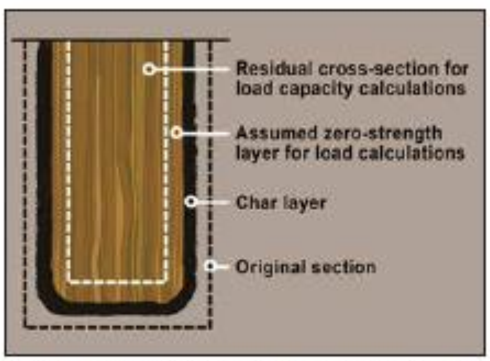



Figure 5—Reduced section approach to fire-damaged wood.

### c) Residual Capacity of Burnt Wood Components

The residual load capacity of a partially burnt wood member can be estimated by assuming the member size is equal to the full charred member size minus the depths of charred layers and also minus assumed 30mm thick layers of thermally degraded wood directly beneath the charred layer. This methodology may provide a reduced load capacity for some sawn wood members. This methodology could possibly be utilized, with additional complications, for residual load capacity calculation for glulam girders. Additional complications arise because outer plies of the original glulam girders were frequently fabricated with higher quality laminations compared to the interior plies of the girder. The majority of ministry glulam girders were creosoted, therefore it is quite possible that the strength of the fire fueled by creosote wood would not leave a residual member with capacity sufficient for re-calculation.

### d) Inspection Details

In addition to focusing on fire damage, an inspector needs to find and document all areas of wood deterioration, including rotten and insect infested areas.

Because metal can conduct heat into the interior of a wood component, assessment of fire damage around connections may be less feasible for visual inspection (ref. 5).

# Appendix 5 - Celsius/Fahrenheit Comparison Table

| Celsius | Fahrenheit |
|---|---|
| 0 | 32 |
| 100 | 212 |
| 200 | 392 |
| 300 | 572 |
| 400 | 752 |
| 500 | 932 |
| 600 | 1112 |
| 700 | 1292 |
| 800 | 1472 |
| 900 | 1652 |
| 1000 | 1832 |
| 1100 | 2012 |
| 1200 | 2192 |
| 1300 | 2372 |
| 1400 | 2552 |
| 1500 | 2732 |
| 1600 | 2912 |
| 1700 | 3092 |
| 1800 | 3272 |
| 1900 | 3452 |
| 2000 | 3632 |

## Appendix 6 – Fire Damaged FSR Bridge Photos

1) Bridge 61-102 (2017 fire)


Photo 1a - Slight damage to ballast wall, curb, and end of one tie


Photo 1b - Cap burnt between girders and under bearing plate

2) <u>Bridge 61-046 (2017 fire)</u>


Photo 2 - Glulam bridge destroyed

3) Bridge 64-001 (2017 Fire)

Commentary: Several ties and deck planks burned. Fire may have been extinguished with water by crews. No girder damage was detected in the inspection. Decking was repaired and the bridge was re-opened to normal loads.

Photo 3a - Timber deck partially burnt

Photo 3b - Close-up view of burnt deck section

Photo 3c - View from under bridge

Photo 3d - View from side of bridge

4) "Scottie" Bridge (2017 Fire)

Photo 4 - Burnt Timber Deck

5) <u>Concrete Bridge (unknown bridge number, approx. 2013, suspected arson)</u>

Photo 5a - Fire evidence seen at top of bridge

Photo 5b - Fire evidence seen on side of bridge

Photo 5c - Melted rebar chairs on deck soffit

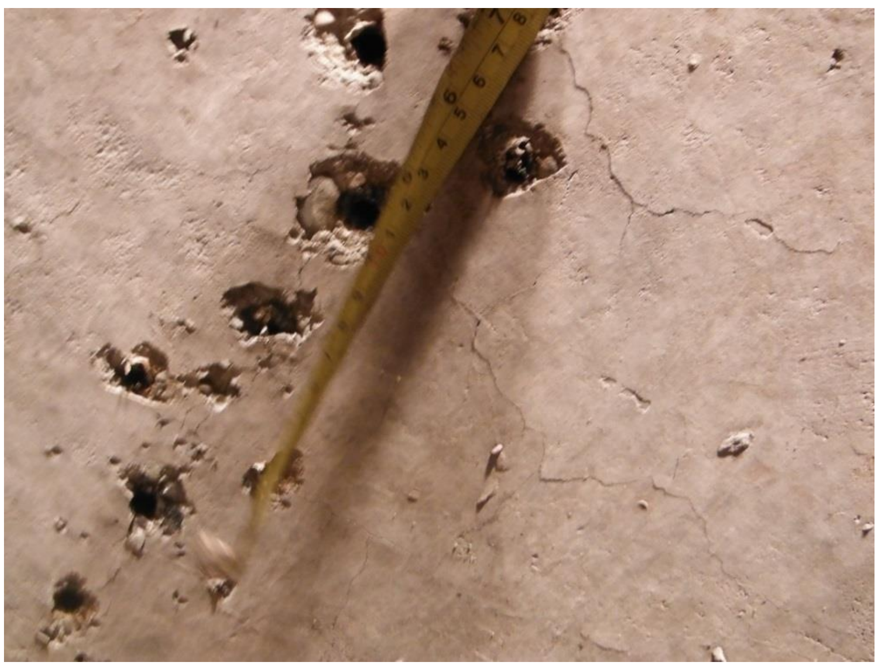


Photo 5d - Concrete spalling and cracking (presumably from fire)

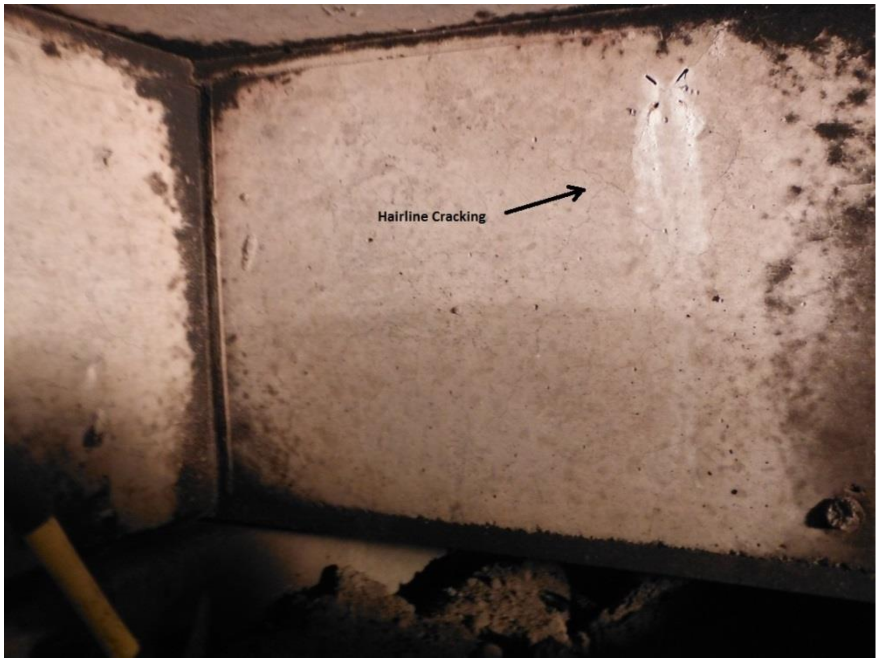



Photo 5e - Hairline cracking on side of girder (top right quadrant of photo)

Photo 5f - Overall view of damage seen from under the deck

6) <u>Miscellaneous Component Damage (2017 Wildfire)</u>

Photo 6 - “Bridge Ahead” sign: fire damaged post

7) Miscellaneous Component Damage (2017 Wildfire)

Photo 7a - Roadway collapse due to melted plastic culvert

Photo 7b - Removed melted plastic culvert

Photo 7c - Roadway collapse near inlet/outlet due to melted plastic culvert

# Appendix 7 – List of Reference Documents

1) Stoddard, Richard; Inspection and Repair of a Fire damaged Prestressed Girder Bridge. Washington State DOT. 2004. 27 Pages.
http://aspirebridge.com/resources/Stoddard_WA%20Fire_IBC_04.pdf

2) Brandt, Thomas et al., Effects of Fire Damage on the Structural Properties of Steel Bridge Elements. University of Pittsburgh for Pennsylvania DOT. 2011. 61 Pages.
http://www.dot7.state.pa.us/BPR_PDF_FILES/Documents/Research/Complete%20Projects/Maintenance/Effects%20of%20Fire%20Damage.pdf

3) Waheed, Abdul et al., Repair Manual for Concrete Bridge Elements; Alberta Infrastructure and Transportation. 2005. 22 Pages.
http://www.transportation.alberta.ca/Content/docType30/Production/RpMConcBrEl2.pdf

4) Wright, William, et al., NCHRP 12-85: **a)** Highway Bridge Fire Hazard Assessment. 2013. 492 pages. **b)** Draft Guide Specification for Fire Damage Evaluation in Steel Bridges. 2013. 75 Pages. Virginia Polytechnic Institute and State University
http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2715

5) Kukay, Brian et al., Evaluating Fire-Damaged Components of Historic Covered Bridges. U.S. Forest Service. 2016. 40 Pages.
https://www.fs.usda.gov/research/treesearch/52667

6) Fire Damaged Reinforced Concrete- Investigation, Assessment and Repair. VicRoads Technical Note No. 102. 4 Pages.
https://www.vicroads.vic.gov.au/~/media/files/technical-documents-new/technical-notes/technical-note-tn-102--fire-damaged-concrete.pdf

7) Davis, Martha, et al., Bill Williams River Concrete Bridge Fire Damage Assessment. Structure Magazine, July 2008. 3 Pages.
http://www.structuremag.org/wp-content/uploads/2014/08/SF-Bill-Williams-Bridge-Fire-Assessment-July-081.pdf

8) Landslide and Flooding Risks after Wildfires in British Columbia. BC Min. of Forests and Range Wildfire Management Branch and Forest Science Program Pamphlet. 2011.
https://www.for.gov.bc.ca/hfd/pubs/docs/bro/Bro91.pdf

9) Gabbert, Bill. Wildfire Today article. 2011
http://wildfiretoday.com/2011/02/26/at-what-temperature-does-a-forest-fire-burn/

10) Garlock, Maria, et al., Fire Hazard in Bridges: Review, assessment and repair strategies. Engineering Structures 35 (2012)

11) Guide for Heat-Straightening of Damaged Steel Bridge Members. U.S. DOT FHA. 2008. 77 Pages.
https://www.fhwa.dot.gov/bridge/steel/heat_guide.pdf

12) Heat-Straightening of Damaged Steel Bridge Girders: Fatigue and Fracture Performance, NCHRP Report 604. 2008. 177 Pages.
https://www.nap.edu/download/23087#

13) Concrete Repair Guide for Concrete Bridge Components. FLNRO. 2017. 17 Pages.
https://www2.gov.bc.ca/assets/download/EEE38657F4994EABAC339CEE04E03070

14) chapter 9: Damage Inspection. Michigan Structure Inspection Manual- Bridge Inspection. 2014, updated 2023. 8 Pages.
https://www.michigan.gov/mdot/-/media/Project/Websites/MDOT/Programs/Bridges-and-Structures/Inspections/MISIM/Chapter-9-Damage-Inspection.pdf?rev=a9c00b7570724b20abfef4dbf06c9978&hash=90700E50D506149A0CB593BA2901EBB9

15) Tide, R.H.R.; Integrity of Structural Steel After Exposure to Fire. Engineering Journal Q1, 1998. 13 Pages.
http://files.engineering.com/download.aspx?folder=e3ecce14-4a0e-4ca0-a089-8a99e0e01e67&file=Integrity_of_Structural_Steel_After_Exposure_to_Fire.pdf