RESPONSE OF FIRE EXPOSED STEEL BRIDGE GIRDERS

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ABSTRACT

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Fire is one of the most severe environmental hazards to which civil structures may be subjected during their lifetime. In recent decades, due to rapid development of urban ground-transportation systems, as well as increasing transportation of hazardous materials, bridge fires have become a growing concern. Steel structural members which are widely used in bridges exhibit lower fire resistance as compared to concrete members due to rapid degradation of strength and stiffness properties at elevated temperature. Therefore, behavior of steel girders under fire conditions is of critical concern from fire safety point of view.

Unlike structural members in buildings, no specific fire resistance provisions (active or passive) are required for bridges as per specifications in AASHTO and other standards. Currently, there is no approach to evaluate fire resistance or residual capacity of steel girders after fire exposure. Also, fire provisions used for building elements might not be directly applicable to bridge elements due to different fire exposure scenario, load level, support conditions, and sectional characteristics.

To overcome some of the current drawbacks, a research program involving both experimental and numerical studies on the fire response of steel bridge girders is undertaken. Three steel-concrete composite girders were tested under simultaneous loading and fire exposure to study the behavior of steel bridge girders. Test variables included; load level, web slenderness, and spacing of stiffeners. Results from fire tests
indicate that typical steel girders can experience failure under standard fire conditions in about 30-35 minutes and the response is highly influenced by web slenderness, and type of fire exposure.

As part of numerical studies a finite element model was developed in ANSYS for tracing thermal and structural response of steel bridge girders under fire conditions. Test data generated from fire experiments were utilized to validate the finite element model. The validated model was applied to carry out detailed parametric studies to quantify critical factors influencing fire response of steel bridge girders, namely fire scenario, exposure scenario, load level, span length, web slenderness, presence of stiffeners, and degree of axial and restraint stiffness. Also, as part of numerical studies, a methodology for evaluating residual capacity of fire exposed steel bridge girders was developed.

Results from the parametric study show that steel bridge girders can experience failure in less than 20 minutes under severe fire exposures, such as hydrocarbon fires. Under such fire scenarios, failure through web shear buckling is the most dominant failure limit state especially when web slenderness exceeds 50. Fire resistance and failure mode is highly influenced by fire intensity, exposure scenario, web slenderness, load level, and span length.

Results from the parametric studies are utilized to develop a strategy for enhancing fire resistance of steel bridge girders. The strategy mainly comprises of applying fire insulation, of different types and configurations, on steel bridge girders to achieve 1 to 2 hours of fire resistance. The information and strategy developed in this dissertation can be utilized to enhance fire resistance of steel girders and thus fire hazard to steel bridges can be mitigated to a great extent.
This dissertation is lovingly dedicated to my Mother. Her support, encouragement, and constant love have sustained me throughout my life. Also, it is dedicated to my brothers, sisters, and friends who have always been with me during various phases of my life.
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CHAPTER ONE

1. INTRODUCTION

1.1 General

Bridges are an integral part of transportation routes for facilitating flow of traffic over natural obstacles or constructed facilities. Recent trends of urbanization and higher traffic demand has led to an increase in number of bridges on highways to minimize interference between the routes. The presence of bridges allow free flow for railroads and freeways resulting in less commuting time, reduced traffic jams and carbon emissions. Hence, bridges form key elements in a highway transportation system and its failure means the failure of the entire route network (Barker and Puckett, 2007).

In recent decades, due to increasing transport of hazardous materials such as flammable materials, spontaneously combustible and poisonous materials, bridge fires have become a growing concern. Fire in bridges can lead to long traffic disruption and significant economic losses. Traffic on a bridge, damaged by a fire, is usually hard to detour and might affect the traffic quality in the entire region. Further, a severe fire may lead to permanent damage or even collapse of the bridge. Therefore, fire is one of the most severe hazards to which a bridge may be subjected during its lifetime.

The most common cause in many of bridge fire incidents is crashing of fuel transporting trucks and burning of gasoline in the vicinity of the bridge. These gasoline
fires are much different than those fires in building and are representative of hydrocarbon fires. These hydrocarbon fires are much more intense as compared to building fires and grow at a rapid heating pace and produce higher peak temperature in which the maximum fire temperature is achieved within first few minutes. In some cases, such intense fires can pose a severe threat to structural members and can lead to collapse of structural members of a bridge depending on critical factors including; fire intensity and characteristic of structural members.

Structural members in bridges are typically made of conventional materials such as concrete, and steel. Steel is widely used in bridge construction due to number of advantages steel possesses, including; higher strength, ductility, and cost considerations. However, steel structural members exhibit lower fire resistance as compared to concrete members due to rapid rise in steel temperatures resulting from high thermal conductivity, low specific heat, and lower sectional mass of steel. Therefore, mechanical properties (strength and stiffness) of steel are very sensitive to elevated-temperatures and steel structural members can lose their load carrying capacity rapidly under fire condition. Furthermore, factors such as high-temperature creep that occur at temperature above 600°C can produce high deformations in steel girders and lead to collapse of girders. Therefore, steel bridges can be vulnerable to fire induced collapse. However, unlike in buildings, no specific fire safety provisions are required for bridges as per current code provisions. Also, to date, there is no approach to evaluate residual capacity of steel bridge girders after fire exposure.

1.2 Magnitude of Fire Hazard in Bridges

While the common perception may be that it is very unlikely that a bridge can
collapse under fire conditions, a recent US-wide survey by the New York state department of transportation has shown that bridge fires are a serious concern and nearly three times more bridges have collapsed due to fire than earthquakes during 1960-2008 period (NYDOT, 2008). This survey on bridge failures carried out across 18 states in US and studied the type of bridge, material type, and cause of bridge collapse. A summary of this survey is illustrated in Figure 1.1. As per the survey, a total of 1746 bridges collapsed for various reasons, where collapse is defined considering serviceability limit state. Although the vast majority of bridges (1006) collapsed for hydraulic reasons (scour, flood) and 515 bridges collapsed due to collision, overload, or deterioration, a total of 52 bridge collapses were due to fire, and only 19 collapses were due to earthquake (seismic states like California participated in this survey). About half of the bridges collapsed due to fire were timber bridges that were damaged by wildfires. This leaves about 27 incidents nationwide where fire caused permanent damage to the bridges.

1.3 Bridge Fire vs. Building Fire

Fire is a rare event but when it occurs, fire can pose significant threat to life, buildings and environment. The main objective of providing fire safety measures is to minimize loss of life and property in a fire event. In buildings, fire safety is achieved through active and passive fire protection systems that are to be provided to mitigate adverse impact of fire. The passive fire protection measures, namely fire resistance, is the duration during which a structural member exhibits acceptable performance with respect to structural integrity, stability and temperature transmission (Buchanan, 2002). When fire grows to significant size, structural performance becomes critical to minimize collapse of structures since structural integrity is the last line of defense. While provision
for appropriate fire safety measures is a major design requirement in buildings, no specific fire safety provisions is required for bridges since bridges are open air structures and life safety is not considered to be a major concern.

The effect of various influencing factors such as member configuration, connections, material properties, and failure limit states on the fire resistance of building elements have been well studied, while no research has been done on the fire performance of typical structural members commonly used in bridges which may be exposed to more severe and rapidly-forming fires. However, the information available in the literature on fires in building elements might not be directly applicable to structural members in bridge due to differences in member characteristics and fire severity (Payá and Garlock, 2012). As a result, the response of bridge structural members under fire can be different than those in buildings due to the following reasons:

1. Fire source: The common source of ignition in bridges is caused by crashing of gasoline tanker trucks and burning of gasoline in the vicinity of the bridge. However, the common source of ignition in buildings is due to burning of combustion materials (mostly wood and plastic based products) in the compartment.

2. Fire ventilation: Most of building fires are in an enclosed compartment, and are limited by the amount of ventilation. However, bridge fires generally burn in an open air condition and have unlimited supply to ventilation (oxygen).

3. Fire severity: Bridge fires can be much more intense than building fires (ISO 834) and are representative of hydrocarbon fires (see Figure 1.2), since the fuel source is generally from gasoline present in trucks or automobiles involved in
collision.

4. Fire protection: In contrast to buildings that are provided with active fire protection systems such as sprinklers, and passive fire protection such as fire insulation, no special fire protection measures are provided in bridges.

5. Failure limit state: Bridge girders (and other structural components) are generally much deeper than beams in buildings and may have quite slender webs. As a result, shear failure due to web buckling is likely to be dominant failure mechanism in bridge girders; while flexural failure is dominant failure mechanism in beams.

6. Connections: The bridge girders are typically supported through bearing of the bottom flange. In contrast, the connections in building are made through the web and/or the flange. These variations in support conditions have an effect on the resulting fire resistance.

1.4 Response of a Steel Bridge Girder under Fire

A typical steel bridge comprise of various structural members namely, piers, abutments, steel-girders with or without stiffeners, lateral bracings, and concrete-slab deck. Steel girders are the main load carrying structural member in bridges that have to be designed to carry bending and shear forces and play a crucial role in transferring the applied loads to the piers and abutments. Piers and abutments are often made of reinforced concrete, while girders, lateral bracings, and stiffeners are made of steel. Under fire conditions, steel girders of bridge are much more vulnerable to fire as compared to piers and abutments that are made of concrete. This is due to the fact that steel girders are not protected with fire insulation and due to high thermal conductivity of
steel that result in rapid rise in steel temperature under fire conditions. As a result, strength and stiffness of steel deteriorate in a faster pace. Therefore, behavior of steel girders under fire conditions is of critical concern from fire safety point of view.

Structural response of a typical steel bridge girder under fire conditions depends on a number of factors including; sectional characteristics, fire scenario, support conditions and load level. The behavior of a typical two spans steel bridge, commonly used in highway transportation system, is shown in Figure 1.3(a and b). At ambient temperature this kind of bridge girders can be modeled as a simply supported. However, under fire conditions, bridge girders not only lose their strength and stiffness due to increasing in temperature but also experience additional restraint forces due to thermal expansion of steel girders beyond the expansion joint of the bridge.

When fire accident occurs under bridges, girders will be subjected to high-temperature as illustrated in Figure 1.4(a). The temperature rise in top flange of girders can be much lower as compared to that of bottom flange. This is mainly due to insulating effect of the concrete slab that dissipates heat from top flange to the slab. Also, the temperatures in the web can be slightly higher as compared to bottom flange and this is because the web is much more slender (lower thickness) than the flanges and this produces rapid rise in web temperatures. The differences between slab and bottom flange temperature result in significant thermal gradients across the girder-slab cross section as shown in Figure 1.4(b).

In the early stage of fire exposure, mid-span deflection increases linearly till first yielding. The time to yielding depends on the temperature progression in the steel section. The mid-span deflection of steel girders at this stage of fire exposure is mainly
due to thermal gradients that induce thermal stresses. Following the yielding, the deflections start to increase at a faster pace as the fire intensity grows and this is due to the deterioration in strength and stiffness properties of steel with increasing temperatures. Once the thermal expansion of the girder exceeds the allowable gaps at expansion joints of the bridge, the ends of the girder will be restrained from expansion. This is due to axial restraint imposed by adjacent structural members as shown in Figure 1.4(c and d). The fire induced axial force causes additional bending moment on the girder due to P-δ effect that result in additional deflections and hence further deterioration in the response of the girder under fire conditions.

With increasing fire exposure time, the mid-span deflections increase at a rapid pace due deterioration of moment and shear capacity of the steel girder as shown in Figure 1.4(c), and spread of plasticity in the web and/or bottom flange of steel girders. Furthermore, the increasing temperature and stresses in steel accelerate the development of high-temperature creep deformations leading to rapid increase in girder deflections. Finally, the failure of the girder occurs through yielding of bottom flange and formation of plastic hinge at mid-span (flexural limit states) if the web have adequate shear strength. Alternatively, failure can occur through buckling of the web (shear limit state) or through interaction of both flexural and shear limit states.

1.5 Objectives and Scope of this Research

To develop strategies for overcoming fire problem in steel bridges, a collaborative research project on the fire response of steel bridge girders is underway between Michigan State University (MSU) and Princeton University (PU). This thesis is undertaken as part of this collaboration, and the following are the objectives of this
thesis:

- Carry out a detailed state-of-the-art review on the fire exposed steel bridge girders and identify knowledge gaps relating to fire response of steel bridges. The comprehensive review will cover both experimental and numerical studies as well as current provisions in codes and standards.

- Develop a numerical model to trace the response of typical steel bridge girders under realistic fire, loading and boundary conditions using the commercially available finite element program. The model for thermal and structural analysis will account for high-temperature properties of materials, geometric and material nonlinearities, as well as nonlinear contact interactions.

- Undertake fire resistance experiments on typical steel bridge girders to generate needed data for model validation on the behavior of steel girders under fire conditions. Also, carry out high-temperature mechanical property tests on structural steel commonly used in bridge applications.

- Validate the above developed numerical model by comparing response predictions from the model with test data obtained from fire resistance experiments on steel bridge girders.

- Carry out parametric studies to quantify the effect of critical factors on the performance of steel bridge girders under realistic fire, loading and restraint conditions.

- Develop an approach for evaluating the residual strength of fire exposed steel bridge girders.

- Develop a strategy to enhance the fire resistance of steel bridge girders.
Figure 1.1: Causes for bridge collapse based on US-wide survey by NYDOT

Figure 1.2: Fire scenarios in bridges and buildings
Figure 1.3: Typical two spans steel bridge girder
Figure 1.4: Response of a typical steel bridge girder exposed to fire
1.6 Layout

The research presented as part of this dissertation is organized into seven chapters. Chapter 1 presents background information on the fire problem in bridges, various factors influencing the fire resistance, and behavior of typical steel bridge girders under fire conditions. Chapter 2 presents state-of-the-art on the fire performance of bridge girders. Bridge fire incidents, previous experimental and analytical studies on fire exposed steel bridge girders, steel beams, and steel-concrete assemblies are reviewed. Chapter 3 presents fire resistance experiments conducted on three uninsulated steel bridge girders. Results from fire tests are used to discuss the response of steel bridge girders under fire conditions. Chapter 4 presents material property tests including high-temperature tensile strength, residual strength, and creep tests of typical steel used in bridges. Results from these tests are used to feed the finite element analysis model. Chapter 5 presents finite element model for both thermal and structural analysis that are developed to predict behavior of steel bridge girders exposed to fire. The validation of the finite element model is also presented in Chapter 5, where response predictions from the model are compared with test data. Chapter 6 presents a methodology for evaluating the residual capacity of fire exposed steel bridge girders. A numerical study and simplified approach on residual capacity of steel bridge girders is also presented in Chapter 6. Results from parametric studies are presented in Chapter 7. Different parameters governing the transient fire response of steel bridge girders are described along with a discussion of the results from the parametric studies. Also, results from parametric studies are utilized to develop a strategy to enhance fire resistance in steel bridge girders. Finally, Chapter 8 summarizes the main findings arising from the current study and recommendations for further research.
CHAPTER TWO

2. STATE OF THE ART REVIEW

2.1 General

Research on fire safety has been focused primarily on buildings, and less attention has been paid to bridges because life safety is not a major concern in bridges. Therefore, there is lack of studies on the response of bridges under fire conditions. The information available on the fire response of structural members in buildings cannot be applied directly on structural members in bridges due to various differences as discussed in Chapter 1. However, fire resistance data on structural members in building can be utilized to gauge the response of bridge structural elements. This chapter provides a state-of-the-art review on the fire problem in bridges, and fire resistance provisions in various codes and standards. Also, experimental and numerical studies on the fire behavior of typical steel beam-concrete slab assemblies used in buildings are presented. Since high-temperature properties are critical for evaluating fire response of steel structures in bridges, the variation of high-temperature material properties of structural steel is also discussed.

2.2 Magnitude of Fire Problem in Bridges

Fires can pose a significant hazard to steel bridges. The magnitude of fire problem in bridges is presented in this section.
2.2.1 Bridge fire statistics

Statistics compiled by National Fire Protection Association revealed that an average of 306,000 vehicle fires occurred per year during 2002-2005 period (Aherns, 2008). These fires caused an average of 520 civilian deaths, 1,640 civilian injuries and $1.3 billion property damage. Cars and trucks accounted for 90% of these vehicle fires. Collisions and overturns were reasons for 3% of these fire events.

Fatal crash statistics by National Highway Traffic Safety Administrations (NHTSA) and Fatal Accident Reporting Service Encyclopedia (FARS) database during 1994-2008 period showed that an average of 36 crash incidents per year lead to bridge fires as summarized in Table 2.1 (William et al., 2013). Half of these incidents occurred on the bridge deck due to impact with parapets and bridge rails. The other half occurred in the vicinity of the bridge and involved impact with piers and abutments. In general, the fires that occur on a bridge deck, cause less damage to the structural members of the bridge as compared to the fires that occur underneath bridges. However, in many cases, large fuel spills over the deck might follow the drainage path, and might cause significant fire underneath the bridge.

The New York state department of transportation carried out a nationwide survey and reported that 1746 cases of bridge collapse occurred across 18 states US during 1960-2008 period for various reasons (NYDOT, 2008). A summary of bridge collapses reported in the survey is illustrated in Figure 2.1. The vast majority of bridges, 1006 collapsed for hydraulic reasons (scour, flood), while 228 and 220 bridges collapsed due to collision and overloading respectively. Deterioration was the reason for collapse of 67 bridges and 56 due to miscellaneous incidents. 52 bridges collapsed due to fire, while only 19 bridges collapsed due to earthquake (seismic states like California also
participated in the survey).

To assess the life safety risk posed by each bridge collapse, NYDOT survey included the number of deaths and injuries occurred due to collapse of bridges. Fatalities were reported in 420 situations as compared to 409 injuries for the overall collapse events. The number of fatalities from 100 collapse events is illustrated in Figure 2.2. It can be seen that most of the fatalities occurred due to collapse of bridges during construction which happened in 13 collapse events. This refers to the fact that the number of deaths is not proportional with the number of collapse. Fire was the second largest cause of death in which 12 fatalities were reported due to fire during the 100 collapse events (William et al., 2013).

In NYDOT survey, collapse is defined considering serviceability limit state. The collapsed bridges are classified in two categories, namely partial and total collapse (see Table 2.2) depending on whether the bridge has partially or totally incapacitated for service after exposure to fire incident. From reviewing other sources of data, most of the bridges listed in total collapse category did not fully collapse during the fire event. However, they were not reopened for traffic due to severe structural damage and had to be replaced.

Review of data from literature shows that actual collapse of bridges during fire incidents occurred only in the cases of MacArthur Maze I-80/880 interchange in Oakland, California, in 2007 and 9-Mile road over I-75 expressway near Hazel Park, Michigan, in 2009 (National Steel Bridge Alliance, 2010, Astaneh-Asl et al., 2009). These bridges were made of steel girders with concrete deck on the top and collapsed due to crashing of gasoline tanker in the vicinity of these bridges. However, in other incidents
such as I-65 bridge in Birmingham, Alabama, and I-95 bridge in Bridgeport, Connecticut, the bridges experienced excessive deflections. As illustrated in Table 2.2, about half of the bridges collapsed due to fire were timber bridges that were damaged by wildfires. This leaves about 27 incidents nationwide where fire caused permanent damage to steel and concrete bridges. These statistics clearly infer that fire do pose a significant hazards to bridges during their lifetime.

2.2.2 Recent bridge fire incidents

In recent years there have been numerous fires in bridges and some of these fires resulted in the collapse of steel girders. However, bridges in other cases either survived the fire or experienced some level of damage. A list of some of major fire incidents that occurred in steel and concrete bridges in the last 15 years is tabulated in Table 2.3 (Kodur et al., 2010 and 2013). The following fire incidents illustrate the magnitude of fire problem in bridges:

On July 15, 2009, a tanker truck carrying 13,000 gallons of flammable liquid was involved in an accident with another truck under the 9-mile road overpass over I-75 expressway near Hazel Park, Michigan. This bridge comprised of ten hot rolled steel girders of a 24 m span which supported a reinforced concrete slab. The intense heat from the burning of flammable gasoline reached about 1100°C and this high-temperature led to weakening of steel girder and resulted in collapse of the overpass as illustrated in Figure 2.3(a). This collapse occurred in about 20 minutes after the start of the fire. It took about 105 minutes for the fire fighters to extinguish the fire. This accident caused millions of dollars of damage. Preliminary recommendations called for rebuilding the entire 9 mile bridge over I-75 and the freeway below also needed some level of repair. It took several
weeks of detours on I-75 and several months for repairing the bridge (National Steel Bridge Alliance, 2010).

Another example of recent fire induced bridge collapse is the one that occurred on MacArthur Maze I-80/880 interchange in Oakland, Californian, on April 29th, 2007. A tanker truck carrying 8600 gallons of gasoline overturned underneath I-580 expressway in which the bridge comprised of six plate girders supporting a reinforced concrete roadway. The firefighters responded to the spot within 14 minutes but the intense fires from the accident resulted in fire temperatures reaching 1100°C. This intense heat lead to deterioration of strength and stiffness properties in steel girders and ultimately the connections at the supports gave away (failed) leading to collapse of two I-580 spans as illustrated in Figure 2.3(b). The failure occurred in 22 minutes after the fire started. Post-fire analysis of the bridge revealed that the failure was due to overstressing of connections under high-temperature effects. This incident cost $9 million to repair the bridge, economic impact of $6 million per day, and took a month to finish the retrofitting (Astaneh-Asl et al., 2009).

2.2.3 Fire safety in bridges

In buildings, fire safety is achieved through provisions of active and passive fire protection systems to mitigate adverse impact of fire. Active fire protection systems control fire by some external action or device such as sprinkler, fire detectors, smoke control system, and fire hoses. This system is used to notify of fire, smoke conditions, slow the progress of fire, and/or suppress fire. Passive systems are built into the structure to prevent spread of fire and collapse, and include; fire barrier (walls), providing fire insulation material, and designing structural systems to maintain integrity and stability
under fire. Unlike buildings, no special fire protection measures are required in bridges. Specifically, while adequate fire resistance provisions are required for structural members in buildings, no fire resistance provisions are required for structural members in bridges.

2.2.4 Current fire resistance provisions in codes and standards

According to National fire protection association, the main objectives of fire safety measures in buildings are to minimize loss of life, property damage, and prevent progressive structural collapse in a fire event (NFPA 5000, 2009). Fire safety objectives in buildings are achieved through a set of active, passive (fire resistance) provisions. For structural members in buildings, fire resistance requirements are specified in International Building Code (IBC, 2009). While, provision for appropriate fire safety measures and fire resistance are a major design requirement in buildings, there no fire resistance or fire safety measures requirements for structural members in bridges specified in design standards and specifications (AASHTO, NFPA 502). However, National Fire Protection Association standard on fire safety in road tunnels, bridges, and other limited access highways states that: “Critical structural members shall be protected from collision and high-temperature exposure that can result in dangerous weakening or complete collapse of the bridge or elevated highway.” (NFPA 502, 2011). Although this statement infers that bridges have to be protected from fire hazards, but no specific guidance is given on how protect bridges from fire.

In many cases, steel structural members in bridges experience some level of damage due to partial exposure to fire or low intensity of fire exposure. The main question that arises in these cases is on the residual load bearing capacity of the structural members after fire exposure. This aspect is important to determine whether the bridge is
safe to reopen, or need some level of repair, or should be replaced. To date, there are no provisions on residual capacity assessment of fire exposed bridge members. However, British code: Part 8 in appendix C, states that steel structural member can be re-used after fire exposure if its mechanical properties have not been significantly changed or the members have not been damaged beyond the tolerances of straightness and shape. Furthermore, structural members which have been distorted or damaged need to be fully assessed to ensure their capacity and stability (BS: Part 8, 1990, BS: Part 2, 1992).

2.3 Factors Influencing Fire Resistance

The response of structural members in bridges under fire conditions can be different than those in buildings due to different fire exposure scenario, load level, support conditions, and sectional characteristics as discussed in Chapter 1. The differences in various factors that can significantly influence the fire resistance of steel bridge girders are discussed here.

2.3.1 Geometric features

Steel structural members in buildings are typically made of carbon steels (A36 and A992) and available in rolled-sections with web slenderness (depth to thickness ratio) range from 30-50. These members are provided with fire protection measures (sprinklers and fire insulation) and connected to adjacent columns in a frame through web and/or flange depends on the type of connection. This is unlike in bridges, where steel sections are typically built-up with web slenderness up to 150 for girders with traverse stiffeners and up to 300 for girder with longitudinal stiffeners depends on the required shear capacity (AASHTO, 2012). These stiffeners are used to increase shear capacity of steel girders through development of tension field action, without increasing the web thickness
that results in higher self-weight (dead load) on the girder. Steel girders in bridges are made of high-strength low-alloy steels and can be designed as statically determinate or indeterminate structures in continuous or single spans. Furthermore, these girders are typically supported through bearing of the bottom flange. To date, steel bridge girders are not provided with any fire protection measures (active or passive).

2.3.2 Loading under fire conditions

In current codes of practice, fire resistance is generally evaluated based on a load level of 50% of ultimate capacity of structural members. Load level is defined as the ratio of the applied loading on the beam under fire conditions to the strength capacity of the beam at room temperature. The load level is evaluated based on number of factors, namely function of the structure, (dead/live) load ratio, safety factors used for design under both room temperature and fire conditions. The load combinations are to be applied on a steel members for design under fire conditions are based on the provisions given in design codes. The ASCE-07 (ASCE-07, 2005) and Eurocode (EC1, 2002) have different specifications for the critical load combination under fire conditions, and these are:

\[
W_{\text{fire(ASCE-07)}} = 1.2 \text{ Dead load} + 0.5 \text{ Live load} + 0.5 \text{ Other live load} \ldots [2.1]
\]

\[
W_{\text{fire(EC1)}} = 1.0 \text{ Dead load} + 0.5 \text{ Live load} + 0.5 \text{ Other live load} \ldots [2.2]
\]

based on these guidelines, typically, the actual load level under fire conditions in buildings is in the range of 50%-70%.

Loading scenario in bridges can be different than those in buildings and this is due to differences in the type of live load acting in buildings as compared to bridges. The live load considered in design of structural members in buildings is mainly from occupancy.
and furniture, while in case of bridges it is mainly from axel loads of trucks and vehicles. In bridges, these axel loads (live load) can be significant at room temperature, however, under fire conditions, the live load could be quite low. This is due to low probability of presence of trucks on the bridge during fire incident. Therefore, load level of 20%-30% under fire condition might be reasonable in case of bridges.

2.3.3 Fire scenario

The response of structural members under fire condition is significantly influenced by fire intensity. Much of the current knowledge on fire resistance of structural members in buildings is based on standard fires, where the exposure is assumed to be that of building (cellulose) fire in which the temperature of the fire assumed to be increased without any decay phase. However, in reality, fire exposure depends on fuel load and ventilation characteristics (Buchanan, 2002). The main source of fire in buildings is cellulose based combustible materials and these fires die down at some point and enters a decay phase due to limitation on available fuel and/or ventilation. The temperature-time curve for ASTM E119 and ISO 834 standard fires are given by:

\[ T_{ASTM\ E119}({}^\circ\text{C}) = 750\left(1 - e^{-3.79553\sqrt{t}}\right) + 170.41\sqrt{t} + T_0 \qquad \text{[2.3]} \]

\[ T_{ISO\ 834}({}^\circ\text{C}) = 345\log_{10}(8t + 1) + T_0 \qquad \text{[2.4]} \]

where; \( t \) is the time (hours) in Eq. [2.3] and (minutes) in Eq. [2.4], while \( T_0 \) is the ambient temperature (°C).

Fire in bridges is totally different than those in buildings since the fire source is generally from gasoline present in trucks or vehicles involved in collisions or combustible cargo in the trucks. Furthermore, bridge fires are generally in open air
conditions and have unlimited supply of oxygen. Therefore, fire in this case is much more intense and has higher heating rate as compared to building fires. Fires in bridges can be represented by hydrocarbon fire or external fire depending on the fuel control and intensity of the fire event. The temperature-time curve for hydrocarbon and external standard fires are given by:

\[ T_{Hydrocarbon\ fire} (\degree C) = 1080(1 - 0.325e^{-0.167t} - 0.675e^{-2.5t}) + T_0 \ldots \ldots \ldots [2.5] \]

\[ T_{External\ fire} (\degree C) = 660(1 - 0.687e^{-0.327t} - 0.313e^{-3.8t}) + T_0 \ldots \ldots \ldots [2.6] \]

where; \( t \) is the time (minutes) and \( T_0 \) is the ambient temperature (\( ^\circ \)C).

2.3.4 Thermal gradients

When a steel beam-concrete slab assembly exposed to fire, the bottom flange and web experience rapid raise in temperature as compared to top flange or concrete slab. This is due to high thermal conductivity and specific heat of steel as compared to concrete. Also, presence of the concrete slab that is in contact with the top surface of top flange, works on dissipating temperature from the top flange resulting in lower raise in temperature in the top flange as compared to web and bottom flange. The difference in temperatures between bottom flange and concrete slab leads to develop thermal gradients across the depth of section. Thermal gradients lead to develop thermal bending stresses in addition to the bending stresses from the applied loading. The extent of bending stresses resulted from thermal gradients depends on sectional geometry of the member, restraint situation, and fire scenario.

In case of steel bridge girders which have deeper sections and exposed to severe fires such as hydrocarbon fire, the extent of thermal gradients can be large and affect the
fire performance of steel bridge girders. The bending stresses resulting from thermal gradient are independent of the applied loading. In other words, even if the steel girder is not loaded, the girder still deflects due to the effect of thermal gradients. The deflection that develop from thermal gradients can dominate the flexural response of the steel girder especially during the early stage of fire exposure when the steel temperatures is below 400°C and steel still has full room temperature strength. With progression of fire, the effect of thermal gradients increases simultaneously with degradation of steel strength and stiffness. As a result, the flexural capacity of the steel girder degrades in a faster pace.

Due to uneven heat distribution in the section, strength and stiffness properties of steel vary across the depth of the section. This variation of strength and stiffness properties of steel leads to an eccentricity between the center of stiffness and center of geometry of the cross section. Because of this gradient-induced shift in the center of stiffness, the axial force in case of axially restraint members will act eccentrically on the section, and thus generates bending moment. This bending moment can cause a shift in the plastic P-M diagram. Therefore, this migration of center of stiffness causes a distortion in the plastic P-M interactive diagram. In a similar fashion, thermal gradient can also cause a shift in the shear center of the section, and this shift influences the lateral torsional buckling capacity of steel beams exposed to realistic fire exposures (Dwaikat and Kodur, 2010).

2.3.5 Failure criteria

According to prescriptive approach, defining failure of a beam or girder under fire conditions is based on thermal and/or strength failure criteria as specified in ASTM
Accordingly, the thermal failure of a steel member occurs when at least one of the following criteria is attained:

- The average temperature in steel exceeds the critical temperature, which is 538°C (1000°F) or when the temperature at any steel plates (flanges and web) exceeds 649°C (1200°F).

- The beam is unable to resist the applied service load (typically 50% of ambient temperature capacity).

In addition to strength limit state, deflection can govern the failure of beams under fire condition. In many scenarios, strength failure in a steel member exposed to fire is reached after undergoing large deflections. This is due to deterioration of member stiffness and also due to temperature-induced creep. Also, the integrity of the structural member cannot be maintained under excessive deformations. Furthermore, evaluating fire resistance based on limiting deflection will help to facilitate the safety of fire fighters and also to safely evacuate occupants prior to structural collapse as per strength limit state. The deflection limit states are specified in BS 476 and failure is said to occur when one of the following criteria is attained:

- The maximum deflection of the beam exceeds L/20 (or L/30 in some cases) at any fire exposure time, or

- The rate of deflection exceeds the limit given by the following expression:

\[
\frac{d\Delta}{dt} = \frac{L^2}{9000d} \text{ (mm/min)}
\]

where; \( L \) is the span length of the beam (mm), and \( d \) is the effective depth of the beam (mm).
The above deflection limit states to define failure are derived based on assuming flexural yielding as the dominant failure limit states in beams. This is not valid in case of steel girders with slender web, in which the web shear buckling might be the most dominate limit state. Therefore, tracing the web buckling response (web out-of plane displacement) near steel girder supports are to be considered together with the above deflection criteria to define failure in steel bridge girders.

The above factors clearly indicate that the fire response of structural members in bridges can be significantly different than those in buildings. Therefore, there is a need to develop test data and research information specific to structural members in bridges under fire conditions.

2.4 Previous Studies

A review of the literature indicates that most of the reported fire experiments and numerical analysis are on hot rolled steel beams typical of building applications, while, there is lack of experimental and numerical studies on the fire response of steel bridge girders. However, the data available on beam-slab members in buildings under fire conditions can be utilized to gauge the fire response of steel bridge girders as discussed previously. Therefore, experimental studies on fire performance of steel beam-concrete slab assemblies are also overviewed in this section. This includes studies that were undertaken to evaluate critical factors that could be correlated to the response of steel bridge girders under fire condition.

2.4.1 Experimental studies

There is limited number of experimental studies on steel beam-concrete assemblies reported in the literature. Based on these experiments, attempts have been
made to study the critical factor that influence the fire performance of steel beam-concrete slab assemblies, namely end restraint, thermal gradients, load level, and fire protection. These factors can provide some guidance on the behavior of steel bridge girders under fire conditions. The following is a summary of reported relevant fire tests on beam-slab assemblies.

Wainman and Kirby, (1987) carried out fire resistance tests on steel beam-concrete slab assemblies (4.5 m), typical of that used in buildings, by exposing them to standard ISO 834 fire exposure. The experimental study consisted of testing 21 non composite beams (without slab interaction) and 2 fully composite steel-slab assemblies. The non composite beams comprised of steel beams (254x146x43 UB, 305x165x46 UB, 356x171x67 UB, and 406x178x60 UB) and non-structural concrete slab, that was cast onto the upper flange of the test beams. The fully composite beams comprised of steel beams (254x146x43 UB) and normal weight concrete slab of 650 mm width and 130 mm thick. To provide full composite interaction between the steel beam and the concrete slab, 32 shear studs (75 mm length and 19 mm diameter) were welded along the top flange in two rows. The layout of the test composite beam-slab assembly is shown in Figure 2.4.

Beam-slab assembly were tested under fire and loading conditions. The loading was applied through four point loads long the beam span. The steel sections were of Grades 43A and 50B/50D (equivalent to A36 and A992 steel respectively) and were not provided with any fire protection. Deflection limit state (L/30) and rate of deflection were used as a failure criteria to define failure. Experimental results indicated that the fire resistance of beam-slab assemblies is influenced by load level, grade of steel, and composite action. The fire resistance of bare non-composite beam-slab assemblies under
55% loading and standard ISO fire exposure can be as low as 25 minutes. Furthermore, considering the composite action arising from steel beam-concrete slab interaction enhances the fire resistance up to 35 minutes under same loading and fire conditions.

Bletzacker, (1966) undertook an experimental study to trace the response of fire exposed steel beam-concrete slab assemblies. This study has been carried out at Ohio State University for the American Iron and Steel Institute (AISI). The experimental program consisted of twelve steel beam-concrete slab assemblies (5.1 m span) consisting of 914 mm wide and 100 mm thick structural concrete slab on 22-gauge steel floor deck supported by a W12×27 steel beam. The beam and floor deck were protected with a spray-applied cementitious fire protection material. The steel beam-concrete slab assemblies were tested under fire (ASTM E119) and loading conditions. The parameters considered in the test program included connection type, composite action between beam and concrete slab (non-composite, partially composite, and fully composite), axial and rotational restraints.

Results from this study showed that any increase of the stiffnesses of end restraints beyond a certain limit does not improve fire resistance of steel beams. In the tests, as the magnitude of restraining stiffness was increased the flanges and/or the web of steel beams became more susceptible to local buckling. This was attributed to the fact that the larger the restraining stiffness, the larger is the fire induced axial force in the beam. Also, test results showed that composite action can enhance fire resistance of steel beams by providing additional restraint from slab effect (especially rotational restraint near the supports) in conjunction with the restraint provided by the beam-to-column connections. It was also found that the higher the load level the lower is the fire
Fike and Kodur, (2011) carried out fire resistance experiment on a composite steel beam-concrete slab assembly. The beam-slab assembly comprised of primary and secondary beams supporting a steel fiber reinforced concrete slab (SFRC) on a steel deck. Three W10x15 steel beams were connected to a pair of W12x16 girders with a composite deck (slab) incorporating shear studs. The two W12x16 steel girders were provided with external fire protection of 22 mm thickness to achieve a two-hour fire resistance rating, while the W10x15 secondary beams were left unprotected. The floor assembly was designed and fabricated based on AISC specifications. The designed floor assembly was exposed to ASTM E119 fire. The layout of the test floor assembly is shown in Figure 2.5. This study aimed to enhance the fire resistance of the composite assembly by using steel fiber reinforced concrete.

Results from this work showed that the combined effect of composite construction, tensile membrane action, and the improved properties of SFRC under realistic fire, loading, and restraint conditions can provide sufficient fire resistance (1 to 2 hours) in steel beam-concrete deck slabs without the need for external fire protection to the secondary beams and steel deck of the slab.

In the last four decades, extensive experimental work has been conducted on steel plate bridge girders at ambient temperature. These studies focused on web buckling, composite action, post-buckling shear capacity, and shear failure mechanism in steel bridge girders (Alexander et al., 2009, Shanmugam and Baskar, 2003, Alinia et al., 2009). A detailed literature review reveals that there is a lack of experimental data on the behavior of steel bridge girders exposed to fire. The only experimental study on the fire
response of steel plate girders is from the work reported by Vimon satit et al., (2007).

Vimon satit et al., (2007) tested numbers of small-scale steel girders (without concrete slab) of 1.66 m span. Theses girders were grouped in to five series (TG1, TG2, TG3, TG4 and TG5), three of which were fabricated (built-up) section (TG3, TG4 and TG5), while the rest were hot-rolled (UC 152x152x23 kg/m, and 203x203x52 kg/m) sections typical to that in buildings. The configuration of test specimens is illustrated in Figure 2.6. The plate girders were of depth of 305mm and web slenderness 112, 152, and 203. The web slenderness was varied by changing thickness of the web from 1.5 to 2.7 mm, while the depth of the section was kept constant. The test specimens were loaded predominantly in shear under a steady-state fire exposure. The tested girders, as well as some of the conditions used in fire tests, do not represent realistic conditions encountered in practice. One specimen from each group was tested at ambient conditions.

The aim of this study was to investigate shear buckling behavior, diagonal web tensile field action, and development of plastic hinge mechanism in flange plates of girder panels. Therefore, failure in bending mode was prevented by stiffening top and bottom flanges. These tests were carried out under isothermal (steady-state) condition were the temperatures maintained constant at 400°C, 550°C, and 700°C during the test (no fire curve). Results from this work indicated that shear capacity of steel girder decreases significantly with increasing temperature. Furthermore, at high-temperatures, shear buckling failure behavior became less apparent. The authors recommended further detailed experiments to trace the fire response of steel girders.

2.4.2 Numerical studies

A review of literature indicates that a limited number of numerical studies have
been carried out on the fire behavior of steel bridge girders. These studies reported results from finite element analysis, and in the analysis a number of assumptions were made to reduce the complexity of the problem. Most of the reported studies on bridges under fire are assessment analysis of fire exposed bridges in field (Mendes et al., 2000, Roche, 2001, Dotreppe et al., 2006, Alnahhal et al., 2006, Nigro et al., 2006, Eisel et al., 2007, Choi, 2008, Bennetts and Moinuddin, 2009). Following is a summary of some of the prominent studies.

Mendes et al., (2000) undertook a 2D bridge deck analysis to simulate an actual ship fire accident in the “Vasco da Gama Bridge” in Lisbon, Portugal, which damaged the concrete decks of the bridge in a real fire. The cross sectional temperatures resulting from fire exposure and time to collapse (fire resistance) were studied. Three fire scenarios defined by the geometric characteristics, fuel type, and burning rate were used in the analysis. Results from analysis indicated that, under fire exposure, the anchorage of the bridge is susceptible to serious damage in 20 to 30 minutes, which leads to progressive failure of the overall bridge.

Dotreppe et al., (2006) carried out a finite element analysis to simulate the collapse of “Vivegnis Bridge” in Belgium resulting from a real fire accident on August 14, 1985. This was a tied-arch steel bridge of 136 m span. The fire occurred at the base of the bridge footing due to explosion of a gas pipe. The finite element analysis was carried out using SAFIR computer program. A 3-D beam element was used to model the main girders, cross girders, concrete slab, and arches while truss elements were used for the bracing and the suspenders. The model accounted for the effects of geometric nonlinearity (large deformation) and material nonlinearity. The analysis was carried out
under hydrocarbon fire exposure. Predictions from the finite element analysis compared well with field observations in term of failure mode and time to failure of the bridge.

Bennetts and Moinuddin, (2009) carried out a study to illustrate the vulnerability of bridges under fire conditions. The analysis was performed on a cable stayed bridge and the effect of three different fire scenarios on the cables and the tower was studied. The fire scenarios were assumed to occur on the bridge. Protected and unprotected cables were considered in the analysis. Furthermore, steel critical temperature was used as a failure limit state to define failure under fire conditions. Time to failure of the cables and the tower were presented for different load levels as ratio to the ultimate strength. Results from analysis show that the time to failure in unprotected cable stayed bridge can be less than 10 minutes under sever fire exposure such as hydrocarbon fire.

Kodur et al., (2010) carried out a case study on a 49 m span high overpass bridge to illustrate the fire performance of steel-bridge girders. Both unprotected and protected steel girders were analyzed using finite element based SAFIR computer program. Realistic load conditions, fire scenarios and high-temperature material properties were considered in the analysis. Results indicate that an unprotected steel girder develops a plastic hinge, leading to significant deflection and collapse in less than 30 minutes. However, protecting the girder with any fire insulation enhances the fire resistance significantly.

Payá-Zafortza and Garlock, (2010) studied the fire performance of a simply supported bridge girder. The cross section of the bridge selected for analysis comprised of five steel girders supporting a reinforced concrete slab, which was not structurally connected to the steel girders. A 3-D simulation of the steel girder was modeled in
computer program LUSAS using solid elements. The fire response of the steel girder was studied considering different loading conditions (dead and live) and axial restraint (fixed or free support) conditions under hydrocarbon fire exposure. Results from this study showed that unprotected steel girder can collapse in less than 10 minutes in to a severe fire such as hydrocarbon burning. Also, the horizontal displacements exceed the joint spacing which requires the consideration of the interaction (restraint) between the deck slab and the adjacent span or abutment.

Kodur al el., (2012) developed a 3-D finite element model in ANSYS to evaluate the fire response of steel beam-concrete assembly. The finite element model was validated by comparing results obtained from the model with experimental data from fire tests. In this study, critical factors that influence fire performance of beam-slab assembly namely, composite action, load level, fire scenarios and connection type were investigated. Results from the numerical studies indicated that the proposed model is capable of predicting the fire response of beam-slab assemblies with a good accuracy. Also, the authors concluded that the composite action arising from steel beam-concrete slab interaction significantly enhances the fire resistance of the composite beam-slab assembly.

2.5 High-Temperature Steel Properties

Material properties of steel play a crucial role in determining the fire resistance of the steel structure members. In most previous numerical studies, researchers have either used material models specified in codes and standards, such as the Eurocode or ASCE manual of practice (EC3 2005, ASCE 1992), or they developed their own specific material models (Poh, 2001, Anderberg, 1988, and William-Leir 1983) for evaluating fire
resistance of steel structures. However, there is a considerable variation in different constitutive relationships presented in different codes and standards for many of the high-temperature properties of steel (Kodur and Harmathy, 2002).

The main reason for the variation in the material properties for steel is that till recently there were no standard methods to conduct material property tests at evaluated temperature. This has led researchers to use their own test methods to measure the high-temperature properties of steel. The differences in test methods, such as heating (rate) conditions, steady or transient conditions, and data collection techniques, lead to an appreciable variation in the different sets of data available in the literature.

Steel is well known to have excellent strength properties at ambient temperature; however, it loses its strength and stiffness with rise of temperature. The temperature dependent properties that are essential for modeling the response of steel structures under fire conditions includes thermal, mechanical, and deformation properties and these properties vary with temperature and dependent on the type and grade of the steel. Much of the current knowledge on the high-temperature properties of steel is based on reported material property tests on carbon steel. Furthermore, the information on high-temperature material properties of structural steel is suitable for the heating phase of fire only. This is because most of these material tests were conducted under either transient or steady state tests with increasing temperature. However, there is a lack of data on material properties of steel during and after the cooling phase of fire.

Thermal properties of steel is important to determine the temperature distribution in the steel sections resulting from fire exposure, while the mechanical properties control the loss of strength and modulus as a function of temperature. Deformation and mechanical
properties determine the extent of deformation of the steel member under fire conditions (Kodur et al., 2009b).

Steel used in construction applications are classified into different categories based on chemical composition, tensile properties, and fabrication process as carbon steel, high-strength low alloy steels (HSLA), heat-treated carbon steels, and heat-treated constructional alloy steels (Brockenbrough and Merritt, 2005). High-strength low-alloy steels provide better performance in terms of strength, weldability and corrosion/weather resistance. One commonly available HSLA is ASTM 572 Gr. 50 and this steel is widely used in bridge and building applications. Most of the high-temperature material properties data available in the literature are for steels used in building such as ASTM A36 and ASTM A992 while there is lack of information about specific high-temperature material properties of steels used in bridge applications. Also, there is lack of data of residual material properties of steel during the cooling phase or after the decay phase of fire.

2.5.1 Thermal properties

The main thermal properties that influence the temperature rise in steel are thermal conductivity and specific heat (often expressed in terms of heat capacity). Figure 2.7 plot the available data on thermal conductivity, and specific heat of steel as a function of temperature, respectively (Kodur et al., 2009b). Relationships from codes and standards (EC3 2005, ASCE 1992), as well as published test data were used to compile Figure 2.7 (Rempe and Knudson, 2008, Dale et al., 2007, Touloukian, 1972, Powel and Tye, 1960, and Yawata, 1969).

It can be seen in Figure 2.7(a) that thermal conductivity decreases with
temperature in an almost linear fashion, and there is little variation between the different models presented in ASCE manual and Eurocode. On the contrary, specific heat models vary considerably between 700°C and 800°C, as can be seen in Figure 2.7(b). In general, the specific heat of steel increases with an increase in temperature with a large spike occurring around 750°C. The spike in the specific heat at around 750°C is due to the phase change that occurs in steel. In overall assessment, for high-temperature thermal properties of steel, minor variations exist in the specified models in design codes and standards.

2.5.2 Mechanical properties

Tests for high-temperature strength properties can be carried out mainly in two ways: transient- and steady-state tests. In transient-state tests, the test specimen is subjected to a constant load and then exposed to uniformly increasing temperature. Temperature and strain are continuously recorded under constant stress. Thermal strain (evaluated from a separate test) is then subtracted from the total measured strain (Outinen, 2007). In the transient-state tests, the heating rate has a great influence on the strain rate and thus different heating rates produce different strain rates. The heating rate of steel depends on the nature of the fire as well as on the thermal protection (insulation) and geometry of the cross section. Generally, for a typical beam with 2-hour fire rated protection, the heating rate of steel can be in the range between 3-7°C/min. However, for unprotected steel sections, the heating rate can be in the range between 25-40°C/min. In the literature, (Outinen, 2007) transient mechanical property tests were conducted at heating rates ranging between 10 to 50°C/min. This heating rate can be suitable for unprotected steel members, but not for protected members with slow heating rates.
Steady-state tests are generally faster and easier to conduct than the transient-state tests. In steady-state tests, the test specimen is heated to a specific temperature and after that a tensile test is carried out. Stress and strain values are recorded continuously under constant temperature. The test can be either load-controlled (loading rate is constant) or strain-controlled (strain rate is constant) (Outinen, 2007 and Anderberg, 1988). Despite the fact that strain rate has a significant effect on the test results, a large amount of test data on conventional steel is published without the information on strain rates. Therefore, test standards are still concerned with defining limits for strain rates in tests (Outinen, 2007, Anderberg, 1988, and Cooke, 1988). The yield strength and modulus of elasticity of steel from different test programs are illustrated in Figure 2.8.

2.5.3 Deformation properties

2.5.3.1 Thermal strain

The deformation properties that influence the fire response of steel structures are thermal strain and high-temperature creep. There have been many tests to characterize thermal strain of steel at elevated temperatures; results from some of which are compiled in Figure 2.9. Variation of thermal strain models as specified in ASCE and EC3 are also plotted in Figure 2.9. Differences exist between the Eurocode and ASCE models for thermal strain of steel above 300°C. In the temperature range of 700°C-850°C the ASCE model assumes a continuously increasing thermal strain while the Eurocode model accounts for the phase change that occurs in steel in this temperature range by assuming a constant thermal strain from 750°C to 850°C, followed by an increasing thermal strain up to 1000°C.
2.5.3.2 High-temperature creep

Steel structures when exposed to elevated temperatures, undergo permanent deformation (plastic) even when applied stress level is below that of yield stress and this time-dependent deformation is referred to as creep. Thus, creep can be defined as an increase in strain in a solid material under constant stress over a period of time. There are two broad mechanisms by which creep takes place in crystalline materials (metals) namely; dislocation creep, and diffusional creep. Dislocation creep occurs due to movement of material dislocations, while diffusional creep is primarily related to transport of material by diffusion of atoms within a grain and could be in two forms namely; grain boundary diffusion and bulk crystal diffusion. The rate of creep strain progression in both these mechanisms depends on the extent of diffusion that occurs in material atoms (Ashby and Jones, 2005).

Creep deformation occurs due to movement of dislocations in the slip plane. Naturally, metal's (steel) composition contains variety of defects (e.g. solute atoms), that act as obstacles to dislocation motion. At room temperature, creep strain occurs at very slow pace since the amount and distribution of these defects remain almost uniform. At high-temperatures, vacancies in the crystalline structure of the material can diffuse into dislocation. This causes the dislocation to move faster to an adjacent slip plane. Therefore, creep deformations in steel accelerate with increase in temperature (Kodur and Dwaikat, 2010). The temperatures at which creep deformations accelerate differ in various materials depending on melting temperature and composition of these materials. In general, creep can be significant at temperatures above 40% of the melting temperature of the material (0.4 $T_m$) (Morovat et al., 2012).

Typically, under a constant temperature and stress level, creep behaviour can be
grouped into three stages namely; primary (initial) creep, secondary (steady state) creep, and tertiary (accelerating) creep (as shown in Figure 2.10). In Stage I, creep strain starts at a relatively high rate but decreases eventually depending on stress level and temperature. Following that, in Stage II, the strain rate remains almost constant with time. Finally in the last stage (Stage III), creep strain increases at a rapid pace with increase in stress level and this results from necking phenomenon (Ashby and Jones, 2005). However, with increasing temperature and stress levels, rate of creep can become very high and this can lead to significant creep deformations as shown in Figure 2.10. Furthermore, with increasing temperature and stress level, it becomes very difficult to distinguish between secondary and tertiary creep stages.

Creep deformations at room temperature are generally small, occur over long period of time, and are often ignored especially for high strength steels subjected to stress levels below yield stress (Cheng et al., 2000). However, under the severe bridge fire condition (high-temperatures), creep deformations can become predominant within a short duration of fire exposure and can influence the failure mode and fire resistance of steel structures. However, the extent of high-temperature creep in steel structures primarily depends on properties of steel (grade and type of steel), exposure temperature, stress level on the member, and time of exposure.

Despite that high-temperature creep leads to larger deformations in steel members under fire (Kodur et al. 2010), codes of practice do not provide sufficient guidance on how to account for high-temperature creep. Eurocode states that "the effects of transient thermal creep need not be given explicit consideration" (Clause 4.3.3(4) of EC3 2005), which indicates that high-temperature creep is partly accounted for in the stress-strain
curves specified in EC3. However, the ASCE manual of practice states that high-temperature creep should be accounted for in fire resistance analysis through one of two options. The first option is to use temperature-stress-strain curves derived from transient-state tests at relevant heating and strain rates (ASCE, 1992 and Buchanan, 2002), while the second option is to use high-temperature creep models developed for structural steel. As such, it is generally left to the user to choose which creep model to use.

Creep tests can be carried in two ways, namely steady-state and transient tests. In steady-state tests, both temperature and stress are kept constant, while strain is continuously recorded during the test. In transient tests, the test specimen is subjected to a constant stress and then exposed to uniformly increasing temperature. Temperature and strain are continuously recorded under constant stress. Thermal strain (evaluated from a separate test) is then subtracted from the total measured strain in both methods. Steady-state method is commonly used in literature because it is easier to conduct, however transient method is more realistic. In general, conducting creep tests at high-temperature is very complex and time consuming. Therefore, there is lack of creep tests data for steel at high-temperatures.

Brinc et al., (2009) carried out a set of creep test on AISI 316Ti (stainless steel 1.4571) steel at various stress and temperature range. The temperature ranged from 400-700°C, while applied stress ranged from 25%-90% of the yield stress of the steel at room temperature. Prior to creep tests, the tensile strength, elastic modulus, and fracture toughness for AISI 316Ti steel were evaluated under freezing and at elevated temperatures (-70-700°C). Results from this study showed that AISI 316Ti steel can withstand high-temperature exposure if stress levels are sufficiently low. Also, creep
strains are not significant at temperature of 600°C when stress levels are low.

Brinc et al., (2011) investigated the high-temperature creep behavior of high-strength low-alloy ASTM A618 steel. The creep test were carried out at three target temperatures (400°C, 500°C, and 600°C) and various stress level. The stress level were varied from 30%-77% of the yield strength of the steel at room temperature. Also, the high-temperature stress-strain relations were evaluated in this study. Experimental results showed that creep strains are not significant at temperatures of 400°C and 500°C, when the stress levels are less than 50% and 40% of the room temperature yield stress respectively. The authors concluded that ASTM A618 steel can be used in structure applications operating in similar environmental conditions.

Morovat et al., (2012) carried out a comprehensive experimental study to evaluate the high-temperature creep behavior of ASTM A992 steel. Number of specimens was cut from web and flange plates of W4x13 and W30x99 wide flange sections. The creep tests were carried out at temperature range 400-700°C and stress level 50%-90% of the room temperature yield stress. Results from creep tests were compared with Harmathy creep model. Experimental results from this study showed that flange can experience higher creep strains as compared to web. Also, specimens from W4x13 showed less creep resistance as compared to that of W30x99. The authors recommend the need for more reliable creep models for structural steel at elevated temperature.

A review of literature indicates that very few experiments have been carried out on high-temperature creep of steel. The limited creep tests reported in literature were carried out on ASTM A618, ASTM A992, and ASTM A316Ti steels, while there is no creep data specific to ASTM A572 steel. ASTM A572 is one of commonly available
high-strength low-alloy steel and this steel is widely used in bridge and building applications because it provides better performance in term of strength, weldability and corrosion/weather resistance. Hence creep data on ASTM A572 steel is crucial for modeling fire response of steel bridge girders.
Table 2.1: Fires resulting from bridge collisions as per to FARS statistics

<table>
<thead>
<tr>
<th>Period</th>
<th>Bridge collisions</th>
<th>No fire occurrences</th>
<th>Fire occurrences</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 years</td>
<td>5,209</td>
<td>4,672</td>
<td>536</td>
</tr>
<tr>
<td>Average per year</td>
<td>347</td>
<td>312</td>
<td>36</td>
</tr>
</tbody>
</table>

Table 2.2: Bridge collapses due to fire as per to NYDOT survey

<table>
<thead>
<tr>
<th>Collapse</th>
<th>Total</th>
<th>Timber bridges</th>
<th>Steel/concrete bridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Partial collapse</td>
<td>12</td>
<td>2</td>
<td>10</td>
</tr>
<tr>
<td>Total collapse</td>
<td>23</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>Not available</td>
<td>17</td>
<td>15</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 2.3: Some of the major fire accidents on bridges in US during the last 15 years

<table>
<thead>
<tr>
<th>Bridge/location</th>
<th>Date of fire incident</th>
<th>Cause of fire</th>
<th>Material type used in structural members</th>
<th>Damage description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-375 bridge over I-75 in Detroit, MI</td>
<td>May 24, 2015</td>
<td>A gasoline tanker carrying 9000 gallons crashed over the bridge and caught into fire</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
<td>Concrete deck was damaged significantly by the fire. Also, the steel girders experienced some damage</td>
</tr>
<tr>
<td>Bridge over freeway 60, Los Angeles, CA</td>
<td>December 14, 2011</td>
<td>A tanker truck carrying 128 m$^3$ of gasoline caught fire, and burned out underneath the bridge</td>
<td>Concrete deck (precast prestressed I girders + cast in place reinforced concrete slab)</td>
<td>Concrete girders were damaged significantly by the fire. The bridge was demolished and replaced</td>
</tr>
<tr>
<td>Metro-North railroad Bridge over Harlem River, NY</td>
<td>September 20, 2010</td>
<td>Explosion in power transformer caused burning of wood pilings under the bridge</td>
<td>Steel truss bridge</td>
<td>Minor structural damage of truss members</td>
</tr>
<tr>
<td>Bridge over I-75 near Hazel Park, MI</td>
<td>July 15, 2009</td>
<td>A gasoline tanker struck an overpass on I-75.</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
<td>Complete collapse of the bridge to the freeway below</td>
</tr>
<tr>
<td>Big Four Bridge, Louisville, KY</td>
<td>May 7, 2008</td>
<td>Electrical problem of the lighting system</td>
<td>Steel truss bridge</td>
<td>Minor structural damage resulting in large amount of debris on the bridge</td>
</tr>
</tbody>
</table>
Table 2.3 (cont’d)

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>Event Description</th>
<th>Damage Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tappan Zee Bridge, over Hudson River, NY</td>
<td>July 2, 2007</td>
<td>A car struck a tractor-trailer and caught on fire near the bridge</td>
<td>Steel truss, cantilever type bridge</td>
</tr>
<tr>
<td>Stop Thirty Road, State Route 386 Nashville, TN</td>
<td>June 20, 2007</td>
<td>A fuel tanker truck rear-ended a loaded dump truck. The tanker erupted into flames beneath the bridge</td>
<td>Concrete hollow box-beam bridge</td>
</tr>
<tr>
<td>Bill Williams River Bridge, AZ</td>
<td>June 20, 2007</td>
<td>A gasoline tanker overturned</td>
<td>Concrete deck (precast prestressed I girders + cast in place reinforced concrete slab)</td>
</tr>
<tr>
<td>Belle Isle Bridge in NW Expressway, Oklahoma City, OK</td>
<td>January 28, 2006</td>
<td>A truck crashed into the bridge</td>
<td>Concrete deck (precast prestressed I girders + cast in place reinforced concrete slab)</td>
</tr>
<tr>
<td>Bridge over the Norwalk River near Ridgefield, CT</td>
<td>July 12, 2005</td>
<td>A tanker truck carrying 30.3 m³ of gasoline overturned, caught fire, and burned out on the bridge</td>
<td>Concrete deck (precast prestressed box girders + cast in place reinforced concrete slab)</td>
</tr>
<tr>
<td>I-80/880 interchange in Oakland, CA</td>
<td>April 29, 2007</td>
<td>A gasoline tanker crashed on underneath the bridge</td>
<td>Composite deck (steel girders + reinforced concrete slab) supported by reinforced concrete columns</td>
</tr>
<tr>
<td>I-95 Howard Avenue Overpass in Bridgeport, CT</td>
<td>March 26, 2003</td>
<td>A car struck a truck carrying 8,000 gallons of heating oil near the bridge</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
</tr>
<tr>
<td>I-20/I-59/I-65 interchange in Birmingham, AL</td>
<td>January 5, 2002</td>
<td>A loaded gasoline tanker crashed</td>
<td>Main span of girders sagged about 3 meters (10 feet)</td>
</tr>
</tbody>
</table>

The bridge sustained very little damage and traffic was reopened after minor repairs.
Concrete girders were damaged by the fire and subsequently repaired, but it was not necessary to replace any of the girders.
Concrete girders were slightly damaged by the fire. The safety of the bridge was assessed and the bridge was reopened to traffic.
The deck was replaced by a new one but its beams were tested by the FHWA.
Two spans of I-580 bridge girders collapsed.
Collapse of the girders of southbound lanes and partial collapse of the northbound lanes.
Table 2.3 (cont’d)

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>Event Description</th>
<th>Damage Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-80W/I-580E ramp in Emeryville, CA</td>
<td>February 5, 1995</td>
<td>A gasoline tanker crashed</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Deck, guard rail and some ancillary facilities were damaged</td>
</tr>
<tr>
<td>I-10 Bayway in Mobile, AL</td>
<td>September, 02, 2009</td>
<td>A truck crashed on the bridge and caught into fire</td>
<td>Concrete girders</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Minor damage of the concrete deck</td>
</tr>
<tr>
<td>Williams River bridge in Bill Williams, AZ</td>
<td>July, 28, 2006</td>
<td>A tanker carrying 7600 gallons diesel crashed on the bridge and the fuel spilled underneath the bridge through the expansion joints</td>
<td>Concrete deck (precast prestressed I girders + cast in place reinforced concrete slab)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Three concrete girders were damaged</td>
</tr>
<tr>
<td>I-10 Escambia Bay, FL</td>
<td>June, 4, 2009</td>
<td>A truck crashed on the bridge and caught into fire</td>
<td>Concrete girders</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Minor damage of the concrete deck</td>
</tr>
<tr>
<td>I-75 Big Slough Canal, FL</td>
<td>February, 03, 2004</td>
<td>A truck crashed underneath the bridge and caught into fire</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Miner structural damage</td>
</tr>
<tr>
<td>I-285, Atlanta, GA</td>
<td>June, 20, 2001</td>
<td>A gasoline tanker carrying 7000 gallons overturned underneath the bridge and caught into fire</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Substructure damaged</td>
</tr>
<tr>
<td>Long fellow Bridge, Boston, MA</td>
<td>May, 02, 2007</td>
<td>Debris under bridge</td>
<td>Steel arch</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>No damage was reported</td>
</tr>
<tr>
<td>I-95, Baltimore, MD</td>
<td>January, 13, 2004</td>
<td>A truck crashed on the bridge and caught into fire</td>
<td>Concrete girders</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Minor damage</td>
</tr>
<tr>
<td>I-80, Denville, NJ</td>
<td>June, 24, 2001</td>
<td>A truck crashed underneath the bridge and caught into fire</td>
<td>Concrete girders</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bridge girders were severely damaged and the bridge was replaced</td>
</tr>
<tr>
<td>Throgs Neck Bridge, NYC, NY</td>
<td>July, 21, 2009</td>
<td>Construction fire under the bridge</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Some steel girders were damaged</td>
</tr>
<tr>
<td>Manhattan Bridge, NYC, NY</td>
<td>July, 8, 2009</td>
<td>A truck crashed on the bridge and caught into fire</td>
<td>Suspension</td>
</tr>
<tr>
<td>--------------------------</td>
<td>-------------</td>
<td>-------------------------------------------------</td>
<td>------------</td>
</tr>
<tr>
<td>I-95 Chester Creek, PA</td>
<td>May, 24, 1998</td>
<td>A gasoline carrying 8000 gallons crashed underneath the bridge and caught into fire</td>
<td>Composite deck (steel girders + reinforced concrete slab)</td>
</tr>
<tr>
<td>37 Expressway near I-95, RI</td>
<td>July, 20, 2000</td>
<td>A gasoline carrying 1000 gallons overturned on the bridge and caught into fire</td>
<td>Concrete girders</td>
</tr>
<tr>
<td>Puyallup River Bridge, Olympic, WA</td>
<td>December, 11, 2002</td>
<td>A tanker carrying 30000 gallons of Methanol crashed underneath the bridge and caught into fire</td>
<td>Concrete deck (precast prestressed I girders + cast in place reinforced concrete slab)</td>
</tr>
</tbody>
</table>
Figure 2.1: Bridge collapses according to NYDOT survey

Figure 2.2: Bridge collapse casualties per 100 events according to NYDOT survey
(a) Hazel Park overpass collapse in Michigan, 2009

(b) Oakland highway bridge collapse in California, 2007

Figure 2.3: Illustration of fire induced girder collapse in bridges
Figure 2.4: Configuration of fully composite beam-slab assembly tested by Wainman and Kirby (1987)

Figure 2.5: Layout of floor assembly tested by Fike and Kodur (2011)
Figure 2.6: Tested steel plate girders by Vimonsatit et al., (2007)
Figure 2.7: Thermal properties of steel as measured in different test programs and models
Figure 2.8: Mechanical properties of steel as measured in different test programs and models
Figure 2.9: Thermal strain of steel as measured in different test programs and models

Figure 2.10: Classical creep response of steel
2.6 Knowledge Gaps

The above state-of-the art review clearly indicate that the fires in bridges can be a significant problem and typically these fires result from crashing of vehicles in the vicinity of the bridge. Such fires can produce adverse conditions for structural members, especially for steel girders. The time to fire induced failure in a steel bridge girder can be as little as 30 minutes and thus very little time is often available for fire fighters to respond. Therefore, fire problem in bridges requires specific attention. The following are some of the key areas where further research is needed:

- There is lack of data on the behavior of bridge girders and the governing failure limit states of under fire conditions. Further, there is lack of material property data for modeling the response of steel bridge girders under fire conditions.
- There are no fire resistance experiments on the response of steel bridge girders under realistic fire condition and combined shear and flexural loading. Data from fire experiments are needed to validate finite element models.
- Limited numbers of numerical models are available in the literature for tracing the response of steel bridge girders under fire conditions. Most of these models do not account for composite action between steel girder and concrete slab. Also, there are limited numbers of numerical studies on the effect of critical factors influencing the fire response of steel bridge girders.
- There is lack of high-temperature properties of steels used in bridge application. Most of the data available in the literature are on structural steel commonly used in buildings.
• There is lack of residual strength analysis based on realistic residual property of steel to evaluate residual capacity of fire exposed steel bridge girders. Such studies are essential to develop an approach to evaluate residual capacity of steel bridge girders after fire exposure.

• To date, there are no fire resistance requirements for steel structural members in bridges. Also, there are no strategies to enhance the fire performance of steel bridge girders are in the literature.
CHAPTER THREE

3. FIRE RESISTANCE EXPERIMENTS

3.1 General

The state-of-the-art review presented in Chapter 2 clearly indicates that there is a lack of understanding on the behavior of steel bridge girders exposed to fire. To develop such an understanding, experimental studies on the fire performance of steel bridge girders were carried out. The test program consisted of fire resistance experiments on three steel bridge girders (Aziz et al., 2015). The main variable in these test specimens included load level, web slenderness and spacing of stiffeners. The girders were tested to failure by subjecting them to combined structural loading and fire exposure. The main objective of these tests is to trace the response of steel girders under fire conditions and to generate test data for validation of finite element models. Full details of the fire resistance experiments, specimen details, instrumentation, test procedure and measured response parameters are presented in this chapter.

3.2 Experimental Details

3.2.1 Design of steel girders

Three steel girders were designed according to AASHTO specifications (AASHTO, 2012). All of the girders were designed to simulate steel girder-concrete slab composite assembly and the design was carried out to satisfy the following limit states:
• Compactness limit state (AASHTO, Article 6.10.6)
• Strength limit state (AASHTO, Article 6.10.7)
• Design of traverse and bearing stiffeners (AASHTO, Article 6.10.9)
• Design of shear studs (AASHTO, Article 6.10.10)
• Design of welding (AASHTO, Article 6.13.3)

Flow charts of Articles (6.10.6), (6.10.7) and (6.10.9) of AASHTO are illustrated in Figures 3.1, 3.2, and 3.3, while design procedures with full details for steel girder G3 is illustrated as a design sample in Appendix A.

3.2.2 Fabrication of specimens

The tested bridge girders, designated as G1, G2, and G3, comprised of a steel section (hot rolled or built-up) supporting a reinforced concrete slab. The first test girder (G1) was a hot rolled section of W24x62 (taken from AISC, 2011), while the other two test girders (G2 and G3) were built-up plate girders. The web slenderness, defined as $D/t_w$ ratio (where $D$ is the web depth and $t_w$ is the web thickness), of girder G1 was 52, while in girders G2 and G3 it was 123. Girders G2 and G3 were stiffened with traverse stiffeners and the aspect ratio, defined as $a/D$ ratio (where $a$ is the stiffener spacing), in G2 was 1.0 and in G3 it was 1.5. All other dimensions for G2 and G3 were kept the same. Bearing stiffeners were provided at both the supports and at the location of the load point at mid-span. Longitudinal and traverse sections of girders G1, G2, and G3 are shown in Figure 3.4, while sectional dimensions are summarized in Table 3.1.

The steel girders were fabricated using A572 Grade 50 steel, which is a high strength, low-alloy steel commonly used in highway bridge applications. Test girders G2 and G3 were fabricated from series of plates, namely flange, web, intermediate and
bearing stiffeners. Initially, the web and flanges plates were assembled using E70XX electrode fillet welding. The weld dimensions were designed as per AASHTO specification as illustrated in Appendix A. Following that, the stiffeners were placed in the proper positions and welded to the girder. All of the steel girders were designed to achieve full composite action with the concrete slab. For this purpose two rows of 19 mm diameter shear studs were placed to ensure full composite action between the steel girder and the concrete slab, as shown in Figure 3.5(a).

The concrete slab is reinforced with a layer of tension steel rebars (as minimum reinforcement) at the bottom and a layer of wire mesh at the top as shown in Figure 3.5(b). The slab was cast with normal weight concrete supplied from a local concrete batch mix plant. The batch proportions of concrete are shown in Table 3.2. After placing the concrete, the concrete slab was covered with a vapor proof barrier and was cured with water for a week. After this, the slab was left to cure at ambient conditions for at least three months, before testing under fire conditions. The relative humidity of concrete was measured at different locations of the slab periodically during the three months of curing and on the day of testing.

3.2.3 Instrumentation

The steel girders were instrumented with thermocouples, strain gauges, and displacement transducers to monitor thermal and mechanical response during fire resistance tests. Cross sectional temperatures were measured using Type-K (0.91 mm thick) Chromel-alumel thermocouples placed at mid-span and quarter-span sections along the girder length. At each section, thermocouples were installed on the steel girders at the bottom flange, web, top flange, stiffeners, shear studs, and also at different depths of...
concrete slab to measure temperature progression during the fire test. High-temperature strain gages were attached to the flanges (top and bottom), shear studs, and the web of girders to directly monitor progression of strain at these locations. Thermocouple locations on steel girders G1, G2, and G3 are illustrated in Figures 3.6 and 3.7.

To measure mid-span deflection and axial displacement of the girders, as well as out-of-plane displacement of the web panel, vertically and horizontally oriented linear variable displacement transducers (LVDT’s) were attached at various distinct locations on each girder. The mid-span deflection was measured through two LVDTs that were attached to the top surface of the concrete slab beneath the loading actuator as shown in Figure 3.8(a). To measure out-of-plane displacement of the web, a well-insulated stiff threaded steel rod was attached to the center of the web panel and extended horizontally parallel to the concrete slab wing. The steel rod extends vertically to pass through the furnace lid for which an opening was made in the lid. The steel frame that carries the LVDT was installed on top surface of the concrete slab (outside the furnace zone). This is to ensure vertical movement of the steel frame (LVDT) during the deflection of the steel girder. A schematic of the set-up that is used to measure the web out-of-plane displacement is shown in Figure 3.8(c). The measured out-of-plane displacement of the web is for the web panel that is adjacent to the mid-span of the steel girder. To measure progression of axial displacement during the fire test, an additional LVDT was placed on the bottom flange location at one end of the steel girders as shown in Figure 3.8(b). In addition, furnace temperatures during the fire test were continuously monitored using six thermocouples distributed spatially inside the furnace.

Data through the above instrumentation network was recorded at five second
intervals through central data acquisition system. Visual observations were also made at five minute intervals to record significant changes (such as local buckling, spalling, etc.) throughout the duration of the test and also after the tests were terminated.

3.2.4 Test set-up

Fire resistance tests were carried out using a specially built fire testing furnace at the Civil Infrastructure Laboratory at Michigan State University. This furnace has been specially designed to accommodate varying conditions of temperature, loading, support conditions and heat transfer to which a structural member is exposed during a fire incident. The test furnace and the steel girder placement within the furnace are shown in Figure 3.9. This furnace and the actuator set-up allow simultaneous application of both thermal and structural loading on test specimens to simulate conditions experienced by a real structural member during a fire event.

The test furnace consists of a steel framework supported by four steel columns, with the furnace chamber inside the framework. The heating chamber of the furnace is 2.44 m wide, 3.05 m long, and 1.78 m high, and this produces a maximum heat power of 2.5 MW. Six natural gas burners located within the furnace to provide the thermal energy, while six type-K Chromel-alumel thermocouples distributed throughout the test chamber to monitor the furnace temperature during a fire test. During the course of the fire test, the gas supply is manually adjusted such that the furnace temperatures follow a pre-determined time-temperature curve of a fire, which can be either a standard fire or a design fire. To facilitate visual observations of test specimens during the fire test, two small view ports are provided on either side of the furnace wall. Four vertical pressure actuators are provided to apply loading on the test specimens.
3.2.5 Test conditions and procedure

The three steel girders tested under fire exposure have different flexural and shear capacities resulting from variations in sectional geometry of each girder (web slenderness, flange thickness, and stiffener spacing). Therefore, for the sake of comparison, the girders were subjected to different load levels; evaluated as a percentage of shear and/or flexural capacity of the girder at room temperature. The flexural capacity, shear capacity, fire scenario, and the load level on the three steel girders are shown in Table 3.1. Girder G1 was subjected to a single point load of 691 kN at mid-span, which is equivalent to 40% of its room temperature flexural capacity and to 27% of its shear capacity. Girders G2 and G3 were subjected to a single point load at mid-span representing 40% and 33% of their flexural capacities, respectively, which equates to 56% of their shear capacities. During fire tests all three steel girders were exposed to ASTM E119 fire from three sides, with slightly higher heating rate in the first five minutes of fire exposure.

During each fire test, one girder was tested by subjecting it to combined thermal and structural loading. The instrumented steel-concrete composite girder assembly was placed inside the furnace as shown in Figure 3.9(a). A length of 3.0 m of the mid portion of the girder was inside the furnace and directly exposed to fire. The fire test furnace was covered with a specially designed reinforced concrete slab lid to facilitate deep girders to experience three-sided fire exposure during the test as shown in Figure 3.9(b). The girders were simply supported at both ends, and a single point load was applied at the mid-span to simulate structural loading.

Prior to fire exposure, each girder was gradually loaded by incrementing hydraulic pressure in the actuator. Once the target load was reached, it was allowed to
stabilize for about 30 minutes. Then, the heating in the furnace was turned-on and furnace temperature was increased to follow ASTM E119 time-temperature fire curve. Throughout the fire test, the loading on the girder was maintained at the specified level. During fire tests, cross sectional temperature, mid-span deflection, out-of-plane displacement of the web, and axial displacement of the girders was recorded at 5 second time intervals.

The girders were considered to have failed and the tests were terminated when the mid-span deflection exceeded \( L/30 \) (where \( L \) is the span length) or when the girders experienced loss of capacity and could no longer sustain the applied loading (Wainman and Kirby, 1989).

### 3.3 Material Properties

To evaluate mechanical properties of steel and concrete used in fabrication of girder assemblies, strength tests were carried out on steel coupons and concrete cylinders. For evaluating tensile strength of steel, three coupons were cut from girders G2 and G3. For evaluating concrete strength, concrete cylinders and prisms were cast from concrete batch mix during fabrication of slab.

The selected steel coupons were tested in a MTS-810 testing machine by subjecting them to load increments till failure and stress-strain response was recorded. The significant points on stress-strain curve from uniaxial tensile strength tests are summarized in Table 3.3. The room temperature stress-strain response of A572 steel coupon 1 is shown in Figure 3.10. It can be seen that the general behavior of stress-strain relationship remained linear-elastic up to yielding, and then the response becomes nonlinear. No well-defined yield plateau was recorded in these tests. Once the peak stress
is reached, steel undergoes plastic deformation through unloading phase up to rupture.

Three concrete cylinders were tested on 14, 28, and test days to evaluate compressive strength and tensile strength. In addition, three concrete prisms were tested to evaluate room temperature flexural strength of concrete. A summary of strength properties of concrete and steel, as derived from room temperature tests, are tabulated in Table 3.3 and 3.4.

### 3.4 Experimental Results

Data generated from fire tests is utilized to trace the response of steel girders under fire conditions. Relative thermal and structural response, as well as failure patterns, are compared to evaluate the effect of web slenderness, stiffeners spacing, and load level on the response of steel bridge girders under fire conditions.

#### 3.4.1 Thermal response

In all three fire tests, girders G1, G2, and G3 were exposed to a typical ASTM E119 fire exposure. During these tests, cross-sectional temperatures were measured at different points on web, stiffeners, shear studs, and concrete slab at two traverse sections C-C and M-M. Recorded temperature data from these tests show that temperatures at sections C-C and M-M of girders G1, G2, and G3 virtually follow the same trend. The temperatures at selected points on each girder are plotted as a function of fire exposure time in Figure 3.11. Generally, temperatures at different points on steel girder increase at a much faster pace than that in concrete slab or in shear studs.

It can be seen from Figure 3.11 that the web temperatures in girders G2 and G3 increase at a slightly higher rate than that in the bottom flange. This can be attributed to the fact that web thickness is much smaller than that of the flange thickness in girders G2
and G3, as compared to G1. Also, temperatures in top flange of steel girders are much lower than that in bottom flange throughout fire exposure duration. This is mainly due to insulating effect of the concrete slab that prevents top surface of the top flange from being directly exposed to fire. Further, much of the heat from the top flange gets dissipated to concrete slab due to high thermal capacity of concrete.

It can be seen in Figure 3.11 that the temperature in shear stud remains much lower than that in top flange, despite the stud being welded to top flange of the girder. This is mainly due to the fact that shear studs are embedded in the concrete slab which acts as a heat sink. The temperatures in upper layer of concrete, both at mid-depth and at 25 mm depth from the top surface of the slab, increase at a very slow rate as compared to temperatures at lower part of slab (depth of 115 mm from the top surface of the concrete slab). This is due to the fact that bottom surface of concrete slab is directly exposed to the fire. The difference in temperatures between bottom flange and concrete slab leads to developing high thermal gradients across the depth of the composite girder section as illustrated in Figure 3.12. These thermal gradients increase with fire exposure time since the temperature at the bottom flange raises rapidly with time. Such thermal gradients induce significant thermal stresses, which in turn influence the structural response of fire exposed steel girders.

3.4.2 Structural response

The structural response of composite girders under fire conditions can be assessed by tracing mid-span deflections, web out-of-plane displacement, and axial displacement with fire exposure time. The composite girders, during fire exposure, were subjected to combined bending and shear effects. Thus, the structural response of steel girders is
highly influenced by flexural and shear interactions.

3.4.2.1 Mid-span deflection and web out-of-plane displacement

The flexural response of tested girders G1, G2, and G3 is illustrated in Figure 3.13(a), where progression of mid-span deflection is plotted as a function of fire exposure time. The shear response is shown in Figure 3.13(b), where out-of-plane displacement in girders is plotted as a function of fire exposure time. The measured out-of-plane displacement is at the center of the web panel that is adjacent to the mid-span of the steel girders. The time to failure in each girder, the time to initiation of web bucking, and the maximum axial displacement and out-of-plane displacement in each girder are summarized in Table 3.5.

The mid-span deflection in each girder generally increases with fire exposure time, however the rate of increase varies in later stages of fire exposure and is determined by the sectional geometry of each girder. The progression of deflection with time can be grouped into three stages, namely Stage 1, Stage 2, and Stage 3. In Stage 1, during first 10 minutes of fire exposure, mid-span deflection increases linearly till first yielding occurs in steel. In Stage 2, between 10 and 25 minutes of fire exposure, deflection starts to increase at a higher pace and this is mainly due to deterioration in strength and stiffness properties of steel with increasing steel temperatures. In Stage 3 of fire exposure, after 25 minutes, the mid-span deflection increases at a rapid pace due to the spread of plasticity in the web and/or bottom flange of steel girders, and also due to high-temperature creep effects.

It can be seen from Figure 3.13(a) that the mid-span deflection in girder G1 increases linearly up to about 10 minutes when the temperatures in the bottom flange and
web reaches about 300°C. The deflection in this stage of fire exposure is mainly due to significant thermal gradients that lead to high thermal stresses and curvature along the girder section. The developed curvature at this stage of fire exposure is independent of loading on the girder, since this curvature results mostly from the thermal gradient effect. In Stage 2 of fire exposure, the mid-span deflection starts to increase (between 10 and 25 minutes) due to degradation of the strength and stiffness properties of the steel as the bottom flange and web temperatures exceed 400°C. In the final stage of fire exposure (after 30 minutes), when steel temperature exceeds 600°C, the mid-span deflection increases at a rapid pace due to spread of plasticity in the bottom flange, and high-temperature creep effects, leading to formation of plastic hinge at the mid-span. The girder attains failure when mid-span deflection exceeds L/30 limit (at 40 minutes). At this time, the girder could not sustain the applied loading. As shown in Figure 3.13(b), the web of girder G1 did not experience any lateral displacement during the entire fire exposure, which implies that no web shear buckling occurred in girder G1.

Girder G2 exhibits similar flexural response as that of girder G1 in Stage 1 of fire exposure (see Figure 3.13(a)). However, the deflection response is slightly more flexible than that in girder G1, and this can be attributed to the differences in temperature progression in the two girders. G2 had a slender web as compared to G1, and hence temperature in bottom flange and web of girder G2 was about 450°C at 10 min., as compared to 300°C in girder G1. In Stage 2 of fire exposure, mid-span deflection in girder G2 increases at a faster pace than that in girder G1 due to rapid deterioration in strength and stiffness properties of steel. This faster degradation in mechanical properties of steel is due to higher temperature in the web and flanges of girder G2 resulting from
larger slenderness of web in G2. Hence, shear capacity in girder G2 deteriorates at a higher pace. As a result, at 27 minutes, when the temperature in the web reaches about 700°C, the web undergoes out-of-plane displacement as shown in Figure 3.13(b). In Stage 3 of fire exposure, mid-span deflection as well as the out-of-plane displacement of the web, in girder G2 increase at a very rapid pace till failure occurs at 35 minutes. This rapid rise in deflection and web lateral displacement is due to spread of plasticity in the web and bottom flange, and also due to high-temperature creep effects, which becomes predominant when steel temperature exceeds 700°C (Kodur and Esam, 2014).

Girder G3 exhibits similar flexural response as that of girder G2 till Stage 2 of fire exposure, as can be seen in Figure 3.13(a). The applied loading on girder G3, however, is 33% of the room temperature moment capacity as compared to 40% in the case of girder G2. This emphasizes the fact that deflection at the early stage of fire exposure is mainly due to thermal gradients and is independent of the load level. Both girders G2 and G3 experience similar temperature rise throughout fire exposure duration since they have similar cross-sectional dimensions. The only difference between girders G2 and G3 is the spacing of traverse stiffeners, which is \( a/D = 1.0 \) for Girder G2 and \( a/D = 1.5 \) for Girder G3. This closer spacing of transverse stiffeners in girder G2 results in higher shear capacity than that in girder G3. As a consequence, girder G3 experienced out-of-plane displacement at earlier time than that of girder G2 as can be seen in Figure 3.13(b). In Stage 3 of fire exposure, web lateral displacement in girder G3 increased rapidly due to spread of plasticity in web panels and also due to considerable degradation in shear capacity (of the web). On the other hand, mid-span deflection in girder G3 increased at a slower pace as compared to girder G2. Failure of girder G3 occurred at 38 minutes
through web shear buckling.

**3.4.2.2 Axial displacement**

The measured axial displacement at the supports in girders G1, G2, and G3 is plotted as a function of fire exposure time in Figure 3.14. These axial displacements were measured at end support of the steel girders close to the bottom flange. However, the steel girders were free to expand from both ends as they were simply supported. These axial displacements result from thermal expansion due to thermal gradients and also due to rotations at the ends of girder. It can be seen from Figure 3.14, that the axial displacement in each of the girder increases with fire exposure time. Further, the axial displacement in all three girders follow similar trend and increase almost linearly in Stage 1 and Stage 2 of fire exposure. However, in Stage 3 of fire exposure there is a significant variation in axial displacements with girder G3 experiencing much lower rise in displacement than girder G2, which in turn experiencing much lower displacement than girder G1. This variation can be attributed to web buckling phenomenon that occurs during Stage 3 of fire exposure. The higher axial displacement in the case of girder G1 is due to the fact flexural factors dominate fire response and no web buckling occurs due to thicker web. In the case of girder G2, shear factors contribute to response of the girder and web buckling occurs towards the end. In the case of girder G3, shear factors dominate at the start of Stage 3 and thus web buckling occurs early in to fire exposure.

**3.4.2.3 Failure modes**

Visual observations were made during and following fire tests to capture significant changes and failure modes in steel girders. These observations, together with recorded mid-span deflections and out-of-plane displacements of the web, were utilized
to identify failure limit states in each of the steel girder.

Test observations on girder G1 indicate that flexural response dominated the entire duration of fire exposure. The out-of-plane web displacement of girder G1 plotted in Figure 3.13(b) clearly infers that this girder had adequate shear capacity to minimize any web lateral displacement. However, this girder experienced significant degradation in flexural capacity (see Figure 3.13(a)) and thus failed through yielding at the bottom flange. Test observations indicated no sign of local buckling of the web since the web of this rolled girder is much thicker. The condition and failure pattern of this girder after fire test is shown in Figure 3.15. The girder forms a V shape at failure due to large deflections, resulting from large rotations at the girder ends and also due to crushing of concrete at the mid-span region.

The failure pattern and level of deformation in girder G2 is illustrated in Figure 3.16. Data from fire test indicate that this girder exhibited flexural response till about 27 minutes into fire exposure, and afterwards web panel experienced buckling due to slender web. At this point, the shear capacity, as well as flexural capacity, degraded significantly. Figure 3.13(b) and Figure 3.16 show that extent of web buckling in G2 was moderate due to closer spacing of traverse stiffeners. Much of the lateral displacement in the web was recovered after the girder cooled down to room temperature following the fire test. The occurrence of large rotations in this girder infers that flexural capacity dominated the response for most of the fire exposure duration, with web buckling occurring just prior to failure. The failure of this girder, as shown in Figure 3.16, was through combined effects of yielding of bottom flange and web shear buckling.

The failure and distortion pattern of girder G3 after fire exposure is illustrated in
Figure 3.17. This girder experienced high level of mid-span deflection, web buckling, rotations, and concrete crushing prior to failure. This can be attributed to the fact that the level of loading (based on shear web buckling capacity) in girder G3 was much higher than in the case of G2. The overall shear capacity in stiffened steel girders G2 and G3, arises from the combination of web buckling strength tension field action facilitated by the stiffeners. The shear web buckling strength, which depends on $D/t_w$ ratio and $a/D$ ratio, is higher in girder G2 as compared to girder G3 due to closer spacing of stiffener. Therefore, girder G3 experienced web buckling at earlier time (t=19 min.) as compared to girder G2 (t=27 min.) and this lead to significant distortion of the web. Therefore, the failure of steel girder G3 was through web shear buckling, as illustrated in Figure 3.17.

Figure 3.18 illustrates the level of distortion at 30 minutes of fire exposure in girders G1, G2, and G3. These magnified images correspond to the location of mid-span bearing stiffener in the central web panel. It can be seen from this figure that all three girders, G1, G2, and G3, experienced significant level of yielding at the bottom flange. Further, it can be seen that there is no visual web buckling in girders G1 and G2, however the web panel in girder G3 experienced significant buckling at 30 minutes.
Table 3.1: Summary of sectional dimensions, test parameters, and loading conditions of tested girders

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Girder G1</th>
<th>Girder G2</th>
<th>Girder G3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sectional geometry</strong></td>
<td></td>
<td>Girder shape</td>
<td>Rolled section W 24x62</td>
<td>Built-up plate girder</td>
</tr>
<tr>
<td>Span (between supports), mm</td>
<td></td>
<td>3658</td>
<td>3658</td>
<td>3658</td>
</tr>
<tr>
<td>Total length (end to end), mm</td>
<td></td>
<td>4167</td>
<td>4167</td>
<td>4167</td>
</tr>
<tr>
<td>Flange plate (b_f x t_f), mm</td>
<td></td>
<td>177.8 x 12.7</td>
<td>177.8 x 12.7</td>
<td>177.8 x 12.7</td>
</tr>
<tr>
<td>Web plate (D x t_w), mm</td>
<td></td>
<td>577.9 x 11.1</td>
<td>587.4 x 4.8</td>
<td>587.4 x 4.8</td>
</tr>
<tr>
<td>Concrete slab (b_eff x t_s), mm</td>
<td></td>
<td>813 x 140</td>
<td>813 x 140</td>
<td>813 x 140</td>
</tr>
<tr>
<td>End panel width (S), mm</td>
<td></td>
<td>254</td>
<td>254</td>
<td>254</td>
</tr>
<tr>
<td>Web slenderness ratio (D/t_w)</td>
<td></td>
<td>52</td>
<td>123.3</td>
<td>123.3</td>
</tr>
<tr>
<td>Panel aspect ratio (a/D)</td>
<td></td>
<td>N/A</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Stiffener</strong></td>
<td></td>
<td>Bearing stiffeners- mid-span (w x t_stf), mm</td>
<td>76.2 x 12.7</td>
<td>76.2 x 15.87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bearing stiffeners- supports (w x t_stf), mm</td>
<td>76.2 x 9.5</td>
<td>76.2 x 9.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Intermediate stiffeners (w x t_stf), mm</td>
<td>N/A</td>
<td>76.2 x 9.5</td>
</tr>
<tr>
<td><strong>Capacity at ambient temperature</strong></td>
<td></td>
<td>Flexural (composite), kN.m</td>
<td>1569</td>
<td>1220</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shear web buckling (V_{cr}), kN</td>
<td>1278</td>
<td>336</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Total shear capacity (V_u), kN</td>
<td>1278</td>
<td>480</td>
</tr>
<tr>
<td><strong>Applied load</strong></td>
<td></td>
<td>Applied load, kN</td>
<td>691</td>
<td>538</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Applied load/flexural capacity</td>
<td>40%</td>
<td>40%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Applied load/total shear capacity</td>
<td>27%</td>
<td>56%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Applied load/shear web buckling</td>
<td>27%</td>
<td>80%</td>
</tr>
<tr>
<td><strong>Fire exposure</strong></td>
<td></td>
<td>ASTM E119</td>
<td>ASTM E119</td>
<td>ASTM E119</td>
</tr>
</tbody>
</table>
Table 3.2: Mix-proportions of concrete used in fabrication of concrete slab

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement type I/II</td>
<td>230 kg/m³</td>
</tr>
<tr>
<td>Fly ash-Class C</td>
<td>77 kg/m³</td>
</tr>
<tr>
<td>Fine aggregate (sand)</td>
<td>930 kg/m³</td>
</tr>
<tr>
<td>Coarse aggregate (gravel)</td>
<td>985 kg/m³</td>
</tr>
<tr>
<td>Water</td>
<td>154 kg/m³</td>
</tr>
</tbody>
</table>

Table 3.3: Mechanical properties of A572 Gr. 50 steel used in fabrication of steel girders

<table>
<thead>
<tr>
<th>Coupon</th>
<th>$f_{y0.2}$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>Strain at rupture</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>436.5</td>
<td>498</td>
<td>0.1432</td>
<td>218,250</td>
</tr>
<tr>
<td>2</td>
<td>503.1</td>
<td>570</td>
<td>0.1249</td>
<td>251,550</td>
</tr>
<tr>
<td>3</td>
<td>501.4</td>
<td>566</td>
<td>0.1347</td>
<td>250,700</td>
</tr>
</tbody>
</table>

Table 3.4: Properties of concrete used in fabrication of concrete

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Compressive strength ($f_c$, MPa)</th>
<th>Indirect tensile strength ($f_t$, MPa)</th>
<th>Flexural strength ($f_{cr}$, MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>50</td>
<td>4.0</td>
<td>4.9</td>
</tr>
<tr>
<td>28</td>
<td>54</td>
<td>3.8</td>
<td>5.6</td>
</tr>
<tr>
<td>210 (test day)</td>
<td>66</td>
<td>3.5</td>
<td>5.9</td>
</tr>
</tbody>
</table>

Table 3.5: Summary of fire test results of girders G1, G2, and G3

<table>
<thead>
<tr>
<th>Description</th>
<th>Steel girder G1</th>
<th>Steel girder G2</th>
<th>Steel girder G3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mid-span deflection limit, (mm)</td>
<td>L/30 = 122</td>
<td>L/30 = 122</td>
<td>L/30 = 122</td>
</tr>
<tr>
<td>web lateral displacement, (mm)</td>
<td>0</td>
<td>10</td>
<td>19</td>
</tr>
<tr>
<td>Axial displacement at one end, (mm)</td>
<td>65.9</td>
<td>48.1</td>
<td>34.5</td>
</tr>
<tr>
<td>Time to initiation of web buckling, (min)</td>
<td>0</td>
<td>27</td>
<td>19</td>
</tr>
<tr>
<td>Time to failure, (min)</td>
<td>40</td>
<td>35</td>
<td>38</td>
</tr>
<tr>
<td>Failure mode</td>
<td>Flexural (yielding)</td>
<td>Flexural (yielding)/ shear (web buckling)</td>
<td>Shear (web buckling)</td>
</tr>
</tbody>
</table>
Figure 3.1: Flowchart for strength limit state design of steel bridge girder based on LRFD 6.10.6 provisions in AASHTO
Figure 3.2: Flowchart for composite section in positive flexure for steel bridge girder based on LRFD 6.10.7 provisions in AASHTO
Figure 3.3: Flowchart for shear design of steel bridge girder based on LRFD 6.10.9 provisions in AASHTO
(a) Longitudinal section

(b) Traverse sections C-C and M-M

Figure 3.4: Layout of steel girders G1, G2, and G3 used in fire tests
(a) Fabricated steel girders before pouring of concrete slab

(b) Details of concrete slab reinforcement

Figure 3.5: Illustration of fabricated girders G1, G2, and G3
Figure 3.6: Layout of thermocouples on steel girders G1
Figure 3.7: Layout of thermocouples on steel girders G2, and G3
(a) Mid-span deflection measurement 

(b) Axial displacement measurement 

(c) Web out-of-plane displacement measurement 

Figure 3.8: LVDT set-up to measure deflection and displacements
Figure 3.9: Placement of steel girder in the furnace at the structural fire testing facility at Michigan State University
Figure 3.10: Room temperature stress-strain response of A572 Gr50 steel

(a) Girder G1

Figure 3.11: Measured temperature profiles in girders G1, G2, and G3 as a function of fire exposure time
Figure 3.11 (cont’d)

(b) Girder G2

(c) Girder G3
Figure 3.12: Thermal gradient along the depth of girder section in steel girders G1, G2, and G3.
Figure 3.12 (cont’d)

(c) Girder G3
Figure 3.13: Variation of mid-span deflection and web out-of-plane displacement as a function of fire exposure time in steel girders G1, G2, and G3
Figure 3.14: Measured axial displacement during the fire tests for G1, G2, and G3
(a) Steel girder G1 after fire exposure

(b) No web buckling  (c) Rotation and thermal expansion  (d) Crushing of concrete

Figure 3.15: Illustration of failure pattern in steel girder G1 after exposure to fire
Figure 3.16: Illustration of failure pattern in steel girder G2 after exposure to fire

(a) Steel girder G2 after fire exposure

(b) Web buckling  (c) Rotation and thermal expansion  (d) Crushing of concrete
Figure 3.17: Illustration of failure pattern in steel girder G3 after exposure to fire

(a) Steel girder G3 after fire exposure

(b) Web buckling   (c) Rotation and thermal expansion   (d) Crushing of concrete
Figure 3.18: Illustration of mid-span deflection and out-of-plane web displacement at 30 minutes of fire exposure in girders G1, G2, and G3
3.5 Summary

Fire resistance tests are carried out to study the behavior of typical steel girders, used in bridges, under fire conditions. As part of experimental studies three steel-concrete composite girders were tested under simultaneous loading and fire exposure. Test variables included; load level, web slenderness, and spacing of stiffeners. Results from fire tests indicate that typical steel girders can experience failure under standard fire conditions in about 30-35 minutes. The time to failure and mode of failure in fire exposed steel girders is highly influenced by web slenderness, spacing of stiffeners, and type of fire exposure. Steel bridge girders fail through flexural yielding when web slenderness is around 50, however failure mode changes to web shear buckling when web slenderness in girders exceed 100.
CHAPTER FOUR

4. MATERIAL PROPERTY TESTS

4.1 General

High-temperature properties of steel are crucial for evaluating response of structural members under fire conditions. The properties of steel vary based on the composition and type of steel. Steel used in construction applications are classified into different categories based on chemical composition, tensile properties, and fabrication process as carbon steel, high-strength low alloy steels (HSLA), heat-treated carbon steels, and heat-treated constructional alloy steels (Brockenbrough and Merritt, 2005). High-strength low-alloy steels provide better performance in terms of strength, weldability and corrosion/weather resistance. One commonly available HSLA is ASTM 572 Gr. 50 steel and this steel, with different chemical composition than conventional steels, is widely used in bridge and building applications.

A review of literature indicates that very few tests have been carried out to evaluate mechanical properties of structural steel at high-temperature. The limited tests reported in literature were carried out on carbon steels commonly used in structural member of buildings (such as A36 and A992 or their equivalent types) and there is no mechanical property data specific to ASTM A572 steel as discussed in Chapter 2.

To fill this knowledge gap, a comprehensive experimental study was undertaken on A572 Gr. 50 steel with the objective of generating high-temperature property data for
use in fire resistance analysis of structural members. Details of high-temperature mechanical property tests carried out on A572 Gr. 50 steel coupons is presented in this chapter. The experimental investigation includes; tensile strength (stress-strain response), residual strength (residual stress-strain response), and creep tests on steel coupons at various temperatures and stress levels. Results from these tests are utilized to propose relations for high-temperature and residual strength properties of A572 steel.

4.2 Experimental Studies

A comprehensive test program was designed to undertake material properties tests on ASTM A572 steel. The tensile and residual strength tests were carried in the temperature range of 200-1000°C, while creep tests were carried out at various temperatures in 400-800°C range and at various stress levels in the range of 11-55% of room temperature yield stress (Kodur and Aziz, 2014).

4.2.1 Test specimens

For high-temperature mechanical properties tests, namely tensile strength, residual strength, and creep, rectangular steel coupons were cut from A572 Gr. 50 steel sheets. The chemical composition of A572 steel is slightly different from that of conventional A36 steel. The chemical composition of both types of steel, together with actual composition of A572 steel used in current test program (as specified by the manufacturer) is tabulated in Table 4.1. The total length of test coupon measured 609.6 mm and the length of tapered section is 304.8 mm. The cross section of the reduced section is 9.53 x 12.7 mm. The configuration of a typical test coupon used in creep and tensile strength tests is shown in Figure 4.1.
4.2.2 Test equipment

Three different test setups were used to carry out high-temperature tensile strength, residual strength, and creep tests on ASTM A572 Gr. 50 steel.

4.2.2.1 Tensile strength tests

The test setup for tensile strength tests comprised of a steel frame that is capable of applying tensile or compressive loads, an electric furnace, and a data acquisition system. The universal testing machine has a loading capacity of 200 kN and the axial deformations of the specimen in tension is measured by a displacement transducer (± 38 mm LVDT), with 0.0254 mm sensitivity that is placed outside the furnace. The programmable furnace that surrounds the steel coupon enables generation of any target temperature up to 1000°C at any given rate of temperature rise and keeps the target temperature constant inside the furnace for the test duration. Three thermocouples are mounted on the furnace to measure temperatures at the upper, middle and lower zones. The average values of these three thermocouples are taken to be the furnace temperature. The furnace has an inner diameter of 203 mm and a height of 305 mm. Therefore the heating length of the specimen is to be 305 mm. The loading machine and the furnace are connected to data acquisition system and computer wherein temperature, load, and displacement during the test are recorded. The test set-up for high-temperature tensile test is illustrated in Figure 4.2.

4.2.2.2 Residual strength tests

The specimens after heating in a furnace and cooling to room temperature are tested in a MTS-810 strength machine to evaluate residual strength of steel. This test set-up is comprised of an universal testing machine and a data collection system. The
universal testing machine has a loading capacity of 250 kN and the axial displacement of the specimen in tension is measured through a MTS extensometer. The data are recorded using advanced data acquisition system. The test set-up for residual strength test is shown in Figure 4.3.

4.2.2.3 Creep tests

For undertaking high-temperature creep tests, a custom built integrated load-heating furnace equipment was utilized. This test set-up comprised of a steel frame for applying tensile loading; a loading lever arm, an electric furnace, and a data acquisition system. The axial tensile load is applied on the coupon using suspended concrete blocks via a lever arm. The concrete blocks are weighed to determine the magnitude of loading. This loading system allows the load to be maintained constant during the entire duration of strength test. The axial deformation in the coupon is measured by a displacement transducer (+/- 38 mm LVDT), with 0.0254 mm tolerance, that is placed outside the furnace.

The electric furnace comprises of cylindrical chamber with an inner diameter of 203 mm and a height of 305 mm. Therefore, the heating length of the test specimen is to be 305 mm. The test coupon is mounted between the base steel girder that is fixed to the floor and the top steel girder that moves vertically with the loading arm. The programmable furnace enables generation of a target temperature, up to 1000°C, at a specified rate of temperature rise and keeps the target temperature inside the furnace constant for the entire duration of strength (creep) test. Three built-in thermocouples are mounted inside the furnace to measure temperatures at upper, middle and lower zones and the average readings of these three thermocouples is taken as the furnace
temperature. The furnace and LVDT are connected to a data acquisition system and a computer through which temperatures and displacements during the test are recorded. A schematic of test set-up for high-temperature creep test is shown in Figures 4.4 and 4.5.

4.2.3 Test procedure

Tensile strength and creep of steel at a specific temperature can be evaluated through transient or steady-state tests. Although transient-state tests are more realistic in simulating real fire conditions, steady-state tests are more commonly adopted due to ease of undertaking these tests (Kodur and Aziz, 2014). In the current study, steady-state test conditions were simulated for undertaking high-temperature strength and creep tests.

4.2.3.1 Tensile strength test

Tensile strength tests were conducted at temperatures of 20, 400, 500, 600, 700, and 800°C. For tensile strength test, the steel coupon was placed in the furnace after attaching a thermocouple at mid-length of the reduced area of the specimen as shown in Figure 4.2. This thermocouple records the actual temperature progression on the coupon during the test, since temperature inside the furnace might not be same as the specimen temperature. Following the placement of coupon, the furnace is turned-on to heat-up the coupon to predefined target temperature. A rate of heating of 10°C/min was used, as done by other researchers (Wang et al., 2012). When the target temperature is attained, the specimen was kept at this temperature for 30 minutes so as to attain a steady-state condition throughout the specimen. Then, a tensile load was applied incrementally on the coupon, while keeping the target temperature constant. The applied loading is controlled manually at a loading rate of 0.5 kN/s. The tensile displacement during the test was measured using high sensitive LVDT. Both LVDT and the loading cell were connected to
the data acquisition system to record the readings during the test.

### 4.2.3.2 Residual strength test

The residual tensile tests were conducted at temperatures of 20, 400, 500, 600, 700, 800, and 1000°C. For residual tensile tests, the steel coupon was placed in the furnace and heated to target temperature at a heating rate of 10°C/min. Then, the specimen was maintained at the target temperature for 2 hours. After that, heating in the furnace was turned-off and the coupon was allowed to cool down to room temperature. Two methods of cooling, namely air cooling and quenching in water were carried out to evaluate the effect of cooling on strength recovery. In the first method, the coupons were left in the furnace after heating, to naturally cool down to room temperature, while in the second method the specimens were quenched in water to cool down to room temperature. No loads were applied on the specimens during heating or cooling phase. Following cooling, residual tensile tests were carried out on the coupons as per (ASTM E8, 2012) provisions using MTS-810 machine test.

The placement of coupon for residual tensile strength test set-up is shown in Figure 4.3. Displacement control loading was applied using a displacement rate of 0.002 mm/sec up to 3 mm displacements (yielding) and then the rate of displacement rate was raised to 0.021 mm/sec up to rapture.

### 4.2.3.3 Creep test

Creep tests were conducted at different temperatures and stress levels in a custom built test set-up described above. For these creep tests, steel coupons were prepared by mounting a thermocouple at mid-length of the tapered cross section as shown in Figure 4.6. This thermocouple is to monitor actual temperature progression in the specimen.
during the creep test. Then, the steel coupon was placed inside the electric furnace and special care was taken to ensure the coupon to be at the centre of the furnace and the loading frame. Following this, a predefined load was gradually applied through a suspended concrete block and this tensile force gets transferred to the coupon through the steel beam (acting as a lever arm). Special care was taken in the design of loading-frame to ensure the loading to be perfectly axial. The total load, that is required to develop a target stress level at the tapered cross section of the coupon, was applied before heating the specimen.

After few minutes of application of loading, the furnace is turned-on to be heated to a target temperature at a heating rate of 10°C/min. Once the target temperature is attained, the steel coupon was maintained at this temperature for 30 minutes to attain a steady-state condition and then the specimen was maintained at the target temperature for the entire test duration. The creep displacement during the test is measured using high sensitive LVDT that is placed outside the furnace. The total length of tapered section of the coupons that is exposed to high-temperature during creep tests is 304.8 mm and this is taken to be the gauge length to calculate corresponding creep strain ($\Delta L/L$). Using this procedure, creep tests were conducted at target stress levels of 11%, 25%, 40% and 55% of room temperature yield strength, and at temperatures of 400, 500, 550, 600, 650, 700, 750, and 800°C.

4.3 Experimental Results

Data from material tests on A572 Gr. 50 steel is utilized to generate the high temperature stress-strain relations, residual stress-strain relations, and high-temperature creep response. Results from these tests are compared with values derived using relevant
relations present in codes and literature for conventional (carbon) structural steel. Also, modified strength relations for high-temperature tensile and residual strength of A572 steel are also proposed.

4.3.1 Tensile strength at elevated temperatures

The variation of stress-strain response of A572 steel at elevated temperatures is evaluated using data from uniaxial tensile strength tests carried out at various target temperatures. The displacements recorded at various load levels, during tension tests, are utilized to generate stress-strain response of A572 steel and theses are plotted in Figure 4.7. It can be seen in this figure that the general trend of stress-strain curve is linear-elastic up to yielding of steel, which varies with temperature level, followed by nonlinear response. Therefore the stress at which yielding occurs in steel coupon is different at different temperatures.

No well-defined yield plateau is recorded at the room temperature test for A572 steel. This can be attributed to very low amount of carbon content and also presence of alloy elements such as Vanadium in the chemical composition of A572 steel as tabulated in Table 4.1. Past the yielding stress, steel undergoes plastic deformation with increasing stress up to reaching ultimate stress point. Once the ultimate stress is reached, plastic deformation continues through unloading phase (necking) up to rapture. Also, it can be seen that A572 steel exhibit some level of strain hardening at temperatures 20-400°C, while this phenomena disappears at temperatures beyond 400°C. This is in agreement with reported trend in literature (EC3, 2005).

For the tensile strength tests, the yield stress is evaluated based on a strain level of 0.2%, while the ultimate stress is defined as the maximum stress reordered during the
tensile test on the coupon. The significant points of stress-strain curve from high-temperature uniaxial tensile strength tests are summarized in Table 4.2. The strength and stiffness reduction factors given in this table are calculated as a ratio of yield strength at a given elevated temperature to that at room temperature. As illustrated in this table, with increasing temperature, modulus of elasticity (stiffness), yield and ultimate stress (strength) of the steel decrease significantly. The extent to which steel undergoes plastic deformation, measured in terms of ductility, can be evaluated through percentage of elongation and reduction in area at rupture. These values at various temperatures are also tabulated in Table 4.2. Test data in this table indicate that, with increasing temperature, the elongation of steel decreases except for temperatures beyond 700°C. However, the reduction in cross-sectional area decreases with increasing temperature from 20 to 800°C. This can be attributed to necking phenomenon that is well defined at room temperature as compared to high-temperature. Therefore, A572 steel exhibits higher expansion, but less necking.

Results from tensile strength test on coupons are utilized to drive strength (yield strength) and stiffness (elastic modulus) reduction factors for A572 steel at high-temperatures. These reduction factors are compared with reduction factors specified in ASCE, EC3 codes, and literature, in Figure 4.8. The strength and stiffness reduction factors specified in EC3 and ASCE are for conventional carbon steel. It can be seen from Figure 4.8(a) that the strength reduction curve for A572 steel lies between the strength reduction curves predicted from ASCE and EC3. Comparisons in this figure shows that strength reduction factors for A572 steel is more comparable with ASCE values for temperatures up to 400°C, while it compares well with EC3 for temperatures beyond
Accordingly, applying EC3 reduction factors to evaluate tensile strength of A572 steel in the temperature range 300-600°C can lead to overestimation, while applying ASCE factors beyond temperature of 400°C can lead to underestimation of tensile strength. These variations are attributed to chemical composition of A572 steel that is different as compared to that of carbon steel as illustrated in Table 4.1.

Reduction factors for strength and elastic modulus of A572 steel are also compared with corresponding values from literature (Poh model) as shown in Figure 4.8 (Poh, 2001). Results from Poh model, developed for conventional steel, show significant variation in the strength and elastic modulus of A572 steel. It can be seen that strength reduction curve predicted from Poh model lies below corresponding reduction curves from ASCE, EC3 and A572 steel, while, elastic modulus reduction curve from Poh model lies above the elastic modulus curves from ASCE, EC3 and A572 steel. The substantial variation between Poh model and reduction factors of A572 is due to the fact that Poh model is a general stress-stress model and does not take the chemical composition of steel into account.

4.3.2 Residual strength

The load-displacement data recorded during uniaxial residual tensile strength tests are utilized to generate residual stress-strain curves after cooling down from various target temperatures. The residual stress-stain response of heated A572 specimens following either air cooling or water quenching is plotted in Figures 4.9 and 4.10 respectively. The residual yield strength is evaluated at 0.2% strain offset, while maximum stress reached during the test is taken as the ultimate stress. The significant points on residual stress-strain curve in air cooled and water quenched specimens, as
obtained from uniaxial tensile strength tests, are summarized in Tables 4.3 and 4.4 respectively. The strength and stiffness factors given in these tables are calculated as a ratio of residual yield strength to that yield strength at room temperature.

The residual strength factors for heated A572 steel coupons after air cooling and water quenching are plotted in Figure 4.11. It can be seen that A572 steel recovers almost all (100%) of its room temperature strength (yield and ultimate) when it is heated to temperature below 600°C and then cooled-down to room temperature regardless the method of cooling (air cooling or water quenching). But, A572 steel loses up to 40% of its room temperature yield strength and up to 30% of its room temperature ultimate strength when it is heated to 800°C and then air cooled to ambient temperature. However, when the same steel is heated to 800°C and then cooled through quenching in water, a 75% of its room temperature yield strength and almost 95% of its room temperature ultimate strength are recovered.

Results from residual tensile strength tests tabulated in Tables 4.3 and 4.4 indicate that the residual modulus of elasticity of A572 steel is fully reversible when heated to 700°C and cooled down to room temperature regardless the method of cooling. When the steel is heated to 800°C and then cooled through water quenching, 45% reduction in residual elastic modulus is observed. Also, heating A572 steel and cooling to room temperature through water quenching results in significant ductility (elongation) reduction as compared to heating and air cooling which experiences lower reduction in ductility.

**4.3.3 Creep response at elevated temperatures**

Data from high-temperature creep tests is utilized to derive creep response of
A572 steel at various stress levels and temperatures. The stress levels and target temperatures at which these tests carried out are tabulated in Table 4.5. The stress level represents ratio of stress due to specified applied loading to room temperature yield stress.

The measured creep strain at four stress levels is plotted as a function of time in Figure 4.12. The general trend of creep response can be grouped into three stages, namely primary (stage I), secondary (stage II), and tertiary creep (stage III). Any primary creep (stage I) that result in these specimens has occurred prior to heating of coupons and this is quite small as compared to secondary and tertiary creep. Therefore, the initial total strain at the start of heating (time= 0) in creep response curves shown in Figure 4.12, represent the sum of mechanical strain from the applied load, thermal strain from heating to target temperature, and strain due to primary creep.

The increase in secondary creep strain with time during Stage II is dependent both on temperature and stress levels. At low stress level of 11%, there is little secondary creep generated at 500, 600, and 700°C. However, at higher temperatures of 750, and 800°C, secondary creep increased at a high rate at this stress level. For moderate stress level of 25%, the secondary creep starts to dominate at slightly lower temperature of 700°C. For high stress level of 40% and 55%, secondary creep increased substantially at temperatures of 600°C and 550°C respectively. The slow rise in creep strain with time in Stage II is due to movement of dislocations that counteract the strain hardening resulting in a balance between strain hardening and thermal softening. Secondary creep is considered to be highly important under fire exposure conditions because it dominates the creep response, occur at a constant rate, and over short period of time.
In Stage III, the tertiary creep increases at a faster rate due to reduced cross section of the specimen resulting from necking phenomenon which results in higher stresses for the same level of applied loading. Finally, the material (steel) flows and this leads to rupture of specimen under the combined effect of mechanical loading and temperature. At high-temperature and stress levels, it becomes difficult to distinguish between Stage II and Stage III of creep because of high rate of creep and accelerated flow of material at high-temperatures.

A review of results in Figure 4.12 indicates that creep deformations are generally appreciable at temperatures above 500°C. At a given stress level, the creep deformation increases with increasing temperature beyond 500°C. This is due to diffusion of atoms within material grains resulting from temperature rise. When temperatures are in the range of 600-800°C, which represents 30% to 50% of steel melting temperature, grain boundary diffusion tends to be the dominant mechanism that results in accelerated creep strain, while bulk diffusion tends to dominate creep deformation in temperature range of 800-1400°C (50-99% of steel melting temperature) (Ashby and Jones, 2005). Hence, for the case of 11% stress level, when steel temperatures rise from 600 to 750°C, creep strains increase significantly and this leads to rupture of the specimen. Figure 4.12 clearly show that for any stress level, there is a critical temperature at which creep deformations tends to accelerate at a rapid pace and produces failure. This critical temperature typically corresponds to initiation of tertiary creep. Higher the stress level, lower is the critical temperature.

Different failure modes that resulted in test specimens at various temperature and stress levels are shown in Figure 4.13. At higher temperature and under low stress levels,
the failure of test specimens is through a well-defined necking phenomenon, indicating ductile response. For example, failure at 800°C and stress level of 11% is in ductile mode since the specimen experienced significant elongation just prior to failure, mainly due to very high creep deformations. However, at lower temperatures and higher stress levels, the failure is through brittle fracture and this can be seen from flat surface at the mid-length of specimens (see Figure 4.13). For example, at 550°C and stress level of 55%, the failure is through brittle fracture and mainly due to relatively lower creep deformations.

4.3.4 Creep response at various stress levels

The effect of stress level on creep response of A572 steel is illustrated in Figure 4.14. The creep strain is plotted at temperature of 650°C for stress levels of 25% and 40%. It can be seen, for moderate stress level of 25%, secondary creep increases with time at a constant rate up to 240 minutes, and then the creep strain increases rapidly in the tertiary creep stage up to rupture. However, at 40% stress level, it becomes difficult to distinguish between secondary and tertiary creep resulting from high creep rate generated due to higher stress level. The trends in the figure indicate that creep deformation increases substantially with increasing stress level. Also, data plotted in Figure 4.12(d) show that creep deformation can be critical for stress levels above 50% and will produce failure of specimens. Increased creep strain at stress level of 40% or more can be attributed to dislocation movement due to diffusion of atoms that occur at higher temperatures and under high stress levels.

Figure 4.15 shows total creep strain just prior to rupture in steel coupon as a function of temperature at four stress levels. The total creep strain at rupture increases with temperature at a given stress level. Furthermore ductility, which is the ability of steel
to deform before rupture, decreases with increasing stress level. As it can be seen in Figure 4.15, the specimens under high stress levels of 40% and 55% ruptured at lower creep strain than those subjected to lower stress level of 11% and 25%.

4.4 Design Recommendations for Mechanical Properties of A572 Steel

High-temperature properties of steel are crucial for carrying out fire resistance analysis to trace the response of structural members under fire conditions. Similarly, residual strength properties are required to undertake residual capacity evaluation on fire exposed structural members. However, these properties vary depending on the type and chemical compositions of steel. For deriving high-temperature properties of A572 steel, a nonlinear regression analysis, with least sum of squares of errors, is carried out on the data generated from high-temperature tensile and residual strength tests on A572 Gr. 50 steel coupons. Using the results from this analysis, the following relations are proposed for strength and stiffness reduction factors at elevated temperature, as well as residual strength of A572 steel Gr. 50 after cooling to room temperature under different cooling regimes. The proposed relations can be utilized for undertaking fire resistance analysis of steel structures, including bridge girders, and to evaluate the residual strength of fire exposed structural members.

4.4.1 Proposed reduction factors for strength and elastic modulus

To predict high-temperature tensile strength and elastic modulus of A572 Gr. 50 steel, the following relations are proposed:

\[
\frac{f_{y,R}}{f_y} = (-1.75e - 11)T_s^3 - (1.38e - 6)T_s^2 + (1.52e - 5)T_s + 1.0 \quad \ldots \ldots \quad [4.1]
\]
4.4.2 Proposed reduction factors for residual strength

To predict residual strength of A572 Gr. 50 steel after heating and cooling to room temperature using two methods of cooling, the following relations are proposed:

a) Air cooling after heating:

For $20^\circ C \leq T_s < 400^\circ C$

$$\frac{f_y}{f_y} = 1.0 \quad \text{.................................................................}[4.4]$$

For $400^\circ C \leq T_s \leq 1000^\circ C$

$$\frac{f_y}{f_y} = (1.0e - 8)T_s^3 - (2.12e - 5)T_s^2 + (1.31e - 2)T_s - 1.43 \quad \text{.................................................................}[4.5]$$

$$\frac{f_u}{f_u} = (7.64e - 9)T_s^3 - (1.68e - 5)T_s^2 + (1.1e - 2)T_s - 1.26 \quad \text{.................................................................}[4.6]$$

b) Water quenching after heating:

For $20^\circ C \leq T_s \leq 1000^\circ C$

$$\frac{f_y}{f_y} = (5.94e - 12)T_s^4 - (9.71e - 9)T_s^3 + (4.04e - 6)T_s^2 - (3.6e - 4)T_s + 1.0 \quad \text{.................................................................}[4.7]$$

$$\frac{f_u}{f_u} = 0.95 \quad \text{.................................................................}[4.8]$$

Prediction from proposed equations [4.1-4.8] as compared to corresponding tests data are shown in Figures 4.16-4.19. It can be seen that predictions from the proposed relations fit very well with results from tests data.
4.4.3 Proposed critical temperatures for creep

Creep deformations can dominate structural response of steel structures under fire conditions, especially when steel temperatures exceed 500°C. For a specific type of steel, creep strain is mainly dependent on temperature and stress level. Therefore defining a critical temperature limit for creep at different stress levels can be highly useful for evaluating fire response of steel structural members. Critical temperature for creep can be defined as the temperature that corresponds to onset of tertiary creep and this point is an indication of imminent failure of a structural member exposed to fire. Based on test data generated in this study, the critical temperature for four stress levels are derived as the temperature corresponding to onset of tertiary creep and these values are tabulated in Table 4.6. Under fire exposure conditions, structural members are typically assumed to be subjected to a loading of about 50% of room temperature capacity which corresponds to a stress level of 50% (Franssen et al., 2009). Hence, stress level beyond 50% is not of too much interest from the point of fire design of structures.
### Table 4.1: Chemical composition of A572 Gr 50 steel as compared to A36 steel

<table>
<thead>
<tr>
<th>Chemical composition</th>
<th>A36 steel (ASTM limits)</th>
<th>A572 Grade 50 steel (ASTM limits)</th>
<th>A572 Grade 50 steel (Actual)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon (C) %</td>
<td>0.25 - 0.290</td>
<td>0.23 max</td>
<td>0.06</td>
</tr>
<tr>
<td>Manganese (Mn) %</td>
<td>1.03</td>
<td>1.35 max</td>
<td>0.91</td>
</tr>
<tr>
<td>Phosphorous (P) %</td>
<td>0.040</td>
<td>0.04 max</td>
<td>0.009</td>
</tr>
<tr>
<td>Silicon (Si) %</td>
<td>0.280</td>
<td>0.40 max</td>
<td>0.02</td>
</tr>
<tr>
<td>Sulfur (S) %</td>
<td>0.050</td>
<td>0.05 max</td>
<td>0.009</td>
</tr>
<tr>
<td>Copper (Cu) %</td>
<td>0.20</td>
<td>------</td>
<td>0.12</td>
</tr>
<tr>
<td>Vanadium (V) %</td>
<td>------</td>
<td>0.06 max</td>
<td>0.0818</td>
</tr>
<tr>
<td>Cobalt (Co) %</td>
<td>------</td>
<td>0.05 max</td>
<td>------</td>
</tr>
<tr>
<td>Iron (Fe) %</td>
<td>The rest</td>
<td>The rest</td>
<td>The rest</td>
</tr>
</tbody>
</table>

### Table 4.2: Summary of high-temperature tensile strength tests on A572 Gr 50 steel coupons

<table>
<thead>
<tr>
<th>Temp. (°C)</th>
<th>( f_{y,0.2%} ) (MPa)</th>
<th>( f_{u} ) (MPa)</th>
<th>Rupture strain</th>
<th>Elongation (%)</th>
<th>Reduction in area (%)</th>
<th>( E_r / E_{20} )</th>
<th>( f_{y,7/f_{u,20}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>408.91</td>
<td>498.49</td>
<td>0.13989</td>
<td>5.73</td>
<td>70.91</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>200</td>
<td>399.81</td>
<td>489.64</td>
<td>0.08507</td>
<td>4.17</td>
<td>53.25</td>
<td>0.81</td>
<td>0.98</td>
</tr>
<tr>
<td>400</td>
<td>301.38</td>
<td>365.93</td>
<td>0.09696</td>
<td>4.82</td>
<td>67.13</td>
<td>0.59</td>
<td>0.74</td>
</tr>
<tr>
<td>500</td>
<td>282.17</td>
<td>311.13</td>
<td>0.08622</td>
<td>4.82</td>
<td>59.31</td>
<td>0.54</td>
<td>0.69</td>
</tr>
<tr>
<td>600</td>
<td>224.91</td>
<td>232.30</td>
<td>0.09320</td>
<td>4.43</td>
<td>44.63</td>
<td>0.49</td>
<td>0.55</td>
</tr>
<tr>
<td>700</td>
<td>123.18</td>
<td>131.60</td>
<td>0.12272</td>
<td>6.25</td>
<td>59.20</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>800</td>
<td>53.93</td>
<td>62.40</td>
<td>0.21383</td>
<td>13.15</td>
<td>55.56</td>
<td>0.16</td>
<td>0.13</td>
</tr>
</tbody>
</table>
Table 4.3: Summary of high-temperature residual tensile strength tests on A572 Gr 50 steel after air cooling

<table>
<thead>
<tr>
<th>Temp. (°C)</th>
<th>( f_y, 0.2% ) (MPa)</th>
<th>( f_u ) (MPa)</th>
<th>Rupture strain</th>
<th>Elongation (%)</th>
<th>Reduction in area (%)</th>
<th>( E, \frac{E}{E_{20}} )</th>
<th>( f_y, \frac{f}{f_{y20}} )</th>
<th>( f_u, \frac{f}{f_{u20}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>452.53</td>
<td>534.66</td>
<td>0.16420</td>
<td>8.15</td>
<td>74.42</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>400</td>
<td>477.89</td>
<td>507.80</td>
<td>0.11044</td>
<td>5.60</td>
<td>68.55</td>
<td>1.20</td>
<td>1.06</td>
<td>0.95</td>
</tr>
<tr>
<td>500</td>
<td>465.92</td>
<td>522.26</td>
<td>0.13054</td>
<td>6.51</td>
<td>69.62</td>
<td>1.17</td>
<td>1.03</td>
<td>0.98</td>
</tr>
<tr>
<td>600</td>
<td>452.32</td>
<td>513.50</td>
<td>0.10985</td>
<td>5.73</td>
<td>72.48</td>
<td>1.32</td>
<td>1.00</td>
<td>0.96</td>
</tr>
<tr>
<td>700</td>
<td>325.54</td>
<td>449.11</td>
<td>0.11242</td>
<td>6.12</td>
<td>75.60</td>
<td>1.29</td>
<td>0.72</td>
<td>0.84</td>
</tr>
<tr>
<td>800</td>
<td>272.77</td>
<td>374.19</td>
<td>0.18524</td>
<td>9.25</td>
<td>77.42</td>
<td>1.26</td>
<td>0.60</td>
<td>0.70</td>
</tr>
<tr>
<td>1000</td>
<td>204.01</td>
<td>324.51</td>
<td>0.23198</td>
<td>11.33</td>
<td>78.25</td>
<td>1.13</td>
<td>0.45</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Table 4.4: Summary of high-temperature residual tensile strength tests on A572 Gr 50 steel after cooling through water quenching

<table>
<thead>
<tr>
<th>Temp. (°C)</th>
<th>( f_y, 0.2% ) (MPa)</th>
<th>( f_u ) (MPa)</th>
<th>Rupture strain</th>
<th>Elongation (%)</th>
<th>Reduction in area (%)</th>
<th>( E, \frac{E}{E_{20}} )</th>
<th>( f_y, \frac{f}{f_{y20}} )</th>
<th>( f_u, \frac{f}{f_{u20}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>486.47</td>
<td>562.52</td>
<td>0.12293</td>
<td>7.03</td>
<td>69.93</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>200</td>
<td>504.28</td>
<td>545.46</td>
<td>0.09465</td>
<td>4.69</td>
<td>72.99</td>
<td>1.18</td>
<td>1.04</td>
<td>0.97</td>
</tr>
<tr>
<td>400</td>
<td>507.52</td>
<td>550.08</td>
<td>0.12046</td>
<td>5.99</td>
<td>72.44</td>
<td>1.18</td>
<td>1.04</td>
<td>0.98</td>
</tr>
<tr>
<td>500</td>
<td>465.74</td>
<td>521.90</td>
<td>0.12970</td>
<td>6.51</td>
<td>73.95</td>
<td>1.25</td>
<td>0.96</td>
<td>0.93</td>
</tr>
<tr>
<td>600</td>
<td>460.79</td>
<td>527.77</td>
<td>0.14966</td>
<td>3.39</td>
<td>73.83</td>
<td>1.31</td>
<td>0.95</td>
<td>0.94</td>
</tr>
<tr>
<td>700</td>
<td>414.05</td>
<td>522.51</td>
<td>0.12801</td>
<td>6.51</td>
<td>73.20</td>
<td>1.06</td>
<td>0.85</td>
<td>0.93</td>
</tr>
<tr>
<td>800</td>
<td>360.62</td>
<td>536.90</td>
<td>0.07809</td>
<td>3.91</td>
<td>72.46</td>
<td>0.54</td>
<td>0.74</td>
<td>0.95</td>
</tr>
<tr>
<td>1000</td>
<td>447.24</td>
<td>568.55</td>
<td>0.06741</td>
<td>3.39</td>
<td>73.23</td>
<td>1.03</td>
<td>0.92</td>
<td>1.01</td>
</tr>
</tbody>
</table>
Table 4.5: Selected temperatures and stress levels for creep tests

<table>
<thead>
<tr>
<th>Stress (MPa)</th>
<th>Stress level (%)</th>
<th>Temperatures (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>11</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>700</td>
</tr>
<tr>
<td>115</td>
<td>25</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>600</td>
</tr>
<tr>
<td>182</td>
<td>40</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>600</td>
</tr>
<tr>
<td>250</td>
<td>55</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>500</td>
</tr>
</tbody>
</table>

Table 4.6: Critical temperatures for A572 steel at different stress levels

<table>
<thead>
<tr>
<th>Stress level (%)</th>
<th>Critical temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>750</td>
</tr>
<tr>
<td>25</td>
<td>650</td>
</tr>
<tr>
<td>40</td>
<td>600</td>
</tr>
<tr>
<td>55</td>
<td>550</td>
</tr>
</tbody>
</table>
Figure 4.1: Coupon for high-temperature tensile strength tests

Figure 4.2: Test set-up for high-temperature tensile strength tests

All dimensions are in millimeters
Figure 4.3: Test set-up for residual tensile strength tests

Figure 4.4: Schematics of test set-up for high-temperature creep tests
Figure 4.5: Custom built creep test equipment for high-temperature creep tests

Figure 4.6: Coupon placement in the furnace for creep test
Figure 4.7: High-temperature stress-strain response of A572 Gr50 steel.
Figure 4.8: Comparison of measured strength and stiffness reduction factors of A572 steel with predicted values from codes.
Figure 4.9: Residual stress-strain response of A572 Gr50 steel after heating and air cooling

Figure 4.10: Residual stress-strain response of A572 Gr50 steel after heating and quenching with water
Figure 4.11: Residual strength of A572 Gr.50 steel after exposure to elevated temperature
Figure 4.12: Creep deformations at various temperatures and stress levels
Figure 4.12 (cont’d)

(c) Creep at 40% stress level

(d) Creep at 55% stress level
Figure 4.13: Failure patterns resulting from creep at various temperatures and stress levels

(a) Stress level = 11%  (b) Stress level = 25%  (c) Stress level = 40%  (d) Stress level = 55%

Figure 4.14: Creep strain at 650°C at various stress levels
Figure 4.15: Total creep strain as a function of temperature
Figure 4.16: Comparison of strength reduction factors from proposed equations with test data
Figure 4.17: Comparison of elastic modulus reduction factors from proposed equations with test data
Figure 4.18: Comparison of residual strength reduction factors from proposed equations with residual strength test data for coupons with heating and air cooling.

(a) Yield strength

(b) Ultimate strength
Figure 4.19: Comparison of residual strength reduction factors from proposed equations with residual strength test data for coupons with heating and water quenching.
4.5 Summary

Material properties tests were undertaken to evaluate the high-temperature mechanical properties of A572 Gr. 50 steel. These tests comprised of high-temperature tensile strength, residual strength, and creep tests. The residual strength tests are carried out after cooling the heated specimens either through, air cooling or water quenching. Data generated in these tests are utilized to generate high-temperature stress-strain, residual stress-strain, and high-temperature creep response of A572 steel. These results are also utilized to propose reduction factor relations for high-temperature strength, elastic modules, residual strength, and guidance on critical temperature of creep. The proposed relations can be used for undertaking fire resistance analysis of steel structures, including bridge girders, and to evaluate residual strength of fire exposed structural members.

Results from mechanical properties tests show that temperature induced strength reduction factors for A572 steel is compatible to ASCE and EC3 provisions, but with slight variations. However, current ASCE and EC3 provisions can lead to overestimation of elastic modulus of A572 steel. Results from residual strength tests indicate that A572 steel recovers almost 100% of its room temperature yield strength when heated to temperature up to 600°C, while it loses up to 40% of its strength when heated to 800°C and then air cooled to room temperature. However, when the same steel is heated to 800°C and then cooled through quenching in water, a 75% of its room temperature yield strength is recovered.

Results from creep tests show that temperature has significant influence on the level of creep deformation in steel, especially when the temperature exceeds 500°C. Furthermore, the extent of creep deformation at a given temperature increases with stress
level and the effect can be substantial when the stress level exceeds 50% of room temperature yield stress.
CHAPTER FIVE

5. NUMERICAL MODEL

5.1 General

Evaluating fire resistance of structural members through fire tests is quite expensive, complex, time consuming and requires sophisticated test facilities. In fire tests, only limited number of parameters can be studied and interdependency of parameters cannot be traced. The alternative to overcome many of the limitations in fire tests is to apply numerical modeling approach for evaluating fire response of structures. For this purpose, the development of a finite element model to undertake fire resistance analysis on steel bridge girders is presented in this chapter. Also, the high-temperature material properties and different material models adopted in fire resistance analysis are described. The validity of the finite element model in tracing thermal and structural response is established by comparing predictions from the analysis with results from fire tests undertaken as part of this thesis and also tests undertaken by other researchers.

5.2 Development of Finite Element Model

Undertaking detailed fire response analysis of structural members requires the use of a finite element based computer software to develop relevant numerical models for tracing thermal and structural response. In fire resistance analysis use of relevant high-temperature thermal and mechanical material properties and suitable material models is
highly important to capture realistic response of structural members under fire condition.
Details of various aspects on fire resistance analysis are presented in this chapter.

5.2.1 Selection of finite element program

There are several computational models that can be applied for simulating response of structural members under fire conditions. These models are based on finite element formulation and include, ABAQUS, ANSYS, VULCAN, and SAFIR. For the current study, a general-purpose finite element program, ANSYS (ANSYS Inc., 2014) software, was chosen to carry out numerical studies because of its diverse capabilities. This is because ANSYS has the ability to efficiently capture highly sophisticated material, and geometrical nonlinearities in coupled and uncoupled thermo-mechanical problems. ANSYS enables the user to specify temperature-dependent thermal and mechanical properties of almost any material. Also, ANSYS provides a wide range of high-temperature creep models, in addition to any user-specified high-temperature creep models. The geometry of the model can be created using ANSYS scripting language (APDL-ANSYS Parametric Design Language) in which different geometric configurations and material models can be defined as parameters. Thus, APDL can be used effectively for creating new models and automating parametric studies. Further, ANSYS contains a rich library of diverse categories of elements; such as solid, shell, and contact elements that are well suited discretization of different types of structural configurations.

Fire resistance analysis was carried out by incorporating all significant parameters, such as material and geometrical nonlinearities, end restraints, fire scenarios, and loading scenarios. Details on the finite element analysis are provided in the following
sections.

5.2.2 General approach

In finite element approach, fire resistance analysis is generally carried out through two stages of analysis, namely thermal and structural analysis. The thermal and structural analysis can be coupled or uncoupled. The thermal analysis generates cross-sectional temperature distribution in a structural member as a function of fire exposure time. The output from the thermal analysis, nodal temperatures, is then applied as an input thermal-body-load to the structural analysis and a transient stress analysis is carried out. The structural analysis generates deformations and stresses resulting due to fire exposure. A flowchart illustrating steps associated with fire resistance analysis of typical structural members is shown in Figure 5.1. The following assumptions are adopted in undertaking fire resistance analysis:

- Fire temperature and heating rate are independent of both thermal and structural response of a structural member.

- Thermal analysis is independent of the structural analysis. The progression of cross-sectional temperature in a structural member (girder) depends only on density, thermal conductivity and specific heat of steel, concrete and insulation. On the other hand, the structural response of the members depends on the mechanical properties, modulus of elasticity, stress-strain relationships of steel and concrete, in addition to other factors such as loading conditions and restraint scenarios.

- Temperature distribution is uniform along the girder span length: The entire length of the girder is assumed to be subjected to the same fire exposure
conditions and the cross-sectional geometry remains the same throughout the member.

The fire scenarios that are typically applied in fire resistance analysis can be grouped under two categories; namely, standard and realistic (design) fire scenarios. In standard fire scenario, fire temperature continues to increase without any cooling phase. Examples of standard fires are hydrocarbon fire, external fire, ASTM E119 or ISO 834 standard fire exposures. In realistic fire scenario fire temperature increase with time in growth phase and then gradually decrease with time in the decay or cooling phase. In this case (realistic fire), a decay phase follows after reaching a maximum fire temperature in growth phase. Unlike in standard fire, where temperature is expressed as a function of time only, the growth and decay phases of realistic fires in bridges are dependent on the extent of fuel and ventilation. Examples of design fires are the Swedish fire curves and the parametric fire curves specified in the Eurocode (EC1, 2002).

Generally, the rise of fire temperature is dependent on fuel and ventilation. Computational fluid dynamics (CFD) models are required to compute the evolution of fire in a certain enclosure. Since this kind of analysis is beyond the scope of this study, fire characteristics are reduced to time-temperature relationships and are taken from the specified standard and design fire curves in codes. The fire resistance analysis, with its thermal and structural phases, is carried out via ANSYS finite element program using the following general procedure:

- Selection of a given fire scenario (time-temperature curve), girder section, loading, etc.

- Different structural components of composite bridge girder, comprising of steel
girder, reinforced concrete slab, shear studs and stiffeners are discretized into elements.

- Room temperature structural response: Static structural analysis is performed on the entire structural model of the composite girder to obtain the room-temperature structural response (deformations and stresses).

- The total fire exposure time is divided into a number of time steps.

- Thermal analysis: the cross-sectional temperature distribution of the composite girder is generated.

- Structural analysis: During the first time step, the temperature distribution of the composite girder obtained from thermal analysis is applied as a thermal-body-load on the room-temperature deformed model of the girder and a stress analysis is carried out. For the subsequent time steps, the temperature distribution of girder cross-section generated from thermal analysis is applied as a thermal-body-load on the deformed model from preceding time step and a stress analysis is carried out.

- The fire resistance analysis is carried out at various time increments till failure occurs in the girder. In each time step, various response parameters from thermal and structural analysis are utilized to evaluate the state of the bridge girder under different failure limit states. At any time step, the analysis terminates if failure is attained, otherwise, the analysis continue to next time step (thermal analysis).

More details about the thermal and structural sub models are presented in the following sections.
5.2.3 Thermal analysis

Since the temperature along the girder span length is assumed to be uniform, heat transfer analysis in the girder is reduced to a two-dimensional problem. Therefore, the governing partial differential heat transfer equation within the girder cross section can be written as:

\[ \rho c(T) \frac{dT}{dt} = \nabla \cdot (k(T) \nabla T) \] .................................................................[5.1]

where, \( k \) = conductivity matrix, \( \rho c \) = heat capacity, \( \rho \) = density, \( c \) = specific heat, \( T \) = temperature, \( t \) = time, and \( \nabla = \) is the spatial gradient operator.

At the fire-girder interface, heat transfer from fire source to girder surface is through radiation and convection. The heat flux on the boundary due to convection and radiation can be given by the following equation:

\[ q_b = (h_{con} + h_{rad}) (T - T_f) \] .................................................................................[5.2]

where, \( h_{rad} \) and \( h_{con} \) are the radiative and convective heat transfer coefficients, and are defined as:

\[ h_{rad} = 4 \sigma \varepsilon (T^2 + T_f^2) (T + T_f) \] .................................................................................[5.3]

\( T_f \) = temperature of the atmosphere surrounding the boundary (fire temperature),

\( h_{con} = 50 \text{ W/m}^2\text{. K} \) for hydrocarbon fire and 25 W/m². K for external, ASTM E119, and ISO 834 fires as recommended in Eurocode (EC1, 2002)

\( \sigma = \text{Stefan-Boltzman constant} = 5.67 \times 10^{-8} \text{ (W/m}^2\text{.^oK}^4) \), and

\( \varepsilon = \text{emissivity factor for steel and it is related to the “visibility” of the exposed surface of structural member to the fire.} \)

Fire is assumed to be emanating from a point source (infinitesimal sphere) perfect black
body radiating heat equally in all spatial directions. To account for heat loss to surrounding environment as fire travels from its source to the girder; it is assumed that only 70% of the radiant heat from fire source reaches the girder surface. Therefore, a steel emissivity factor of 0.7 specified in Eurocode (EC3, 2005) is assumed in the analysis.

The heat flux and temperature gradient are related through Fourier’s law of heat conduction as:

\[ q = -k \nabla T \] \[ \text{[5.4]} \]

According to Fourier’s law, the governing heat transfer equation on any boundary of the girder cross-section can be expressed as:

\[ k \left( \frac{\partial T}{\partial y} n_y + \frac{\partial T}{\partial z} n_z \right) = -q_b \] \[ \text{[5.5]} \]

where, \( n_y \) and \( n_z \) = components of the vector normal to the boundary in the plane of the cross-section. The right hand side of Eq. [5.5] depends on the type of imposed boundary condition. As the girder is exposed to fire from three sides, two types of boundary equations are to be considered in thermal analysis, namely:

- On fire exposed boundaries where the heat flux is governed by the following equation:
  \[ q_b = -h_f(T - T_f) \] \[ \text{[5.6]} \]

- On unexposed boundary (top surface of the girder) where the heat flux equation is given by:
  \[ q_b = -h_0(T - T_0) \] \[ \text{[5.7]} \]

where, \( h_f \) and \( h_0 \) = heat transfer coefficient on fire side and the cold side, respectively, and \( T_f \) and \( T_0 \) = temperature at fire exposed and cold side, respectively.
The nodal temperatures of any element are related by the appropriate shape functions matrix \((N)\) to arrive at the temperature in the element

\[
T = \{N\}^T T_e .................................................................[5.8]
\]

Then the heat transfer equation (Eq.[5.1]) subjected to appropriate boundary conditions (Eq.[5.5]) can be discretized as (Cook et al., 2002):

\[
C_e^t T_e + K_e^t T_e = Q_e .................................................................[5.9]
\]

where, \(C_e^t\) is the specific heat matrix, \(K_e^t\) is the thermal “stiffness” matrix, and is the sum of conductivity and convection matrices. \(Q_e\) is the applied nodal thermal load and is composed of the convective and radiative heat fluxes. \(T_e\) are the nodal temperatures.

The heat transfer analysis of the selected composite steel-concrete girder is carried out by discretizing the girder with two types of elements available in ANSYS, namely SOLID70 and SURF152. For discretization of the girder, slab, and the stiffeners, SOLID70 elements are used. SOLID70 is a 3-D element with three-dimensional thermal conduction capability and has eight nodes, with a single degree of freedom, namely temperature, at each node. This element is applicable to three-dimensional, steady–state or transient thermal analysis. The temperature within SOLID70 element is interpolated from the nodal degrees of freedom \((T_e = T_{i,j,k,l,m,n,o,p})\) using the following isoparametric function:

\[
T = \frac{1}{8} \left[ T_i (1-s)(1-t)(1-r) + T_j (1+s)(1-t)(1-r) + T_k (1+s)(1+t)(1-r) \right.
\]

\[
+ T_l (1-s)(1+t)(1-r) + T_m (1-s)(1-t)(1+r) + T_n (1+s)(1-t)(1+r) \left. + T_o (1+s)(1+r)(1+r) + T_p (1-s)(1+t)(1+r) \right] \quad \ldots \ldots \ldots \ldots \ldots \ldots [5.10]
\]

where, \(s, t\) and \(r\) are the isoparametric locus of a point in the element domain. In order to carry out the numerical integrations, 2x2x2 integration points are used in SOLID70 elements.
The SURF152 element is generally used for various load and surface effect applications. The geometry, nodes locations, and the coordinate system for SOLID70 and SURF152 elements are shown in Figure 5.2. In the thermal analysis, SURF152 elements is overlaid onto the external surface of SOLID70 elements to simulate the effect of both thermal radiation and heat convection from ambient air to the exposed boundaries of the bridge girder. The ambient (bulk) temperature on the nodes of SURF152 element is assumed to be equal to either the fire temperature \( T_f \) in case the boundary is exposed to fire, or to room-temperature, in case the boundary is not exposed to fire. The discretization adopted for thermal model is shown in Figure 5.3.

The steel girder-slab assembly segment AB that is shown in Figure 5.3(a) is meshed with SOLID70 and SURF152 elements. The 3-D mesh of segment AB of the girder is shown in Figure 5.3(b). In the thermal analysis, a convection coefficient of \( \alpha_c = 50 \text{ W/(m}^2\text{C)} \) is used under hydrocarbon fire, while \( \alpha_c = 25 \text{ W/(m}^2\text{C)} \) is used under both external and ISO 834 fires and this is based on (EC1,2002) recommendations. Stefan-Boltzmann radiation constant of \( 5.67 \times 10^{-8} \text{ W/(m}^2\text{C)} \) was applied in the thermal analysis. The temperature (T) generated via finite element analysis are averaged at every time step for each component (flange, web or slab portion) of the steel girder-concrete slab composite section. This is done by taking the arithmetic mean of the temperatures at several points for each of these components as shown in Figure 5.3(c).

5.2.4 Structural analysis

The structural analysis is carried out in ANSYS based on the principle of virtual work. According to this principle, any virtual change of the internal strain energy must be balanced by a change in the external work due to the applied loads.
\[ \delta U = \delta V \] .................................................................[5.11]

where; \( U \) is the strain energy and \( V \) is the external work. Variation in strain energy (\( \delta U \)) can be evaluated as:

\[ \delta U = \int_{\text{vol}} \{ \delta \varepsilon \} \sigma \, \text{dvol} \] .................................................................[5.12]

For steel structural members subjected to fire conditions, the strain vector (\( \varepsilon \)) is the sum of thermal (\( \varepsilon_{th} \)), mechanical (\( \varepsilon_{m} \)), and creep strains (\( \varepsilon_{cr} \)):

\[ \varepsilon = \varepsilon_{m} + \varepsilon_{th} + \varepsilon_{cr} \] .................................................................[5.13]

Variation of external work (\( \delta V \)) due to the applied nodal forces (\( F_{e}^{n} \)) can be computed by assuming a variation of nodal displacement \( \{ \delta u \} \) as:

\[ \delta V = \{ \delta u \}^{T} \{ F_{e}^{n} \} \] .................................................................[5.14]

The nodal displacements (\( u_{e} \)) of the finite elements are related to the nodal displacement field through shape functions matrix (\( N \)) as follows:

\[ u_{e} = \{ N \}^{T} \cdot u \] .................................................................[5.15]

Then the virtual work equation (Eq. [5.11]) can be rewritten in matrix form as:

\[ K_{e} u_{e} - F_{e}^{th} = F_{e}^{n} \] .................................................................[5.16]

where; \( K_{e} \) is the element stiffness matrix, and \( F_{e}^{th} \) is the element thermal load vector.

More details on the governing equations for structural analysis are presented in Appendix B.

In the structural analysis different components of composite girder, comprising of steel girder, reinforced concrete slab, steel-concrete interface, shear studs, support (restraint) conditions, and stiffeners were taken into consideration. These components
were modeled using suitable elements from ANSYS library. The bottom flange, web, top flange and stiffeners of the steel girder were modeled with SHELL181 elements, and the concrete slab was modeled with SOLID185 element.

To account for composite action between the concrete slab and the top flange of the steel girder, 3-D nonlinear surface-to-surface contact elements (CONTA174 and TARGE170) are used. The contact pair is to model the contact between two boundaries, one of the boundaries represents contact or deformable surface (CONTA174) and the other one represent rigid surface taken as a target surface (TARGE170). The contact pair can be full bonded to simulate the full interaction between the concrete slab and steel girder or it can be unbonded (standard) to account for the slip that occurs between the concrete slab and steel girder in case of partial interaction. Partial interaction occurs when there is not enough shear studs to prevent the slip that might occurs due to normal force between concrete slab and steel girder, which results in reduction in the composite section capacity. To resist the normal force between the concrete and steel girder, COMBIN39 nonlinear spring elements were used to model the shear studs. High-temperature load-slip relations for the shear stud (COMBIN39) are defined in the material model section.

To model the steel reinforcement in the concrete slab, LINK8 elements were used. Perfect bond was assumed between the reinforcing bars and the surrounding concrete. The restraint condition at the end of the girder was modeled using COMBIN14 spring elements. More details on end restraint conditions are discussed in the following section. Full description regarding degree of freedom, geometry, nodes locations and coordinate systems for the elements, that is used to discretize the structural model of steel-concrete
bridge girder, is given below:

- **SHELL181 Element**
  This element has four nodes with six degrees of freedom per node; three translations in x, y, and z directions and three rotations about x, y, and z-axes. This element can capture buckling of flange and web and also lateral torsional buckling of the girder and therefore is well-suited for large rotation, large strain and nonlinear problems. The geometry, node locations, and the coordinate system for this element is shown in Figure 5.4.

- **SOLID185 Element**
  This element is defined by eight nodes having three degrees of freedom at each node: translation in the nodal x, y, and z directions. This element has plasticity, hyperelasticity, stress stiffening, creep, large deflection, and large strain and material damage capabilities. The geometry, node locations, and the coordinate system for this element is shown in Figure 5.4.

- **LINK8 Element**
  This 3-D spar element is a uniaxial tension-compression element with three degrees of freedom at each node: translation of the nodes in x, y, z-directions. No bending of the element is considered. Plasticity, creep, swelling, stress stiffening, and large deflection capabilities are included. The geometry, node locations, and the coordinate system for this element is shown in Figure 5.4.

- **CONTA174 and TARGE170 Element**
  This 3-D nonlinear surface-to-surface (contact-pair) element was used to model the nonlinear behavior of the interface surface between concrete slab and steel girder. This element has three degrees of freedom at each node: translation of
the nodes in x, y, z-directions. The contact pair consists of the contact between two boundaries, one of the boundaries represents contact, slid, and deformable surface taken as a contact surface (CONTA174) and the other represents rigid surface taken as a target surface (TARGE170). The geometry, node locations, and the coordinate system for this element is shown in Figure 5.4.

- **COMBIN39 Element**
  This element is a unidirectional element with nonlinear generalized force-deflection capability that can be used in any analysis. The element has longitudinal or torsional capability in one, two, or three dimensional applications. The longitudinal option is a uniaxial tension-compression element with up to three degrees of freedom at each node: translation in the nodal x, y, and z directions. The torsional option is a purely rotational element with three degrees of freedom at each node: rotation about the nodal x, y, and z axes. The geometry, node locations, and the coordinate system for this element is shown in Figure 5.4.

- **COMBIN14 Element**
  This element has longitudinal or torsional capability in 1-D, 2-D, or 3-D applications. The longitudinal spring-damper option is a uniaxial tension-compression element with up to three degrees of freedom at each node: translation in the nodal x, y, and z directions. The torsional spring-damper option is a purely rotational element with three degrees of freedom at each node: rotations about the nodal x, y, and z axes. The geometry, node locations, and the coordinate system for this element is shown in Figure 5.4.
The 3-D discretization of structural model of the selected typical steel bridge girder with and without stiffeners is shown in Figure 5.5

**5.2.5 Modeling end restraints**

Steel bridges can be designed as statically determinate or indeterminate structures in single or continuous spans. Restraint conditions (axial and rotational) on the boundary of the steel bridge girders arise due to thermal expansion of the girder with respect to adjacent structural members or the continuity of the span. The axial restraint at the support is a function of a number of factors including the adjacent girder span, stiffness of adjacent span, and the continuity of the girders. Rotational restraint that occurs in the plane of bending is also dependent on many factors including the continuity of the girder (to adjacent spans), and the extent of composite action that develops between concrete slab and steel girder. The computation of these restraint stiffnesses (axial and rotational) at room temperature can be done using conventional methods of analysis for indeterminate structures. In the structural analysis, the axial and rotational restraints on the ends of the composite steel-concrete girder are assumed to be elastic, constant, and symmetric as shown in the illustration in Figure 5.6.

The location of the axial restraint force can vary depending on the boundary condition of the bridge girders. The axial restraint stiffness may not pass through the centroidal axis of the cross section, but may be eccentric by a distance “y” from the center of geometry (centroid) of the section. In most of the previous studies on restraint structural members under fire condition, the location of axial restraint stiffness was always assumed to be in the centroid of the section.

In order to account for the above factors, axial and rotational restraints are
modeled using axial and rotational springs on each end of the girder. The linear springs were modeled using COMBIN14 spring element available in ANSYS. It is assumed that these springs have constant elastic stiffness properties.

Using the configuration shown in Figure 5.7, the axial restraint stiffness ($K_a$), and rotational restraint stiffness ($K_r$), are obtained as:

$$K_a = \frac{E_s A_s}{L} + \frac{E_c A_c}{L}, F = K_a u \quad [5.17]$$

$$K_r = \frac{E_s I_s}{L} + \frac{E_c I_c}{L}, M = K_r \theta \quad [5.18]$$

where; ($E_s$, $A_s$, $I_s$) and ($E_c$, $A_c$, $I_c$) are (elastic modulus, cross-sectional area, moment of inertia) of steel and concrete respectively. $L$ is the girder span, while; $F$ and $M$ are the restraint forces. By changing the values of $K_a$ and $K_r$, different degrees of axial and rotational restraint stiffness can be imposed on the girder. In order to model rotational and axial restraints, the axial and rotational spring elements must act on a rigid diaphragm. The node at which the spring element located is considered as a master node, while all the other nodes at the restraint edge of the girder are followed the master node.

5.3 Selection of Material Models

There are significant variations among different constitutive models for high-temperature properties of structural steel as presented in Chapter 2. These variations can influence the predicted response of the steel bridge girders under fire conditions. Therefore, it is critical to decide on the proper material models for use in finite element models.

5.3.1 High-temperature thermal properties

Temperature distribution in steel section depends on the fire exposure scenario
and thermal properties of steel and insulation. These properties are thermal conductivity and specific heat and they vary as a function of temperature. High-temperature thermal properties of steel and concrete specified in Eurocodes (EC3, 2005) and (EC2, 2004) are used in numerical analysis. The temperature-dependent thermal property relationships of steel and concrete are presented in Tables 5.1 and 5.2. For insulated section, the thermal properties of the insulation (thermal conductivity and specific heat) are also varying as a function of temperature.

5.3.2 High-temperature mechanical properties

The mechanical properties of steel and concrete that are critical for fire resistance analysis are stress-strain relationships and modulus of elasticity which vary with temperature. The following material models were used in analysis:

- **Steel**
  To simulate the behavior of steel in compression and tension, multilinear stress-strain relationships with Von-Mises plasticity yielding criterion and isotropic hardening plasticity model were used in fire resistance analysis of steel bridge girders. However, for residual strength analysis, the same stress-strain relationships were used but with kinematic hardening plasticity model. This is due to the fact that kinematic hardening rule is more realistic for residual strength problems that comprise of different stages of loading. The Eurocode3 stress-strain model in Figure 5.8 was used in analysis. The significant points on Eurocode3 stress-strain curves at elevated temperatures are given in Tables 5.3 and 5.4. The nominal stress-strain curves were converted into true stress-strain curves using the following relations:
\[ \sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \quad \text{and} \quad \varepsilon_{\text{true}} = \ln(1 + \varepsilon_{\text{nom}}) \] ................................. [5.19]

where; \( \sigma_{\text{true}}, \varepsilon_{\text{true}} \) represent true stress and strain, while \( \sigma_{\text{nom}}, \varepsilon_{\text{nom}} \) represent nominal stress and strain respectively.

- **Concrete**
  Eurocode2 high-temperature stress-strain model that is illustrated in Figure 5.9 was used to simulate the behavior of concrete slab in compression. The mechanical properties along with constitutive relationships and strength reduction factors specified in Eurocode (EC2, 2004) used to generate nominal stress-strain-temperature relationships. The constitutive relationships and the reduction factors for concrete are presented in Tables 5.5 and 5.6. The nominal stress-strain curves were converted into true stress-strain curves.

- **Shear stud**
  To simulate the behavior of shear studs, the nonlinear high-temperature load-slip \( (P-\lambda) \) relationship shown in Figure 5.10 was used in analysis (Huang et al., 1999).

\[ P / P_u = A(1 - e^{-B\lambda}) \quad \text{and} \quad P_u = f_u(\pi d^2 / 4) \] ................................. [5.20]

where; \( A \) and \( B \) are high-temperature parameters, \( d \) is the shear stud diameter, and \( f_u \) is the ultimate shear strength of the shear stud. The high-temperature parameters \( A \) and \( B \) are given in Table 5.7.

- **Fire insulation**
  Since the insulation material has significantly low strength and stiffness, the strength contribution from the insulation is neglected.
5.3.3 Failure criteria

The failure criteria adopted for structural analysis in ANSYS program is the one based on attaining ultimate plastic strain in steel. Accordingly, the “failure” of the girder is said to occur when the ultimate plastic strain of steel in any of the girder plates exceeds 20% in 20-1000°C temperatures range (ANSYS Inc, 2014). When the strain in steel reaches this limiting value, the governing finite element equations do not converge leading to non-convergence of the solution.

Another convergence criterion for structural analysis in ANSYS is force and moment. Force and moment convergence are said to be achieved if the error between successive equilibrium iterations is less than 0.1% (ANSYS Inc, 2014). Similarly, in thermal analysis the temperature convergence is assumed to be reached when the temperature difference at each node between successive equilibrium iterations is less than 0.1 °C.

In some analysis cases, the ultimate plastic strain can be exceeded due to stress concentration problems or the numerical solutions do not converge due to large loading steps. In these cases, relaying on ANSYS criteria only to define failure might be misleading. Therefore, the overall structural response (mid-span deflection and web-out-of-plane displacement) of structural member (girder) is to be traced. Hence, in numerical analysis, deflection limit (L/30) is used to define failure (fire resistance) under fire condition (Wainman and Kirby, 1989). However, there is no out-of-plane displacement limit specified for web under fire condition.

5.4 Model Validation

The above developed finite element model is validated by comparing predicted
response parameters from the analysis with those measured in fire tests on a typical steel beam-concrete slab assembly and also on composite steel-concrete girders. The validation process covered both thermal and structural analysis.

5.4.1 Steel beam-concrete slab assembly taken from literature

There is lack of fire test data on the steel bridge girders under fire conditions prior to conducting fire tests that are presented in Chapter 3. Therefore, initial validation of the above developed ANSYS model is carried out on a steel beam-concrete slab assembly typical to that in buildings, tested by British Steel Corporation under ISO 834 fire exposure (Wainman and Kirby, 1989). The uninsulated steel beam-concrete slab assembly (4.5 m span), together with sectional dimensions is shown in Figure 5.11. This composite assembly has steel yield strength of 255 MPa and a concrete compressive strength of 30 MPa.

This steel beam-concrete slab assembly is discretized using ANSYS elements as described above for both thermal and structural analysis. In the analysis, the temperature dependent thermal and mechanical properties of steel and concrete are assumed to follow as that of EC2 and EC3 provisions.

Figure 5.12(a) shows a comparison of predicted steel temperatures (by the finite element model) with that measured in the fire test. It can be seen that the temperature in bottom flange, web, and top flange increased with increasing fire exposure. However, the top flange of the beam experienced much lower temperatures as compared to bottom flange. This is due to the “heat-sink” effect of concrete slab that dissipate the temperature in the top flange because of lower thermal conductivity and higher thermal capacity of concrete as compared to steel. The web temperatures are slightly higher than that in the
bottom flange and this is due to the fact that thickness of the web is much lower than that of the flanges. Overall, predicted temperatures from analysis compare well with measured data from the test. The slight differences can be attributed to variation of the heat transfer parameters, such as emissivity and convection coefficients, used in the analysis as compared to actual values present during fire test (in the furnace).

The comparison of mid-span deflections predicted by ANSYS model and those measured in the test is shown in Figure 5.12(b). It can be seen that the mid-span deflection gradually increases with time at the early stages of fire exposure (up to 13 minutes). These initial deflections are mainly due to high-temperature gradients that develop across the top and bottom flanges of the steel section and the slight reduction in elastic modulus of steel resulting from increased temperatures in steel. After 13 minutes, the rate of deflection increases slightly due to spread of plasticity that result from faster strength and stiffness degradation of steel as a result of higher temperatures. At about 21 minutes, bottom flange and web temperatures exceed 600°C and this leads to rapid increase in mid-span deflection due to the formation of plastic hinge at the mid-span section. The failure of the beam-slab assembly occurs at 23 minutes when the mid-span deflection exceeds the deflection limit (L/30) that is defined as limiting criteria under fire conditions.

Overall, predictions from the ANSYS match well with the reported test data. The slight differences in deflection predictions can be attributed to minor variations in idealization adopted in the analysis, such as stress-strain relationship of steel and concrete. It can be seen that ANSYS model can predict the time to failure with a good acceptability. For instance the predicted failure time was almost at the same time (23
minutes) from both ANSYS model and the test considering deflection limit state as the governing failure criterion.

5.4.2 Steel bridge girders from MSU fire resistance tests

To establish further validation of the above developed numerical model, response predictions from ANSYS is compared with data from fire tests presented in Chapter 3 on girders G1, G2, and G3. Girder G1 is a hot rolled section, while the other two tested girders (G2 and G3) are built-up plate girders. The girders are subjected to different load levels; evaluated as a percentage of shear and/or flexural capacity of the girder at room temperature. The loading is applied as a single point at the mid-span of the girders. The web slenderness, stiffener spacing, fire scenario, load level, time to failure for the three steel girders are illustrated in Table 5.8. During fire tests all three steel girders are exposed to ASTM E119 fire from three sides, with slightly higher heating rate in the first five minutes of fire exposure. A length of 3.0 m of the mid portion of the girder (3.658 m span) is directly exposed to fire inside the furnace.

The steel girders are fabricated using A572 Grade 50 steel, which is a high strength, low-alloy steel commonly used in highway bridge applications. All of the steel girders are designed to achieve full composite action with a 140 mm thick concrete slab. These composite girders have average steel yield strength of 480 MPa and a concrete compressive strength of 66 MPa (on the day of fire test, which is 210 days after casting of concrete slab).

The steel girders are disceritized using ANSYS elements that described previously for both thermal and structural models. The 3-D discretization of steel girders G1, G2, and G3 for both thermal and structural model is show in Figure 5.13. In the
analysis, the temperature dependent thermal and mechanical properties of steel and concrete are assumed to follow as that of EC2 and EC3 provisions. The validation process included comparison of both thermal and structural response predictions from the analysis with that measured during fire tests.

As part of thermal response validation, cross-sectional temperatures at selected points including; bottom flange, mid-depth of the web, top flange, and mid-depth of the slab are compared against corresponding temperatures measured in fire tests. The temperature validation is presented for girders G1 and G2 only since girder G3 experiences similar temperature rise as that of girder G2 due to similar cross-sectional geometry of these two girders. Figure 5.14 shows a comparison of predicted steel and concrete temperatures from the finite element model with that measured during fire tests for girders G1 and G2 respectively. It can be seen that the top flange of the steel girders experienced much lower temperatures as compared to that at bottom flange and this is mainly due to heat dissipation from top flange of girder to concrete slab, resulting from high thermal capacity of concrete. In the case of girder G1, the temperatures rise in web and bottom flange follow the same trend since flange and web thicknesses are almost same. However, in girder G2, the web temperature is slightly higher than at the bottom flange and this is due to the fact that thickness of web is much lower than that of flanges. The temperature in concrete at mid-depth of slab remains low, below 100°C, till failure of the girder.

Also, results plotted in Figure 5.14 show that the temperature rise in flanges and web in girder G2 is at a higher rate as compared to corresponding temperature in girder G1. This is due to thicker web in girder G1 as compared to girder G2. Overall, predicted
temperature from the analysis compare well with measured data from fire tests throughout the fire exposure duration. The slight differences between model predictions and test data can be attributed to variation of heat transfer parameters, such as emissivity and convection coefficients, used in the analysis as compared to actual conditions present in the furnace.

A comparison of mid-span deflections predicted by ANSYS with those measured in fire tests for girders G1, G2, and G3 are shown in Figure 5.15. It can be seen that the mid-span deflection gradually increases linearly with fire exposure time till about 10 min. These initial deflections are mainly due to thermal expansion resulting from high-temperature gradients that develop along the girder section. Between 10 to 25 min into fire exposure, the mid-span deflections start to increase at a slightly higher pace due to degradation in strength and elastic modulus of steel resulting from increased temperatures in the steel girder, which reaches to about 400°C. In the final stage of fire exposure (between 25-30 min), the rate of deflection increases due to spread of plasticity in bottom flange and web arising from faster strength and stiffness degradation of steel at high-temperature and also due to the effects of high-temperature creep. Finally, the steel girder experiences failure due to excessive deflections and loss of load bearing capacity.

Data presented in Figure 5.15 indicate a good comparison between predicted and measured mid-span deflections in all three girders throughout fire exposure duration. There is slight variation between measured and predicted deflections in girder G3, prior to failure of the girder. This can be attributed to shear web buckling that dominated the response of girder G3 in final stages of fire exposure. To illustrate this point, the deformed shapes of girders G1, G2, and G3, as obtained from ANSYS, at failure times, is
shown in Figure 5.16. As can be seen in this figure, girder G3 experienced significant web buckling due to lower initial shear capacity (at room temperature) and also due to faster degradation of shear capacity during fire exposure. The failure patterns shown in Figure 5.16 for girders G1, G2, and G3 are in good agreement with the observed failure patterns after fire tests.

Overall, predicted deflections, time to failure, and failure modes from ANSYS compare well with the reported data in fire tests.
Table 5.1: High-temperature thermal properties of steel (EC3, 2005)

<table>
<thead>
<tr>
<th>Property</th>
<th>Formula</th>
</tr>
</thead>
</table>
| Thermal strain            | $\varepsilon_{thS} = \begin{cases} 
1.2 \times 10^{-5} T + 0.4 \times 10^{-8} T^2 - 2.416 \times 10^{-4} & 20^\circ C \leq T \leq 750^\circ C \\
1.1 \times 10^{-2} & 750^\circ C < T \leq 860^\circ C \\
2 \times 10^{-5} T - 6.2 \times 10^{-3} & 860^\circ C < T \leq 1200^\circ C 
\end{cases}$ |
| Specific heat (J/kg K)    | $c_s = \begin{cases} 
425 + 7.73 \times 10^{-1} T - 1.69 \times 10^{-3} T^2 + 2.22 \times 10^{-6} T^3 & 20^\circ C \leq T < 600^\circ C \\
666 + \frac{13002}{738 - T} & 600^\circ C \leq T < 735^\circ C \\
545 + \frac{17820}{T - 731} & 735^\circ C \leq T < 900^\circ C \\
650 & 900^\circ C \leq T \leq 1200^\circ C 
\end{cases}$ |
| Thermal conductivity (W/m K) | $k_s = \begin{cases} 
54 - 3.33 \times 10^{-2} T & 20^\circ C \leq T < 800^\circ C \\
27.3 & 800^\circ C \leq T \leq 1200^\circ C 
\end{cases}$ |
Table 5.2: High-temperature thermal properties of concrete (EC2, 2004)

| Thermal conductivity (W/m K) | All types:  
|                            | Upper limit:  
|                            | \( k_c = 2 - 0.2451 \left( \frac{T}{100} \right) + 0.0107 \left( \frac{T}{100} \right)^2 \)  
|                            | for \( 20^\circ C \leq T \leq 1200^\circ C \)  
|                            | Lower limit:  
|                            | \( k_c = 1.36 - 0.136 \left( \frac{T}{100} \right) + 0.0057 \left( \frac{T}{100} \right)^2 \)  
|                            | for \( 20^\circ C \leq T \leq 1200^\circ C \)  

| Thermal capacity | Specific heat (J/kg C)  
|                 | \( c = 900, \) for \( 20^\circ C \leq T \leq 100^\circ C \)  
|                 | \( c = 900 + (T - 100), \) for \( 100^\circ C < T \leq 200^\circ C \)  
|                 | \( c = 1000 + (T - 200)/2, \) for \( 200^\circ C < T \leq 400^\circ C \)  
|                 | \( c = 1100, \) for \( 400^\circ C < T \leq 1200^\circ C \)  

| Density change (kg/m³) | \( \rho = \rho(20^\circ C) = \text{Reference density} \)  
|                        | for \( 20^\circ C \leq T \leq 115^\circ C \)  
|                        | \( \rho = \rho(20^\circ C) (1 - 0.02(T - 115)/85) \)  
|                        | for \( 115^\circ C < T \leq 200^\circ C \)  
|                        | \( \rho = \rho(20^\circ C) (0.98 - 0.03(T - 200)/200) \)  
|                        | for \( 200^\circ C < T \leq 400^\circ C \)  
|                        | \( \rho = \rho(20^\circ C) (0.95 - 0.07(T - 400)/800) \)  
|                        | for \( 400^\circ C < T \leq 1200^\circ C \)  

\( \text{Thermal Capacity} = \rho \times c \)
Table 5.3: Constitutive relations for high-temperature mechanical properties of steel (EC3, 2005)

### Stress-strain relationships

\[
\sigma_s = \begin{cases} 
\varepsilon_s E_{s,T} & \varepsilon_s \leq \varepsilon_{sp,T} \\
\frac{f_{sp,T} - c + b f}{f_{sy,T}} & \varepsilon_{sp,T} < \varepsilon_s < \varepsilon_{sy,T} \\
\frac{f_{sy,T} \left( 1 - \frac{\varepsilon_s - \varepsilon_{st,T}}{\varepsilon_{st,T} - \varepsilon_{su,T}} \right)}{0} & \varepsilon_{st,T} < \varepsilon_s \leq \varepsilon_{su,T} \\
\varepsilon_s > \varepsilon_{su,T} & \varepsilon_s > \varepsilon_{su,T} 
\end{cases}
\]

Parameters

\[
\varepsilon_{sp,T} = \frac{f_{sp,T}}{E_{s,T}} \quad \varepsilon_{sy,T} = 0.02 \quad \varepsilon_{ss,T} = 0.04 \quad \varepsilon_{st,T} = 0.15 \quad \varepsilon_{su,T} = 0.2
\]

Functions

\[
a^2 = (\varepsilon_{sy,T} - \varepsilon_{sp,T}) \left( \varepsilon_{sy,T} - \varepsilon_{sp,T} + \frac{c}{E_{s,T}} \right)
\]

\[
b^2 = c\varepsilon_{sy,T} - \varepsilon_{sp,T} E_{s,T} + c^2
\]

\[
c = \frac{(f_{sy,T} - f_{sp,T})^2}{(\varepsilon_{sy,T} - \varepsilon_{sp,T}) E_{s,T} - (f_{sy,T} - f_{sp,T})}
\]

Values of \( f_{sp,T}, f_{sy,T} \) and \( E_{s,T} \) can be obtained from Table 5.4

### Stress-strain relationships (Strain-hardening region)

For temperatures below 400°C

\[
\sigma_s = \begin{cases} 
50(f_{sT} - f_{sT} \varepsilon_s + 2f_{y,T} - f_{sT}) & 0.02 < \varepsilon_s \leq 0.04 \\
f_{sT} & 0.04 \leq \varepsilon_s \leq 0.15 \\
f_{sT} \left[ 1 - 20(\varepsilon_s - 0.15) \right] & 0.15 < \varepsilon_s < 0.2 \\
0 & \varepsilon_s \geq 0.2
\end{cases}
\]

\[
f_{u,T} = \begin{cases} 
1.25f_{sy,T} & T < 300°C \\
f_{sy,T} (2 - 0.0025T) & 300°C \leq T < 400°C \\
f_{sy,T} & T \geq 400°C
\end{cases}
\]
Table 5.4: Values for the main parameters of the stress-strain relations of steel at elevated temperatures (EC3, 2005)

<table>
<thead>
<tr>
<th>Steel temperature (°C)</th>
<th>$f_{sy,T}/f_{sy20}$</th>
<th>$f_{sp,T}/f_{sy20}$</th>
<th>$E_{sT}/E_{s20}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>200</td>
<td>1</td>
<td>0.807</td>
<td>0.9</td>
</tr>
<tr>
<td>300</td>
<td>1</td>
<td>0.613</td>
<td>0.8</td>
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<tr>
<td>400</td>
<td>1</td>
<td>0.42</td>
<td>0.7</td>
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<tr>
<td>500</td>
<td>0.78</td>
<td>0.36</td>
<td>0.6</td>
</tr>
<tr>
<td>600</td>
<td>0.47</td>
<td>0.18</td>
<td>0.31</td>
</tr>
<tr>
<td>700</td>
<td>0.23</td>
<td>0.075</td>
<td>0.13</td>
</tr>
<tr>
<td>800</td>
<td>0.11</td>
<td>0.05</td>
<td>0.09</td>
</tr>
<tr>
<td>900</td>
<td>0.06</td>
<td>0.0375</td>
<td>0.0675</td>
</tr>
<tr>
<td>1000</td>
<td>0.04</td>
<td>0.025</td>
<td>0.045</td>
</tr>
<tr>
<td>1100</td>
<td>0.02</td>
<td>0.0125</td>
<td>0.0225</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
Table 5.5: Constitutive relations for high-temperature properties of concrete (EC2, 2004)

| Stress-strain relationships | \( \sigma_c = \frac{3 \varepsilon f_{c,T}'}{\varepsilon_{c1,T}} \left( 2 + \left( \frac{\varepsilon}{\varepsilon_{c1,T}} \right)^3 \right) \), \( \varepsilon \leq \varepsilon_{cul,T} \nolinebreak[4]
| For \( \varepsilon_{c1(T)} < \varepsilon \leq \varepsilon_{cul(T)} \), the Eurocode permits the use of linear as well as nonlinear descending branch in the numerical analysis.
| For the parameters in this equation refer to Table 5.6 |
| Siliceous aggregates: |
| \( \varepsilon_{th} = -1.8 \times 10^{-4} + 9 \times 10^6 T + 2.3 \times 10^{-11} T^3 \) |
| for \( 20^\circ C \leq T \leq 700^\circ C \) |
| \( \varepsilon_{th} = 14 \times 10^{-3} \) |
| for \( 700^\circ C < T \leq 1200^\circ C \) |
| Calcareous aggregates: |
| \( \varepsilon_{th} = -1.2 \times 10^{-4} + 6 \times 10^6 T + 1.4 \times 10^{-11} T^3 \) |
| for \( 20^\circ C \leq T \leq 805^\circ C \) |
| \( \varepsilon_{th} = 12 \times 10^{-3} \) |
| for \( 805^\circ C < T \leq 1200^\circ C \) |
Table 5.6: Values for the main parameters of the stress-strain relations of normal strength concrete at elevated temperatures (EC2, 2004)

<table>
<thead>
<tr>
<th>Temp. (°C)</th>
<th>Siliceous Aggregate</th>
<th>Calcareous Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\frac{f_{c,T}}{f_{c20}}$</td>
<td>$\varepsilon_{c1,T}$</td>
</tr>
<tr>
<td>20</td>
<td>1</td>
<td>0.0025</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
<td>0.004</td>
</tr>
<tr>
<td>200</td>
<td>0.95</td>
<td>0.0055</td>
</tr>
<tr>
<td>300</td>
<td>0.85</td>
<td>0.007</td>
</tr>
<tr>
<td>400</td>
<td>0.75</td>
<td>0.01</td>
</tr>
<tr>
<td>500</td>
<td>0.6</td>
<td>0.015</td>
</tr>
<tr>
<td>600</td>
<td>0.45</td>
<td>0.025</td>
</tr>
<tr>
<td>700</td>
<td>0.3</td>
<td>0.025</td>
</tr>
<tr>
<td>800</td>
<td>0.15</td>
<td>0.025</td>
</tr>
<tr>
<td>900</td>
<td>0.08</td>
<td>0.025</td>
</tr>
<tr>
<td>1000</td>
<td>0.04</td>
<td>0.025</td>
</tr>
<tr>
<td>1100</td>
<td>0.01</td>
<td>0.025</td>
</tr>
<tr>
<td>1200</td>
<td>0</td>
<td>-</td>
</tr>
</tbody>
</table>
Table 5.7: Values for the main parameters of the load-slip relations of shear studs at elevated temperatures (Huang et al., 1999)

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Equation parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>≤ 100</td>
<td>1</td>
</tr>
<tr>
<td>200</td>
<td>1</td>
</tr>
<tr>
<td>300</td>
<td>0.9063</td>
</tr>
<tr>
<td>400</td>
<td>0.8567</td>
</tr>
<tr>
<td>500</td>
<td>0.5909</td>
</tr>
<tr>
<td>600</td>
<td>0.3911</td>
</tr>
<tr>
<td>700</td>
<td>0.1964</td>
</tr>
<tr>
<td>≥ 800</td>
<td>0.1472</td>
</tr>
</tbody>
</table>
Table 5.8: Summary of test parameters, loading conditions, and test results in girders G1, G2, and G3

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Girder G1</th>
<th>Girder G2</th>
<th>Girder G3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sectional geometry</strong></td>
<td>Girder shape</td>
<td>Rolled section W 24x62</td>
<td>Built-up plate girder</td>
<td>Built-up plate girder</td>
</tr>
<tr>
<td>Span (between supports), m</td>
<td></td>
<td>3.658</td>
<td>3.658</td>
<td>3.658</td>
</tr>
<tr>
<td>Web slenderness ratio (D/t_w)</td>
<td></td>
<td>52</td>
<td>123.3</td>
<td>123.3</td>
</tr>
<tr>
<td>Panel aspect ratio (a/D)</td>
<td></td>
<td>N/A</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Material strength</strong></td>
<td>Steel yield strength, MPa</td>
<td>480</td>
<td>480</td>
<td>480</td>
</tr>
<tr>
<td>Concrete compressive strength,</td>
<td></td>
<td>66</td>
<td>66</td>
<td>66</td>
</tr>
<tr>
<td>MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Applied load</strong></td>
<td>Applied load, kN</td>
<td>691</td>
<td>538</td>
<td>448</td>
</tr>
<tr>
<td>Applied load/flexural capacity</td>
<td></td>
<td>40%</td>
<td>40%</td>
<td>33%</td>
</tr>
<tr>
<td>Applied load/total shear capacity</td>
<td></td>
<td>27%</td>
<td>56%</td>
<td>56%</td>
</tr>
<tr>
<td>Fire exposure</td>
<td></td>
<td>ASTM E119</td>
<td>ASTM E119</td>
<td>ASTM E119</td>
</tr>
<tr>
<td><strong>Test results</strong></td>
<td>Time to initiation of web buckling, (min)</td>
<td>0</td>
<td>27</td>
<td>19</td>
</tr>
<tr>
<td>Time to failure at L/30, (min)</td>
<td></td>
<td>40</td>
<td>35</td>
<td>38</td>
</tr>
<tr>
<td>Failure limit state</td>
<td>Flexural (yielding)</td>
<td>Flexural (yielding)/shear (web buckling)</td>
<td>Shear (web buckling)</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5.1: Flowchart for fire resistance analysis of structural members

Figure 5.2: Element geometry used in thermal analysis
(a) Typical steel girder in a bridge

(b) 3-D mesh of segment (A-B)

(c) Cross section (A-A)

Figure 5.3: 3D discretization of typical steel bridge girder for thermal analysis
Figure 5.4: Element geometry used in structural analysis
Figure 5.5: 3D discretization of typical steel bridge girder for structural analysis
Figure 5.6: Steel girder with symmetric restraint conditions

Figure 5.7: Structural discretization and modeling of end restraint
Figure 5.8: Eurocode stress-strain model for structural steel

Figure 5.9: Eurocode stress-strain model for concrete
Figure 5.10: Load-slip model in shear stud, used in analysis

(a) Longitudinal elevation

(b) Transverse section

Figure 5.11: Tested beam-slab assembly selected for validation
Figure 5.12: Comparison of predicted and measured response parameters in fire exposed beam-slab assembly
Figure 5.13: 3-D discretization of typical MSU steel girders G1, G2, and G3 for thermal and structural model
Figure 5.14: Comparison of predicted and measured cross-sectional temperatures in steel girders G1 and G2
Figure 5.15: Comparison of predicted and measured mid-span deflections in steel girders G1, G2, and G3.
Figure 5.16: Predicted failure pattern in girders G1, G2, and G3 from ANSYS
5.5 Summary

This chapter presents the development and validation of numerical model for tracing the response of steel bridge girder exposed to fire. The numerical model, comprising of thermal and structural finite element sub models, are developed using commercially available finite element software ANSYS. Two stages associated with the fire resistance analysis, namely: thermal and structural analyses, modelling restraint situations, are explained. The developed model accounts for high temperature thermal and mechanical properties, various fire scenarios, fire induced axial restraint effects, composite action arising from steel girder-concrete slab interaction, and geometrical nonlinearities.

The validity of the thermal and structural models is established through comparing the analysis results to data generated from fire resistance tests of typical steel girders. This validation indicates that ANSYS is capable of tracing the fire response of steel bridge girders and thus can be used for undertaking numerical studies. In Chapter 7, this validated model will be applied to carry out a set of parametric studies on the effect of critical parameters influencing fire response of steel bridge girders. Also, the developed numerical model will be utilized in Chapter 6 to undertake numerical studies to evaluate the residual capacity of steel bridge girder after fire exposure using relevant residual material properties.
6. RESIDUAL CAPACITY EVALUATION

6.1 General

In many cases, fires in bridges burn-out quickly or are extinguished through firefighting. Thus structural members in bridges might retain much of their load carrying capacity after exposure to a fire. The extent of capacity degradation depends on the severity and duration of the fire, geometrical properties of the girder, and the degradation of strength and stiffness properties of materials that form the structural members (Franssen and Kodur, 2001). At present, there is no methodology or approach specified in bridge codes and standards to evaluate the residual capacity of steel bridge girders after fire exposure. Therefore, a methodology for evaluating residual capacity of steel structural members can be useful for practicing engineers. Also, a simplified approach for assessment of residual capacity of fire exposed steel bridge girders is necessary before routing the traffic on the bridges. Such an assessment also helps in developing strategies for retrofitting structural members in bridges (Aziz and Kodur, 2012).

The finite element model presented in Chapter 5 can be utilized to simulate the multi-stages analysis to evaluate the residual capacity of steel bridge girders after fire exposure. However, relevant material properties and suitable material models for steel and concrete at different stages are to be considered in the analysis. In this chapter, a methodology for evaluating the residual capacity of steel bridge girders after fire
exposure using finite element program ANSYS is presented. Results from analysis using
the proposed methodology are utilized to verify a simplified rational approach for
evaluating the residual capacity of fire exposed steel bridge girders.

6.2 Methodology for Evaluating Residual Capacity

A methodology for evaluating the residual capacity of fire exposed steel bridge
girders is developed. Full details of this methodology that can be implemented through
finite element based programs are outlined below.

6.2.1 General methodology

For evaluating residual capacity, strength analysis on the bridge girder has to be
carried out in three stages, namely at ambient conditions (stage 1), during exposure to fire
(stage 2), and following the cooling of the fire exposed girder (stage 3). The strength
analysis can be carried out using finite element based computer programs such as
ANSYS or ABAQUS. The three stages of the analysis for evaluating residual capacity of
fire exposed bridge girders are illustrated in Figure 6.1.

In the first stage of the analysis, prior to undertaking fire resistance analysis, load
carrying capacity of the bridge girder is to be ascertained through specified strength
equations in design standards. Alternatively, detailed finite element analysis can be
carried out by gradually incrementing the load on the girder till failure is attained. For
this analysis, room temperature strength and stiffness properties of structural steel,
concrete, and reinforcing steel are to be considered.

In the second stage of analysis, the bridge girder is to be analyzed by exposing the
girder to a given fire scenario, load level, and restraint conditions that are present during
fire exposure. Both thermal and structural response of the girder is to be traced to
evaluate the fire performance of the girder. In this stage, temperature dependent properties of structural steel, concrete, and reinforcing steel, including the change in the material properties that occur during the cooling phase of fire, are to be considered. This stage of the analysis is carried out at various time increments till failure of the girder occurs or till the total duration of fire exposure. Response parameters from the thermal (temperatures) and structural analysis (deflections, stresses, and capacity) are to be utilized at the end of each time increment to evaluate the state of the bridge girder under different failure limit states.

Following the cooling down of the bridge girder, if there is no failure in second stage, third stage of analysis is to be carried out. In this stage of the analysis, the bridge girder is loaded incrementally and the structural response of the girder is traced. For this analysis, residual properties of materials (concrete, steel, steel reinforcement) are to be considered. The load increments continue till the girder attains failure. The load at which failure occurs in bridge girder, indicate the residual capacity of the girder after exposure to fire.

6.2.2 Finite element model

Evaluating residual capacity in steel bridge girders requires strength (structural) analysis in stages 1, 2, and 3, as well as thermal analysis in Stage 2. For undertaking structural and thermal analysis in these stages, finite element approach can be applied. The finite element model developed in Chapter 5 using ANSYS program can be utilized to simulate the different stages of analysis mentioned above. This can be done by taking into consideration relevant material properties (thermal and mechanical) at each stage of analysis. For structural analysis in stage 1, 2, and 3, the composite bridge girder,
comprising of steel girder, concrete slab, and steel reinforcement can be discretized using SHELL181, SOLID185, LINK8, COMBIN39, CONTA174/TARGE170, COMBIN14 elements in ANSYS as described in Chapter 5. Analysis in Stage 2 requires heat transfer calculations to evaluate temperature in the steel girder and concrete slab. For undertaking heat transfer analysis in ANSYS, the thermal model can be discretized using SOLID70 and SURF152 elements as described in Chapter 5.

6.2.3 Properties of constituent materials

The properties of constituent materials play a crucial role in determining the capacity and response of a girder at different stages of analysis. The properties of steel and concrete degrade with temperature and are also affected by the heating and cooling phases of fire. Thus, relevant material properties are to be used at each stage of the analysis. In stage 1 of analysis, the room temperature strength and stiffness properties of structural steel, concrete, and steel reinforcement are to be used. During exposure to fire, (stage 2) the progression of temperatures in the steel girder and concrete slab depend on the fire intensity and thermal properties of constituent materials, namely thermal conductivity, specific heat and thermal expansion. These thermal properties vary as a function of temperature. The mechanical properties of steel and concrete that govern the fire response of bridge girder are yield strength, modulus of elasticity (stress-strain response) and these are also a function of temperature. The thermal and mechanical properties of steel and concrete degrade during heating phase of the fire; however these materials regain part of their strength and stiffness during cooling of the girder in the cooling phase of the fire.

In stage 3 of the analysis, after cooling down of the girder, the properties of steel
and concrete recover to some extent. The level of recovery in strength and stiffness properties of steel and concrete after cooling of the girder and slab depends on the maximum temperature attained during fire exposure (Kirby et al., 1986). This variation in properties during cooling phase is to be properly accounted for in evaluating realistic residual capacity of fire exposed structural members. However, there is lack of information on residual strength properties of steel and concrete after experiencing high-temperature in fire.

6.2.4 Failure criteria

In undertaking residual strength analysis of a bridge girder, different failure criteria are to be applied at each stage of the analysis, for evaluating failure of the girder. In the first stage of analysis, at ambient conditions, strength limit state generally governs the failure and the capacity of the bridge girder corresponds to the point at which failure occurs under flexure or shear limit state. In the second stage of analysis, during fire exposure, the girder experiences high-temperatures, and buckling in the web might dominate the failure limit state in steel girders due to higher slenderness of the web as compared to the flanges. Also, significant deflections that develop under fire conditions can lead to high level of fire induced forces at the connections when the girder is restrained from thermal expansion. These aspects (web buckling and state of the connections) are to be considered at each time increment to evaluate failure. Therefore, deflection, strength and stability limit state criteria are to be considered to evaluate the failure in the second stage of the analysis. In the third stage, strength limit state generally governs the failure and this has to be used to evaluate residual capacity of the fire exposed girder.
6.2.5 Validation of the residual capacity model

Validation of the developed model for Stage 1 of the analysis is carried out on a typical steel beam-concrete slab assembly, tested at room-temperature by (Chapman and Balakrishnan, 1964). The steel beam-concrete slab assembly (5.5 m span), together with sectional dimensions is shown in Figure 6.2. This composite assembly has steel yield strength of 302 MPa and a concrete compressive strength of 27 MPa. This steel beam-concrete slab assembly is disceritized for structural analysis using ANSYS elements as described above. In the analysis, the room-temperature stress-strain relation of steel and concrete are assumed to follow as that of Eurocode 2 and 3 provisions.

The comparison of mid-span deflections predicted by ANSYS model and those measured in the capacity test is shown in Figure 6.3. It can be seen that the mid-span deflection increases linearly with increasing loading up to yielding which occurs at 200 kN. After the yielding, mid-span deflection increases nonlinearly at a faster pace with increasing applied loading due to spread of plasticity in the beam. With further increase in loading beyond 375 kN, the mid-span deflection increases rapidly and this due to more plastic deformation. Finally, the beam-slab assembly loses its load carrying capacity and failure occurs at about 420 kN, through formation of a plastic hinge at the mid-span. Overall, predictions from the ANSYS match well with the reported test data. The slight difference in deflection predictions can be attributed to minor variations in idealization adopted in the analysis, such as actual and assumed stress-strain relations of steel and concrete.

The developed model for Stage 2 of the analysis is validated by comparing mid-span deflections predicted by ANSYS model and those measured in fire tests carried out at Michigan State University on three typical steel bridge girders (Aziz et al., 2015). The
validation shows that the developed model is capable of tracing the thermal and structural response of steel bridge girders under fire conditions. More details on this validation are presented in Chapters 5.

There is lack of test data on the residual capacity of steel bridge girders or beam-slab assemblies typical to that in buildings. Therefore, the developed model was not validated for Stage 3 of the analysis.

6.3 Numerical Studies on Residual Capacity

To illustrate the applicability of the proposed methodology in evaluating the residual strength of steel bridge girder after fire exposure, the following numerical studies are carried out.

6.3.1 General

The above discussed methodology in Section 6.2 was applied for evaluating the residual strength of fire exposed bridge girder. The analysis was carried out using the finite element computer program ANSYS. For the analysis, a typical steel bridge girder is selected. In Stage 1 of the analysis, the strength analysis is carried out and the room temperature capacity was evaluated. The response of the girder during fire exposure (Stage 2 analysis) is traced through two sets of discretization models, one for undertaking thermal analysis and the other for undertaking mechanical (strength) analysis. Results from thermal analysis are applied as thermal-body-loads on the structural model, uniformly along the girder span. High-temperature thermal and mechanical properties of steel and concrete, in both heating and cooling phase, are incorporated in the analysis. The state of the girder, as well as capacity was evaluated by applying relevant failure limit state. Following the cooling of the girder, residual capacity of the girder was
evaluated by undertaking Stage 3 of the analysis.

6.3.2 Selection of bridge girder

To illustrate the evaluation of residual capacity of a typical bridge girder exposed to fire, a steel bridge girder designed by FHWA was selected for analysis. The steel bridge comprised of five hot rolled steel girders of W33x141 supporting a reinforced concrete slab of 200 mm thickness. The steel girder is assumed to be in full composite action with slab and to be laterally supported by transverse diaphragms at the mid-span, as well as the both ends, to prevent lateral movement as shown in Figure 6.4. This bridge girder has simply supported span of 12.2 m and has two expansion joints at its ends with a width of 36 mm. The girders are fabricated from Grade 50 steel (yield strength of 350 MPa), while the concrete used in slab has a compressive strength of 40 MPa.

6.3.3 Material properties during heating, decay and after cooling

Thermal and mechanical properties of steel and concrete vary in different stages of the analysis. For Stage 1 of analysis, at room temperature, typical stress-strain model of Grade 50 steel ($f_y = 345$ MPa) is used for structural steel.

In Stage 2, during heating phase, the temperature dependent thermal and mechanical properties of steel and concrete are assumed to follow as that of Eurocode 2 and 3 provisions as discussed in Chapter 5. The variation of mechanical and thermal properties with respect to temperatures is different in heating phase as compared to cooling phase of fire and depends on the maximum temperature reached during the heating phase. During the cooling phase, the residual strength of steel is evaluated using linear interpolation between the strength of maximum temperature attained and residual strength at room temperature from Chapter 4 and as shown in Figure 6.5(a). Also, the
Compressive strength of concrete after cooling is assumed to be 10% less than the strength attained at the maximum temperature based on Eurocode 4 provisions and as shown in Figure 6.5(b) (EC4, 2003). This deterioration in strength properties are assumed to vary linearly between the maximum temperature attained and the room temperature. However, all the thermal properties of steel and concrete including; thermal expansion, thermal conductivity and specific heat are assumed to be fully reversible during the decay phase.

In stage 3, after cooling of the fire exposed girder, the residual yield strength of steel after cooling down to room temperature is assumed to have decreased according to the residual strength reduction factor presented in Chapter 4 (see Figure 6.5). The residual compressive strength of concrete after cooling down to room temperature is assumed to be 10% less than the strength attained at the maximum temperature. This assumption is according to Eurocode 4 provisions (EC4, 2003). For steel reinforcement, the residual strength is assumed to be fully reversible since temperatures in steel reinforcement do not exceed 600°C due to presence of concrete around the steel reinforcement.

6.3.4 Fire and loading scenarios

The structural analysis on the bridge girder was carried out by subjecting the girder to applied loading equivalent to 30% of the composite girder capacity at room temperature and different fire exposure scenarios. Three fire scenarios namely, hydrocarbon fire, moderate design fire, and external design fire were considered to study the effect of fire severity on the residual capacity of the bridge girder. In Case 1, the bridge girder was exposed to hydrocarbon fire, while in Case 2, a design fire exposure
was considered with the peak fire temperature reaching to 800°C, followed by a 60 minutes steady state burning and then entering the decay (cooling) phase. In Case 3 an external design fire with a maximum temperature of 680°C and a 45 minutes steady state burn-out prior to decay phase was used. The time-temperature curves representing Cases 1, 2, and 3 are shown in Figure 6.6.

6.4 Results and Discussion on Residual Capacity Studies

For evaluating residual capacity, strength analysis on the selected bridge girder is carried out in stages 1, 2 and 3 using the finite element model developed in Chapter 5 in ANSYS. The selected girder is analysed under above specified loading and fire exposure scenarios. The response in three stages of the analysis is presented below.

6.4.1 Response in Stage 1

Result of stage 1 of the analysis of the girder assembly, at room temperature is presented in Figure 6.7 in the form of load-mid-span deflection response. It can be seen that the mid-span deflection increases linearly with increase in load till yielding of steel and then the response becomes nonlinear due to the onset of material and geometric nonlinearity that have been incorporated in the analysis. In the nonlinear range of response, the mid-span deflection increases at a faster pace with small increments in loading and this is mainly due to spread of plasticity in the girder. This increase in load carrying capacity can be attributed to strain hardening of steel. Finally, the girder attains failure when it can no longer sustain any further increase in load. Results in Figure 6.7, show 6% variation between the predicted capacity of steel bridge girders from analysis with that predicted using design equations (Segui, 2013).
6.4.2 Response in Stage 2

In stage 2 of the analysis both thermal and structural responses are critical in evaluating the fire performance of the girder. Results from ANSYS thermal analysis are plotted in Figure 6.8 to illustrate the temperature distribution in the steel-concrete composite girder as a function of time for three cases of fire exposure under Cases 1, 2, and 3. It can be seen in Figure 6.8 that the top flange temperature in all three cases is much lower as compared to the bottom flange. This is mainly due to the insulating effect of the concrete slab that helps dissipating heat from the top flange to the concrete slab. Also, temperatures in the web are slightly higher as compared to that in bottom flange and this is because the web is much more slender (lower thickness and higher surface area) than the flanges and this produces rapid rise in web temperatures. But after the steady state period and entering the decay phase, web temperature decreases at a faster rate than the bottom flange temperature due to lower thickness of the web as compared to the flange. In the cooling phase, the top flange looses heat slower as compared to the web and bottom flange. This is because of the heat sink effect from the concrete slab that absorbs and dissipates heat slowly, due to lower thermal conductivity and higher specific heat of concrete. Therefore, it takes longer time for the concrete and top flange to cool down.

The large difference in temperatures between the web and mid-depth of the slab leads to significant thermal gradients across the girder-slab cross section. These thermal gradients are primarily influenced by the fire scenario (fire severity). For example, at 15 minutes, the thermal gradient is 900°C in Case 1 fire, as compared to 580°C in Case 2 fire and 480°C in Case 3 fire scenario. In general, higher thermal gradients produce higher thermal strains at the bottom of the steel girder (and in web), as compared to that
in concrete slab. Thus, a significant curvature (thermal bowing) is developed in the girder, resulting in high thermal stresses even in a statically determinate girder (unrestrained girder). The developed curvature at the initial stages of fire exposure is independent of applied loading because this curvature results mostly from the thermal gradient effect. Therefore, the curvature, resulting from the thermal gradients alone, contributes to increase in deflections at the early stage of fire exposure.

The structural response of the bridge girder in stage 2 of the analysis, during fire exposure is illustrated in Figure 6.9, wherein mid-span deflection is plotted as a function of fire exposure time. The load-deflection response is plotted for three fire exposure scenarios (Case 1, Case 2, and Case 3) that are considered in the analysis. The general trend of deflection progression can be grouped into different stages. At the early stage of the fire exposure, the mid-span deflection increases due to significant thermal gradients that develop along the cross section of the girder. During the intermediate stage of fire exposure, mid-span deflection increases linearly up to occurrence of first yielding, which depends on the temperature progression in the girder cross section. Therefore the time at which yielding occurs is different in different fire exposure cases. After temperature in steel exceeds 400°C, during the heating phase of fire exposure, the deflection increases with time at a faster rate due to spread of plasticity and deterioration in strength and stiffness properties of steel and concrete. During the steady state burning (when fire temperatures remain constant), the progression of mid-span deflection slows down significantly due to steady state temperature in the girder (steel) section. However, under Case 1 fire scenario, the mid-span deflection continues to increase till failure since the plasticity spread to much of the girder section, that resulted from high fire temperatures.
Towards final stages of fire exposure (in the cooling phase), the mid-span deflections in Cases 2 and 3 decreases since the girder temperature reduced significantly. This is due to recovery of strength and stiffness properties of steel and concrete due to cooling phase of fire.

The effect of fire scenario on the performance of the bridge girder can be gauged by comparing mid-span deflections from Cases 1, 2, and 3 as shown in Figure 6.9. In Cases 2 and 3, the bridge girder survived burn-out conditions under moderate design fire and external design fire scenario respectively; however in Case 1, the girder failed at 14 minutes into hydrocarbon fire exposure. This can be attributed to the fact that the fires in Cases 2 and 3 are less intense as compared to hydrocarbon fire in Case 1. For instance, the maximum fire temperature attained in hydrocarbon fire is about 1100°C in 8 minutes, (Case 1) as compared to 680°C in the case of external design fire (Case 3) and 800°C in the case of moderate design fire (Case 2). Also, the heating rate at early stages of fire is much higher in a hydrocarbon fire, than under external or moderate design fires, and this produces higher thermal gradients in the section. As an illustration, the fire temperatures at 12 minutes into fire is 1053°C in Case 1, as compared to 800°C and 680°C in Case 2 and 3 respectively. This differential in peak fire temperature and variation in heating rate between these three fires lead to slower deterioration in strength and stiffness properties in steel and concrete under Cases 2 and 3, as compared to Case 1. As a result, the bridge girder sustained the applied loading for the entire fire duration under external and moderate fire exposure scenarios (Case 2 and Case 3) and survived in the fire.

6.4.2 Response in Stage 3

A summary of the analysis results, including the post-fire residual capacity of the
bridge girder under different fire scenarios are presented in Table 6.1. The residual capacity (strength) of bridge girder exposed to maximum fire temperature of 800°C (in Case 2) is about 57% of the room temperature capacity, as compared to 72% under fire scenario with 600°C peak (fire) temperature (Case 3). The lower residual capacity in Case 2, is due to higher temperatures reached in the steel section in Case 2, as compared to that in Case 3. Therefore, steel (girder) under Case 2 fire exposure lost about 40% of its room temperature (yield) strength and stiffness permanently, which occurred mainly due to steel temperatures exceeding 600°C. As a result, the steel girder in Case 2 regained less stiffness and strength properties after cooling down to room temperature as compared to that of girder in Case 3 fire exposure. This resulted in permanent residual strains (deformations), which can be seen in Fig 6.10, and this reflects the level of plasticity reached in the girder during fire exposure. The residual deformation is higher in Case 2, as compared to Case 1, since the steel temperature in Case 2 reached 800°C as compared to 680°C in Case 3. Furthermore, the concrete slab in Case 2 also lost some of its strength due to spread of temperature in the bottom layer of the slab which reached higher levels as compared to that in Case 3.

6.5 Simplified Approach for Evaluating Residual Capacity

The main question that arises after bridge fire incidents is on the load bearing capacity of the structural members. This information on residual capacity is critical to determine whether the bridge is safe to reopen for traffic, or need some level of repair, or should be demolished and replaced. Therefore, an approach for assessment of residual capacity of fire exposed steel bridge girders is necessary before routing the traffic on the bridges.
6.5.1 General

Results from residual strength tests on steel presented in Chapter 4 is utilized to
developed a simplified empirical approach for evaluating residual capacity of fire
exposed steel bridge girder. This simplified approach is based on rational principles and it
is in form of simple equations. The proposed approach is verified by numerical studies
presented previously in this chapter.

6.5.2 Simplified approach

The simplified empirical approach for evaluating the residual strength of steel
bridge girders after fire exposure involves the following steps.

1. Estimation of maximum fire temperature and duration of fire exposure
(burning) using information from fire-fighters, visual observations of fire-
exposed steel-concrete section, and discoloration in steel and concrete.
2. Based on fire temperatures, estimation of the maximum temperature reached in
the steel during fire exposure using step-by-step or lump heat capacity method.
3. Estimation of the residual strength of steel and concrete using strength–
temperature relationships.
4. Computation of the residual capacity of steel bridge girders using moment
capacity equation of composite steel-concrete section at ambient temperatures
and with reduction factors to account for reduced strength of steel and concrete
after fire exposure.

The development of a design fire scenario in bridges involves three phases,
namely heating, steady-state, and decay phase as shown in Figure 6.4. In the heating
stage, during growth of fire, the temperature of the fire increases rapidly and the
maximum temperature is reached within short time. The intensity of fire and heating rate in this stage depends mainly on the fuel type. The steady-state stage starts once the maximum fire temperature is reached. In this stage, the burning continues with constant temperature. The duration of the steady-state stage depends on the amount of fuel available for burning. Eventually, the fire enters the decay stage, where the fire temperature drops once the fuel burns out or the fire is extinguished by firefighters. The maximum fire temperature, heating rate, and fire duration can be estimated from firefighters reports, steel and concrete discoloration and visual observation during fire event.

Knowing fire temperatures, the temperature progression in the steel section during fire exposure can be predicted using step-by-step or lumped heat capacity method. Sept-by-step method is capable of evaluating steel temperature in both bare and protected steel section and under both standard and design fires. This method assumes that a steel member is at a uniform temperature and adopts a quasi-steady-state lumped heat capacity approach (Buchanan, 2002). According to this approach the increment in temperature of unprotected steel member can be computed as follows:

\[
\Delta T_s = \frac{F_p}{A_s \rho_s c_s} \left[ h_c (T_f - T_s) + \sigma \varepsilon (T_f^4 - T_s^4) \right] \Delta t \text{........................................}[6.1]
\]

where \( \Delta T_s \) is the temperature rise in steel (°C), \( F_p \) is the heated perimeter (m), \( A_s \) is the steel cross-section (m\(^2\)), \( \rho_s \) is the density of steel (kg/m\(^3\)), \( c_s \) is the specific heat of steel (J/kg K), \( h_c \) is the convective heat transfer coefficient (W/m\(^2\) K), \( \sigma \) is the Stefan-Boltzmann coefficient (5.67x10\(^{-8}\) W/m\(^2\) K\(^4\)), \( \varepsilon \) is the effective emissivity, \( T_f \) is the fire temperature (K), \( T_s \) is the steel temperature (K), and \( \Delta t \) is the time increment in seconds.

Once the maximum temperature in the steel section is predicted, then residual strength of steel can be evaluated using strength-temperature relations. For A572 steel,
these strength-temperature relations (as presented in Chapter 4) are:

c) Air cooling after heating:

For $20^\circ C \leq T_s < 400^\circ C$

$$\frac{f_{y,R}}{f_y} = 1.0 \quad \text{..........................................................}[6.2]$$

For $400^\circ C \leq T_s \leq 1000^\circ C$

$$\frac{f_{y,R}}{f_y} = (1.0e - 8)T_s^3 - (2.12e - 5)T_s^2 + (1.31e - 2)T_s - 1.43 \quad \text{.............}[6.3]$$

d) Water quenching after heating:

For $20^\circ C \leq T_s \leq 1000^\circ C$

$$\frac{f_{y,R}}{f_y} = (5.94e - 12)T_s^4 - (9.71e - 9)T_s^3 + (4.04e - 6)T_s^2 - (3.6e - 4)T_s + 1.0 \quad \text{..........................................................}[6.4]$$

Finally, the residual capacity of a simply supported, composite steel-concrete girder can be computed using moment capacity equation which can be written as follows when the neutral axis locates in the concrete slab (Segui, 2013):

$$M_{u,R} = \phi A_s f_{y,R} \left( \frac{d}{2} + t_s - \frac{A_s f_{y,R}}{1.7 f_{c,R} b_{eff}} \right) \quad \text{..........................................................}[6.5]$$

where, $\phi$ is reduction factor =0.9, $M_{n,R}$ is the residual moment capacity, $A_s$ is the cross-sectional area of steel section, $f_{y,R}$ is the residual yield strength of steel, $d$ is the depth of the steel section, $t_s$ is the thickness of concrete slab, $b_{eff}$ is the effective width of the concrete slab, and $f_{c,R}$ is the residual compressive strength of concrete.

### 6.5.3 Validity of the simplified approach

The validity of the above simplified empirical approach is established by comparing results from the simplified approach with results from residual ANSYS
analysis presented above on steel bridge girders subjected to moderate (Case 2) and external (Case 3) fire scenarios. In simplified approach, step-by-step method is used to predict the temperature progression in the steel section. Results from thermal analysis using ANSYS (Stage 2) together with steel section computed from step-by-step method are plotted in Figure 6.11 for fire exposure in cases 2 and 3. It can be seen that steel temperature predictions from step-by-step method compare well with results from thermal analysis to predict the maximum temperature reached in the steel section. Step-by-step method is mainly derived to trace the average temperature in a steel section, while it is not capable of predicting temperature in concrete slab. Therefore, average (constant) temperature of 300°C is assumed in the concrete slab under both moderate and external fire exposure. This assumption is based on measured temperature profile in the concrete slab from fire tests on steel bridge girders presented in Chapter 3.

The maximum temperature of steel is utilized to compute the residual yield strength of steel from Eq. [6.3], while the residual compressive strength of concrete is assumed based on EC4. Air cooling is assumed during the decay phase since the material properties in Stage 3 of the numerical analysis was on the same base. Finally, the residual moment capacity of the steel girder is evaluated from Eq. [6.5] using the residual strength of steel and concrete. The residual moment capacity for cases 2 and 3 using the proposed simplified approach is illustrated in Table 6.2. Results from residual analysis for cases 2 and 3 together with residual capacity (%) prediction from the simplified approach are shown in Figure 6.12. It can be seen that the simplified approach is capable of predicting the residual capacity after fire exposure. Residual capacity (%) from ANSYS and the simplified approach is also compared in Table 6.3. As illustrated in that Table, results
from ANSYS and the proposed approach are comparable with 94% accuracy.
Table 6.1: Results from residual strength analysis of fire exposed bridge girder

<table>
<thead>
<tr>
<th>Case</th>
<th>Fire scenario</th>
<th>Max. fire temperature</th>
<th>Max. steel temperature</th>
<th>Room temperature capacity (kN.m)</th>
<th>Residual capacity (kN.m)</th>
<th>% of original capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Hydrocarbon fire</td>
<td>1100°C</td>
<td>1000°C</td>
<td>5104</td>
<td>Failure under fire</td>
<td>----</td>
</tr>
<tr>
<td>Case 2</td>
<td>Moderate fire</td>
<td>800°C</td>
<td>795°C</td>
<td>5104</td>
<td>2915</td>
<td>57%</td>
</tr>
<tr>
<td>Case 3</td>
<td>External fire</td>
<td>680°C</td>
<td>670°C</td>
<td>5104</td>
<td>3665</td>
<td>72%</td>
</tr>
</tbody>
</table>

Table 6.2: Residual capacity results of fire exposed bridge girder from the proposed approach

<table>
<thead>
<tr>
<th>Case</th>
<th>Fire scenario</th>
<th>Max. fire temperature</th>
<th>Max. steel temperature</th>
<th>Room temperature capacity (kN.m)</th>
<th>Residual capacity (kN.m)</th>
<th>% of original capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 2</td>
<td>Moderate fire</td>
<td>800°C</td>
<td>795°C</td>
<td>5104</td>
<td>2732</td>
<td>54%</td>
</tr>
<tr>
<td>Case 3</td>
<td>External fire</td>
<td>680°C</td>
<td>670°C</td>
<td>5104</td>
<td>3450</td>
<td>68%</td>
</tr>
</tbody>
</table>

Table 6.3: Comparison of residual capacity results from finite element analysis and the proposed simplified approach

<table>
<thead>
<tr>
<th>Case</th>
<th>Residual capacity % (ANSYS)</th>
<th>Residual capacity % (Simplified approach)</th>
<th>Residual capacity(Simplified approach)/Residual capacity (ANSYS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 2</td>
<td>57%</td>
<td>54%</td>
<td>95%</td>
</tr>
<tr>
<td>Case 3</td>
<td>72%</td>
<td>68%</td>
<td>94%</td>
</tr>
</tbody>
</table>
Figure 6.1: Flow chart illustrating various stages in undertaking residual strength analysis

1. **Stage 1**: Discretization of girder for thermal and structural analysis
   - Room temperature mechanical properties
   - Evaluate capacity at room temperature

2. **Stage 2**: High-temperature thermal and mechanical properties
   - Evaluate response during fire exposure
   - No failure
   - Failure

3. **Stage 3**: Residual strength (temperature dependent) properties
   - Evaluate the residual strength after cooling
   - Stop
Figure 6.2: Tested beam-slab assembly selected for validation of Stage 1

Figure 6.3: Comparison of predicted and measured mid-span deflections in beam-slab assembly at ambient condition
Figure 6.4: Longitudinal and transverse sections of the typical steel bridge
Figure 6.5: Residual strength of steel and concrete used in the analysis
Figure 6.6: Time-temperature curves for different fire scenarios used in analysis

Figure 6.7: Structural response of the bridge girder at ambient conditions (Stage 1)
Figure 6.8: Temperatures progression in a bridge girder subjected to different fire scenarios (Stage 2)
Figure 6.8 (cont’d)

(c) External design fire (Case 3)

Figure 6.9: Effect of fire severity on structural response of bridge girder during fire exposure (Stage 2)
Figure 6.10: Effect of fire severity on residual capacity of fire exposed bridge girder (Stage 3)

Figure 6.11: Comparison of steel temperatures obtained from thermal analysis and step-by-step method
Figure 6.12: Comparison of residual capacity predicted from finite element analysis and the proposed simplified approach
6.6 Summary

A detailed methodology and a simplified empirical approach for evaluating the residual capacity of fire exposed steel bridge girders are presented. The detailed methodology involves three stages of analysis that is to be carried out at ambient conditions, during exposure to fire, and then after cooling of the fire exposed bridge girder using finite element based programs. The proposed methodology is applied to carry out a set of numerical studies on typical steel girder using the finite element computer program ANSYS. Results from the numerical studies indicate that the maximum fire temperature (and associated temperature in steel) is the most critical factor that influences the residual strength of a fire exposed bridge girder. A girder exposed to typical “external” fire conditions, with maximum fire temperatures reaching to 600-700 °C, retains about 70 to 75% of its strength on cooling. On the other hand, a steel bridge girder exposed to hydrocarbon fire, with a maximum temperature of about 1100 °C, loses most of its strength during heating phase of the fire and experiences failure.

A simplified empirical approach for evaluating residual capacity of fire exposed steel bridge girder is presented in this chapter. Such an approach can assist in evaluating residual capacity of fire exposed steel bridge girders to make engineering judgement before routing traffic on fire exposed bridges or to develop strategies for retrofitting of fire exposed bridges. This simplified approach is derived based on rational principles utilizing strength-temperature relations for steel from residual strength tests presented in Chapter 4 using two methods of cooling, namely air cooling and water quenching. The validity of the simplified approach is established by comparing predictions from the proposed approach with results from detailed finite element analysis. This validation indicates that the simplified empirical approach is capable on evaluating the residual
capacity with good accuracy.
CHAPTER SEVEN

7. PARAMETRIC STUDIES

7.1 General

Although fire tests can provide better insight into the behavior of steel bridge girder under fire conditions, it is not feasible to undertake large number of fire experiments due to high complexity, huge costs and time constraints. Further, there are number of limitations relating to number of factors that can be varied in fire tests, reliable instrumentation that can be mounted for monitoring response, and availability of test facilities for undertaking fire tests. Therefore, use of numerical modeling can be an effective way to trace the fire response of steel bridge girders. Such numerical models can be applied to undertake parametric studies to evaluate the influence of various factors on fire resistance of steel bridge girders.

The validated numerical model, presented in Chapter 5, is applied to quantify the effect of various parameters on the fire response of steel bridge girder. The varied parameters include: load level, fire scenario, exposure scenario, web slenderness, stiffeners spacing, span length, axial and rotational restraint.

7.2 Definition and Range of Parameters

Based on a review of available fire resistance studies, the main parameters influencing fire response of steel bridge girders are introduced and defined in this section.
The range of these parameters in realistic and practical scenarios is also discussed.

7.2.1 Load level

The load level (LL) is a measure of stress level in the girder just prior to fire exposure. LL can be defined as the ratio of the maximum bending moment ($M_{\text{max}}$) induced in the girder due to the applied loading to the unfactored bending capacity ($M_{\text{capacity}}$) of the composite girder (steel girder-concrete slab) at room temperature, i.e.

$$LL = \frac{M_{\text{max}}}{M_{\text{capacity}}} \times 100\%$$

[7.1]

As discussed in Chapter 2, the extent of load on a structural member during fire conditions depends on number of factors including, magnitude of loading present, type of live load, (dead/live) load ratio, safety factors used for design under both room temperature and fire conditions. Since fire is a rare event, the load level during a fire event is less than the ultimate load used for the room-temperature design of the structural member. In current codes of practice, fire resistance in structural members in buildings is generally evaluated based on a critical load level of 50%. This is based on probability of existence of much lower live load (furniture, people) during fire events. This ratio can be less than 50% in case of bridges due to low probability of presence of vehicle loading (axle loads) during fire incidents. Therefore, in this study, the load level on a bridge girder is considered to be in the range of 20% to 50%.

7.2.2 Fire scenario

Fire scenarios considered in structural fire design are generally grouped under two categories, namely; standard and design fire scenarios. In the case of a standard fire, the heat flux from the fire source to the exposed structural members assumed to increase
throughout the duration of fire exposure. In the case of a design fire, the heat flux from the fire to the exposed structural members is assumed to increase till maximum temperature is attained then it assumed to be decreased gradually in the decay phase. In standard fires a continuous supply of fuel and oxygen is present, while in design fires the growth and decay are controlled by fuel and ventilation availability. Fire intensity in bridges is generally dependent on the type and amount of combustible/flammable material (gasoline, wood, etc.) carried by vehicles (trucks) involved in collision or other fuel available in the vicinity of the bridge. In this study, standard fire are represented by a hydrocarbon fire (gasoline fire), or an ISO 834, and an external fire scenarios (other combustible material such as wood) to simulate different levels of fire severity in different fire situations. For design fire scenario, a design fire is constructed with maximum temperature of 930°C attained within 20 minutes and with decay rate of 12°C/min. This is to simulate a gasoline fire that burns-out or extinguished by firefighters. The time-temperature curves for theses fire scenarios are shown in Figure 7.1.

7.2.3 Web slenderness and stiffeners spacing

The girders in bridges are generally much deeper than beams used in buildings and often have quite slender webs. Rolled steel beam sections typically used in buildings have web slenderness \((D/t_w)\) in the range of 30 to 50. However, typical steel girders used in bridges have web slenderness up to 150 for girders with traverse stiffeners and up to 300 for girder with longitudinal stiffeners. Web slenderness adopted in a specific steel bridge girders depend on the required shear capacity, span to weight ratio, and cost considerations. Increasing web thickness in steel bridge girders result in higher shear
capacity, but in the meantime, increases self-weight of the girder (dead load). Therefore, an alternative way to increase shear capacity in steel bridge girders is to provide stiffeners. Presence of stiffeners increases the shear capacity of steel girder through tension field action that occurs after buckling of the web. The stiffeners spacing to steel girder depth ratio \( (a/D) \) can vary depend on the required shear capacity of steel girders; however, commonly the \( (a/D) \) ratio range is 1 to 2. In this study, the web slenderness \( (D/t_w) \) is varied from 50-100, while stiffeners spacing ratio \( (a/D) \) is varied from 1 to 1.5.

### 7.2.4 End restraints

Steel bridge girder can be designed as statically determinate or indeterminate structures in continuous or single spans. In statically determinate bridges, single girders are separated by expansion joints, while for continuous spans, the expansion joints are located between the end of the girder and bridge abutment as illustrated in Figure 7.2. When fire induced axial expansion exceeds expansion joint, bridge girders behave as restraint members. Restraint conditions also can present in statically indeterminate bridges due to support conditions (pin-pin supported single or continuous span).

The end restraint conditions present in a bridge girder is modeled using axial and rotational springs as described in Chapter 5. The axial and rotational stiffness of the springs \( (K_a \text{ and } K_r) \) are varied independently to study the influence of axial and rotational restraint conditions on the fire response of steel bridge girders. The axial restraint ratio \( (Ar) \) is defined as the ratio between the axial restraint stiffness \( (K_a) \) applied on the girder from adjacent span to the axial stiffness for the fire exposed girder, i.e.:

\[
Ar = \frac{K_a (\text{adjacent span})}{K_a (\text{fire exposed span})} \times 100\% \Rightarrow Ar = \frac{[(E_s A_s / L) + (E_c A_c / L)]_{\text{(adjacent span)}}}{[(E_s A_s / L) + (E_c A_c / L)]_{\text{(fire exposed span)}}} \times 100\% \quad ........[7.2]
\]
where, $E_s =$ elastic modulus of steel section, $A_s =$ cross-sectional area of steel section, $E_c =$ elastic modulus of concrete, $A_c =$ cross-sectional area of concrete slab, and $L =$ girder span.

In the same manner, rotational restraint ratio ($Rr$) is defined as the ratio between the rotational restraint stiffness ($K_r$) applied on the girder due to continuous adjacent span to the rotational stiffness for the fire exposed girder:

$$Rr = \frac{K_r(\text{adjacent span})}{K_r(\text{fire exposed span})} \times 100\% \Rightarrow Rr = \frac{[(E_s I_s / L) + (E_c I_c / L)]_{(\text{adjacent span})}}{[(E_s I_s / L) + (E_c I_c / L)]_{(\text{fire exposed span})}} \times 100\% \text{ .........}[7.3]$$

where, $E_s =$ elastic modulus of steel section, $I_s =$ Moment of inertia of steel section about center of stiffness of the girder section, $E_c =$ elastic modulus of concrete, $I_c =$ Moment of inertia of concrete slab about center of stiffness of the girder section, and $L =$ girder span.

The degree of axial and rotational restraint in a steel bridge girder depends on the axial and rotational stiffness offered by adjacent spans as illustrated in Figure 7.2. Assuming similar cross-section for both fire exposed and adjacent spans, then the variation of axial and rotational stiffness is dependent on span length of the adjacent span. Axial and rotational restraint ($Ar$ and $Rr$) of 30%, 50%, 100% and 200% are considered to account for adjacent spans of $3L$, $2L$, $L$, and $0.5L$ respectively, where $L$ is the span of fire exposed girder. The axial and rotational restraints are assumed in this study to be elastic and constant. Also, the restraint stiffness is assumed to be invariable with respect to time of fire exposure. This assumption is justified for the case when the adjacent span (to which the girder extends) is not exposed to fire.

Location of axial restraint force can vary depending on how the fire exposed girder and the adjacent girder or the bridge abutment get into contact after exceeding the expansion tolerance at the joint as shown in Figure 7.2. In the restraint situation
illustrated in Figure 7.2(a), contact of the fire exposed girder with the adjacent girder occurs through lower part of the girder near the bottom flange. Therefore, location of the restraint force is assumed to be at a distance of \((d/6)\) from the bottom flange, where \((d)\) is depth of the girder section. For the restraint situation illustrated in Figure 7.2(b), in which the adjacent span get in to contact with the abutment, the location of restraint force is considered to be at the center of stiffness of the composite section. Assuming the restraint location to be at the center of stiffness results in fire induced axial force not generating significant moment around the girder neutral axis. However, if the location of the axial restraint is not at the stiffness center of the section, then a significant bending moment might develop at supports due to the eccentricity of the fire induced restraint force. The generated bending moment, resulting from fire induced restraints can affect the response of steel bridge girder under fire.

7.3 Analysis Details

The validated numerical model presented in Chapter 5 is applied to carry out parametric studies on a typical bridge girder that is shown in Figure 7.3. The selected bridge girder is analyzed by exposing it to a given fire scenario, load level, and support conditions. In the analysis, temperature dependent properties of structural steel, concrete, and reinforcing steel, are assumed to follow Eurocode 2 and 3 and these properties (thermal and mechanical) are presented in Chapter 5. The fire insulation applied on the steel girder in numerical analysis has specified thermal conductivity of 0.078 W/m\(^\circ\)C and density of 240 kg/m\(^3\) at room temperature (Isolatek International, 2008). For the case of intumescent coating, the properties at room temperature are 0.1 W/m\(^\circ\)C (thermal conductivity) and 1300 kg/m\(^3\) (density). The high-temperature thermal properties of fire
insulation and intumescent coating (both thermal conductivity and specific heat) used in numerical studies are assumed to follow as that recommended by (Bentz and Prasad, 2007) and (Krishnamoorthy and Bailey, 2009) respectively. These thermal properties for both fire insulation and intumescent coating are given in Tables 7.1 and 7.2.

The selected steel bridge girder is discretized for thermal and structural analysis using ANSYS elements as described in Chapter 5. Results of thermal analysis are applied as nodal temperatures on the structural mesh of the steel girder to evaluate structural response. The analysis is carried out at various time increments till failure occurs in the girder. In each time step, various response parameters from thermal and structural analysis are utilized to evaluate the state of the bridge girder under different failure limit states. At any time step, the analysis terminates if failure is attained, otherwise, the analysis continues to the next time step.

To define failure in steel bridge girders, the overall structural response (mid-span deflection and web-out-of-plane displacement) is to be traced. Therefore, in parametric studies, strength and deflection limit states are applied for evaluating failure. The failure is said to occur when the mid-span deflection of the girder exceeds (L/30) or when the girder is unable to resist the applied loading.

7.4 Results from Parametric Studies

Parametric studies are carried out to study the influence of critical factors on the fire response of steel bridge girders. In parametric studies a total of 51 girders (cases) are analyzed for structural analysis. Thermal and structural response, as well as failure patterns, are compared to evaluate the effect of load level, fire scenario, exposure scenario, web slenderness, stiffeners spacing, span length, axial and rotational restraint on
the response of steel bridge girders under fire conditions. Results from structural analysis are presented in terms of mid-span deflection and web out-of-plane displacement as a function of time exposure. Results of web out-of-plane displacement presented in parametric studies is at mid-depth of the girder, at a distance of \((d/2)\), from the support, where \((d)\) is the depth of steel section. Parametric studies matrix and the corresponding fire resistance results for the 51 analysis cases are presented in Table 7.3.

7.4.1 Effect of fire scenario

To study the influence of fire scenario on response of steel bridge girders, four types of fire exposures are selected for analysis, namely hydrocarbon fire, design fire, ISO 834 fire, and external fire. The selected bridge girder is subjected to four fire scenarios, whose time-temperature curves are shown in Figure 7.1. The composite girder is assumed to be exposed to fire from 3 sides, with concrete slab on top of the girder.

**Thermal response**: Results from thermal analysis on the steel-concrete composite girder are shown in Figure 7.4 in which temperature profile in the girder is plotted as a function of time exposure under four fire scenarios. It can be seen that in all cases of fire scenario, the top flange temperature is much lower as compared to bottom flange. This is mainly due to insulating effect of the concrete slab that dissipates heat from top flange of steel girder to the slab. Also, the temperatures in the web are slightly higher as compared to bottom flange and this is because the web is much more slender (lower thickness) than the flanges and this produces rapid rise in web temperatures. The plateau that occurs at steel temperature between 750-800°C (in steel girder) is attributed to the phase change that occurs to the structural steel and this process absorbs a considerable amount of heat.

In the case of design fire (Figure 7.4(d)), the maximum average temperature in
steel generally occurs during the cooling phase of the fire. This is mainly attributed to the thermal lag effect and redistribution of heat inside the steel section due to presence of concrete slab. The temperature distribution in the concrete slab is much lower than that of steel section and this due to high thermal conductivity and low specific heat of steel as compared to concrete. The differences between slab and steel temperatures result in significant thermal gradients along the cross-section of the girder.

The thermal gradients across the girder-slab cross section are plotted in Figure 7.5 for the four fire scenarios. The thermal gradients are computed as the temperature difference between the mid-depth of the slab and mid-depth of the web. In the cases of hydrocarbon and external fire exposure, the thermal gradients in the girder decrease as the maximum temperature is attained, while in case of ISO 834 fire exposure, the thermal gradients continue to increase but at a slower pace. This can be attributed to the fact that the steel girder tends to attain thermal equilibrium as fire progresses. For design fire scenario, the thermal gradients reduce after achieving maximum temperature in steel and then it reverses direction due to decay phase.

Results plotted in Figure 7.5 show that the thermal gradients that develop in steel girder is influenced by the type of fire exposure (fire severity and heating rate). This can be seen by comparing resulting temperature gradients from hydrocarbon fire with the other fire scenarios. For example, at 15 minutes, the thermal gradient is 1003°C in case of hydrocarbon fire exposure, as opposed to 700°C, 550°C and 500°C under design, ISO 834 and external fire scenario respectively. This is attributed to the fact that hydrocarbon fire is much more severe (higher fire temperatures and heating rate) as compared to the other fire scenarios. Other factors that influence thermal gradients include the depth of
the steel girder and the thickness and properties of concrete slab which acts as a heat sink for the girder.

**Structural response:** The effect of fire scenario on structural response of steel bridge girder is shown in Figure 7.6 in which the mid-span deflection and web out-of-plane displacement are plotted as a function of fire exposure under hydrocarbon fire, design fire, ISO 834 fire and external fire scenarios. The general trend of mid-span deflection progression can be discussed under three different stages. It can be seen that the mid-span deflection gradually increases linearly with fire exposure time at the early stage of fire exposure (Stage 1) up to first yielding, which depends on the temperature progression in the girder cross section. These initial deflections (when steel temperature is less than 400°C) are mainly due to thermal curvature resulting from high temperature gradients that develop along the girder section. Therefore, steel girder experiences different level of deflection depending on thermal gradients arising due to fire severity. This can be seen in Figure 7.6(a) for time exposure up to 10 minutes when the mid-span deflection under hydrocarbon fire is higher as compared to other fire scenarios.

With temperature progression, the mid-span deflection, in Stage 2, starts to increase at a slightly higher pace due to increasing of thermal gradients and degradation in strength and elastic modulus of steel resulting from increased temperatures in the steel girder, which exceeds 400°C. Therefore, the mid-span deflection continues to increase at higher level under hydrocarbon fire, as compared to other fire scenarios. In this stage, the web starts to experience some instability due to local buckling (see Figure 7.6(b)) resulting from high temperature raise in the slender web.

In the final stage (Stage 3) of fire exposure the rate of deflection increases
significantly due to spread of plasticity in bottom flange and more buckling of the web that result from faster strength and stiffness degradation of steel at high temperature and also due to the effect of high temperature creep. Finally, the steel girder experiences failure due to loss of load bearing capacity resulting from excessive web buckling and mid-span deflection.

Results plotted in Figure 7.6 show that fire resistance (failure time) of a steel girder is highly influenced by fire scenario (level of fire severity). The steel girder exposed to hydrocarbon fire failed in 14 minutes, while the steel girder exposed to external fire survived 120 minutes of fire exposure. This variation can be attributed to severity of fire intensity that is much higher in the case of hydrocarbon fire as compared to that of external fire. In the cases of design and ISO 834 fires, the steel girder exhibit better performance as compared to case of hydrocarbon fire, and failure occurs at 22 and 33 minutes respectively. The low fire resistance of steel bridge girder under hydrocarbon fire, typical of fires resulting from crashing of gasoline tankers, infers that firefighters have very little time to respond to such fire incidents. Under such scenario steel girder are highly susceptible to collapse.

The web out-of-plane displacement under the four fire exposure scenarios is shown in Figure 7.6(b). It can be seen that web buckling occurs under all fire exposures at different times. The girder under hydrocarbon fire experiences intense buckling once the buckling started, while the girder in other cases, exhibit enhancement in buckling response after first buckling. This depends on the temperature progression in the web, flange, and bearing stiffener, which is different based on the fire exposure severity for each fire scenario. For example, the temperature in the web at buckling time is 960°C.
under hydrocarbon fire, while it is 703, 505, 420°C under design, ISO 834, and external fires respectively.

The enhancement in the web buckling response in the case of design and ISO 834 fires can be attributed to some level of tension field action exhibited by the flange and bearing stiffener that experience lower temperature as compared to hydrocarbon fire scenario. Therefore, less enhancement in buckling response is observed in this case (hydrocarbon fire) as compared to other cases. With progression of fire exposure and rise in web, flange, and bearing stiffener temperatures, in the case of design and ISO 834 fires, web buckling start to increase due to softening of the flange and bearing stiffener, resulting in run-away buckling of the web. Comparison of failure time predations from deflection limit state in Figure 7.6(a) with that of web buckling response (run-away buckling time) in Figure 7.6(b) show good agreement and this is due to the fact that increase in web buckling result in increase in mid-span deflection. However, web buckling run-away tends to occur earlier.

The effect of fire scenario on resulting failure modes in steel bridge girders is illustrated in Figure 7.7 in which the failure mode predations from ANSYS are shown under different fire scenarios. This figure is to illustrate how the web of a steel bridge girder is highly vulnerable for buckling under high temperature. Therefore, failure occurs through web shear buckling and flexural yielding interaction, however shear failure tends to occur first. It can be seen that the extent of web buckling depends on the fire exposure severity. The intense buckling of the web and mid-span deflection under hydrocarbon fire as compared to other fire scenarios is due to fire severity and higher heating rate for hydrocarbon fire that result in significant reduction in shear and flexural capacity of the
steel girder. Therefore, the girder fails in 14 minutes under hydrocarbon fire exposure through web shear buckling. Furthermore, the buckling of the bearing stiffener in case of hydrocarbon fire (see Figure 7.7) explains why no enhancement in buckling response is observed in this case.

7.4.2 Effect of load level

To investigate the effect of load on fire resistance of steel bridge girders, the numerical analysis is carried under four load levels, namely LL = 20%, 30%, 40% and 50%. Figure 7.8 shows the effect of load level on the response of the selected steel bridge girder under hydrocarbon fire scenario. It can be seen from Figure 7.8(a) that the mid-span deflection increases with increasing the applied load level on the girder. This is due to higher stresses developed in the girder prior to fire exposure, and this causes earlier spread of plasticity in the girder as compared to the cases with lower load level. Therefore, fire resistance of a girder decreases with increasing the load level. For example, fire resistance decreases from 16 to 11 minutes when applied load level is increased from 20% to 50%. Also, increasing the applied load level on the girder lead to early occurrence of web buckling and this is illustrated in Figure 7.8(b). The early occurrence of web buckling is due to earlier spread of plasticity in the web resulting from increase in applied loading.

7.4.3 Effect of localized burning (exposure scenario)

Severity of fire exposure on a bridge depends on the location of fire in the vicinity of the bridge with respect to bridge superstructure, size of the bridge structure, the amount and type of fuel available for burning. In some cases the entire span with of a bridge might be exposed to fire, while in other cases only part of the span width can be
exposed to fire. To study, the effect of localized burning on the fire response of steel bridge girders, the selected girder is analyzed under hydrocarbon fire using three exposure scenarios, namely entire span exposure, mid-span zone exposure, and support zone exposure. These fire exposure scenarios are illustrated in Figure 7.9. In the first case, the entire span of the girder (12.2 m) is exposed to fire, while only 4.2 m of the mid-zone of the span of 12.2 m is exposed to fire in second case. For third case, 4 m length of the span from the end support is exposed to fire. The mid-span deflection and web out-of-plane displacement of these cases are plotted in Figure 7.10 as a function of time exposure.

It can be seen that extent of mid-span deflection and web out-of-plane displacement is varied depend on the exposure scenario. The mid-span deflection in first case (entire span exposure) is higher than other cases and this is due to degradation of shear and flexural capacity simultaneously due to fire exposure on the entire span. However, in other cases (support zone or mid-span zone exposure); either the shear or the flexural capacity degrades due to fire exposure depends on the exposure zone. Therefore, the mid-span deflection is higher in case of mid-span zone exposure as compared to support zone exposure case. Figure 7.10(b) show that web buckling occur in the cases of entire and support zone exposure, while in the case of mid-span zone exposure no web buckling is indicated. This is expected since the web near the support is not exposed to fire in case of mid-span zone exposure. Results from Figure 7.10 indicate that progression of mid-span deflections prior to failure occur in a faster pace when web buckling is attained. This results in higher fire resistance (25 minutes) in case of mid-span zone exposure as compared to (14 minutes) for other cases.
It can be seen from Figure 7.11 that the failure mode of steel bridge girders change depends on the exposure scenario. This is due to the fact that the exposure scenario controls the failure limit state. For example, when the entire span is exposed to hydrocarbon fire, the girder fails through combined effect of web shear buckling and flexural yielding and this is clear from the extent of web-out-of plane displacement and mid-span deflection. However, when only the mid-zone of the girder span is exposed to fire, the girder exhibits flexural response and fails through flexural yielding and this is based on the fact no web displacement is indicated near the support, however, the girder in this case experiences web crippling at the mid-span due to compressive force delivered by the top flange (see Figure 7.11(a)). On other hand, the girder fails through web shear buckling when only the support zone is exposed to fire as shown in Figure 7.11(b). This is due to significant degradation in shear capacity resulting from elevated temperature.

7.4.4 Effect of web slenderness

The slender web of steel girders in bridges is highly vulnerable to failure under web shear buckling mode rather than flexural (bending) mode of failure under fire condition. To study the effect of web slenderness on the fire response of steel bridge girder, five different girders with varying web slenderness ($D/t_w = 30, 40, 50, 70, \text{ and } 100$) are analyzed under hydrocarbon fire exposure. The web slenderness of the girder is varied by changing the thickness of the web while the depth of the section kept constant. Therefore, cross-sectional temperature progression in the girder changes accordingly. The temperature profile in the girder for these cases under hydrocarbon fire scenario is shown in Figure 7.12.

The shear and flexural capacity in each of the above cases is varied based on
the web thickness. The analysis is carried by subjecting the girders to a loading equivalent to 30% of the flexural capacity of each girder and this translates to 12%-22% of the shear capacity for the web slenderness range adopted in the analysis. Result from analysis plotted in Figure 7.13 show that fire resistance is highly influenced by the web slenderness. It can be seen that a decreased web slenderness from 50 to 30 results in an increase in fire resistance from 14 to 26 minutes. This is due to the fact that an increased web thickness enhances the overall shear capacity of the girder and also leads to slower temperature raise in the web and resulting in flexural failure. Increasing web slenderness from 50 to 100 led to decrease in fire resistance of the steel girder from 14 to 7 minutes. This is due to lower web thickness that results in higher temperature rise in the web as compared to the flanges. As a result, the shear capacity degrades faster than flexural capacity resulting in shear web buckling failure.

As discussed previously, failure in fire exposed steel bridge girders can be through flexural yielding, web shear buckling, or through the combined effect of both. It can be seen from Figure 7.13(b), that the girder with web slenderness less than 40 experiences no web buckling during the fire exposure. Therefore, the failure in these cases is through flexural yielding. However, for web slenderness greater than 50, the failure occurs through web shear buckling. Failure through web shear buckling seems to be more aggressive as compared to flexural failure and this can be seen from steep (run away) deflection prior to failure resulting in lower fire resistance.

7.4.5 Effect of stiffener spacing

Stiffeners are often provided in bridge girders to enhance shear capacity of plate girders through development of tension field action. Presence of such stiffeners enhances
the shear capacity for thin webs, when \( D/t_w > 1.12\sqrt{Ek/f_y} \), where \( D/t_w \) is web slenderness, \( E \) = the elastic modulus of steel, \( k \) = buckling coefficient, and \( f_y \) = yield strength of the steel (web). When, the web is thicker \( D/t_w \leq 1.12\sqrt{Ek/f_y} \) (lower slenderness), presence of stiffeners has no specific advantage to enhance the shear capacity of steel girders. Therefore, a girder with web slenderness \( D/tw = 80 \) is selected in the current section since this ratio satisfies the above limit state for thin web. In parametric studies, stiffeners spacing (aspect ratio) of \( a/D=1.0 \) and 1.5 are used, where \( a = \) stiffeners spacing and \( D = \) web depth.

To study the effect of stiffeners on the fire response of steel bridge girders, a girder with \( D/tw = 80 \) is analyzed using three cases. In the first case, no stiffener is used, while in other two cases, stiffener spacing to depth ratio of 1.0 and 1.5 is used. The analysis is carried out under hydrocarbon fire and 30\% applied loading of flexural capacity at room temperature which is equivalent to 34\%, 26\%, and 24\% of room temperature shear capacity for cases of no stiffeners, \( a/D=1.5 \), and \( a/D=1.0 \) respectively.

The effect of presence of stiffeners on the fire resistance of steel bridge girders is shown in Figure 7.14. Results from this figure indicate that effect of presence of stiffeners on increasing the shear capacity of fire exposed steel girder, in turn enhancing the fire resistance, is not significant. This is due to softening of stiffeners resulting from high temperature that limited the development of tension field action. The extent of stiffening of the flexural response in Figure 7.14(a) reflects the level of enhancement in the shear capacity of the girder (developed tension field action). The girder in all cases fails through web shear buckling, however, presence of stiffeners decrease the extent of web out-of-plane displacement as shown in Figure 7.14(b).
7.4.6 Effect of span length

Bridges are classified based on span length for short (up to 15 m), medium (15-50 m), large (50-150 m) and extra-large (over 150 m) spans. Rolled steel girders can be used in short and medium span bridges, while steel plate girder can be existed in bridges with spans length up to 100 m.

To investigate the effect of span length the on fire response of steel bridge girders, the numerical analysis is carried considering different spans length of girders, namely 12.2 m, 17 m, and 22 m. Figure 7.15 shows the effect of span length on the response of the selected steel bridge girder under hydrocarbon fire scenario. It can be seen from Figure 7.15(a) that the mid-span deflection increases with increasing span length of the girder. This is due to the fact longer span lead to higher applied shear force and bending moment resulting in higher stresses developed in the girder. This cause earlier spread of plasticity in the girder as compared to the cases with shorter span resulting in earlier occurrence of web buckling and this is shown in Figure 7.15(b). Therefore, the fire resistance decreases with increasing span length. For example, the fire resistance decreases from 14 to 11 minutes with increasing the girder span from 12.2 m to 22 m.

7.4.7 Effect of axial restraint condition

Steel girders in bridges (simply supported spans) are separated by expansion joints. These expansion joints are designed for room temperature (20-100°C) and typically are 36 mm. Analysis results of the selected bridge girder under different fire scenarios indicate that the fire induced axial displacement exceeds the expansion joint under all of hydrocarbon, design, ISO 834 and external fire as shown in Figure 7.16. This refers to the fact that contact will occur with the adjacent span once the expansion
joint is exceeded. In parametric studies, analysis is carried out by varying the axial restraint stiffness from 0% to 200% as a percentage of the axial stiffness of the fire exposed girder.

To study the effect of axial restraint condition on the fire response of steel bridge girders, the selected girder is analyzed under wide range of axial restraint ratios, \( Ar = 0, 10\%, 30\%, 50\%, 100\%, \) and \( 200\% \), as well as the case of pin-pin support conditions. The axial restraint location is assumed to be at distance \( d/6 \) from bottom flange, where \( d \) is depth of steel section. Results of these cases are plotted in Figure 7.17 as a function of fire exposure time. It can be seen in Figure 7.17(a) that increasing axial restraint stiffness (\( Ar \)) enhances the flexural response of the girder. This is due to development of significant moments from the fire induced axial restraint force, that acts near bottom flange about the neutral axis. Increasing the axial restraint force with fire exposure leads to spread of plasticity resulting in web buckling following by buckling of the bottom flange near the support as. The web and bottom flange buckling occurs earlier with higher axial restraint stiffness (\( Ar \)) resulting in sudden decrease in the compressive axial force and this limits the positive influence of the fire induced axial restraint forces on the flexural response of the girder. With fire progression, web and bottom flange buckle excessively due to more spread of plasticity, and P-delta effect resulting in significant increase in mid-span deflection. Results from Figure 7.17 indicate no enhancement in fire resistance of the girder with increasing the axial restraint stiffness (\( Ar \)). This can be attributed to the web shear buckling that dominates the failure limit state rather than flexural failure.
7.4.8 Effect of rotational and axial restraint conditions

Degree of axial and rotational restraint in a steel bridge girder depends on the axial and rotational stiffness offered by adjacent spans. For homogenous spans (material and cross-section), the variation of axial and rotational stiffness is dependent on adjacent girder span as discussed previously. Axial and rotational stiffness are varied from 30%-200% as a percentage of the axial and rotational stiffness of the fire exposed girder. This is to simulate various span length scenarios of adjacent girder (3L, 2L, L, and 0.5L).

To investigate the effect of rotational restraint on fire resistance, the girder analyzed considering two restraint cases. In the first case, the analysis carried out for five rotational restraint ratio, namely Rr=0%, 30%, 50%, 100% and 200%, while the girder is allowed to expand freely (Ar=0). In the second case, the girder is assumed to be restraint rotationally and axially at the same time. In this case, the analysis is carried out under five rotational and axial restraint ratio, namely Rr=Ar= 0, 30%, 50%, 100%, and 200%. The axial restraint location is assumed to be at the center of stiffness of the composite section. Results from these two cases are plotted in Figures 7.18 and 7.19, respectively, as function of time exposure.

The results shown in Figures 7.18 indicate that higher rotational restraint ratio leads to improved performance under fire. This can be attributed to the fact that higher rotational restraint stiffness leads to greater overall stiffness of the girder. The results also show that it requires small value of rotational restraint (Rr = 30%) to cause a significant improvement in fire response. Increasing the rotational restraint stiffness of the girder, results in earlier occurrence of web buckling. However, no fire resistance enhancement is indicated with increasing the rotational stiffness and this can be attributed to web shear buckling that dominates the failure limit state.
Result from Figure 7.19 show that increasing rotational and axial restraint stiffness simultaneously enhances the flexural response of steel girder. This is due to fact that increasing the rotational stiffness improves the flexural response of the girder. In this case, the axial restraint doesn’t contribute to enhance the flexural response since it acts at the center of stiffness. However, increasing the axial restraint stiffness leads to earlier spread of plasticity resulting in earlier web buckling in the girder as shown in Figure 7.19(b). Occurrence of web buckling decrease the fire induced axial restraint force. With fire exposure progression, buckling of the web increases significantly due to more spread of plasticity, and P-delta effect resulting in significant increase in mid-span deflection. Results from Figure 7.19 indicate no enhancement in fire resistance of the girder with increasing the axial and rotational restraint stiffness (Ar and Rr). This can be attributed to the web shear buckling that dominates the failure limit state rather than flexural failure as shown in Figure 7.20. Results from analysis indicate enhancement in the flexural response after exclusive web buckling and this due to tensile catenary forces in the girder as shown in Figure 7.19(b).

7.5 Strategy for Enhancing Fire Resistance in Steel Bridge Girders

Results from above parametric studies indicate that fire resistance in steel bridge girders range from 7 minutes to 33 minutes and depend on many influencing factors including; fire scenarios, exposure scenarios, load levels, girder span length, web slenderness, stiffeners spacing, and restraint conditions. Results from parametric studies also show that variation of geometrical characteristics of steel section (web thickness and stiffeners spacing) might alter the mode of failure from shear to flexural but has limited effect on enhancing the fire resistance of steel bridge girders. Therefore, the most
effective way to enhance fire resistance of steel bridge girders is through provision of external fire insulation.

Based on overall understanding of fire response of steel bridge girder that developed from undertaking parametric studies, the follow strategies are proposed to enhance fire resistance of steel bridge girders using applications of fire insulation and intumescent coating in different configurations and various thicknesses as follows:

7.5.1 Apply fire insulation on web plate

Results from parametric studies show that high web slenderness in steel bridge girder (thinner web plate as compared to flanges) makes steel girder more susceptible to web bucking due to rapid rise of temperature in the web as compared to flanges. This result in failure through web shear buckling that occurs earlier than flexural failure mode. Therefore, one of the feasible ways to enhance the fire resistance of steel bridge girder is through insulating the web plate from both sides.

To study the effect of insulating web plate on the fire response of steel bridge girder, four insulation thicknesses are applied including; 6.4 mm, 12.7 mm, 19 mm, and 25.4 mm. The cross-sectional temperature profile in the girder for these case under hydrocarbon fire is plotted in Figure 7.21, while the structural response of the steel girder under these cases is plotted in Figure 7.22. Results show that applying 12.7 mm of fire insulation on web increases the fire resistance from 14 to 31 minutes. Furthermore, using thicker insulation on the web (of 25.4 mm) can increase fire resistance to 53 minutes. This is due to thermal effect of insulation that results in lower temperatures in the web. As a result, the shear capacity of the girder degrades at a slower pace. Therefore, no web buckling is indicated in Figure 7.22(b) when insulation applied on the web. The insulated
web results in lower temperature progression in the bottom flange to some extent due to redistribution of the heat in the section, but without decreasing the thermal gradients across the section significantly. For these reasons, the failure mode in the girder altered from web shear buckling to flexural bending.

7.5.2 Apply fire insulation on steel girder section

Results from previous section (applying fire insulation on web plate) show that insulating web plate from two sides, enhances the fire resistance in steel bridge girders by protecting web plate, as a result prevent failure through web shear buckling. However, when the web is well insulated, the failure occurs through flexural yielding since the bottom flange is not insulated and can experience rapid raise in temperature. Therefore, the other way to enhance the fire resistance of steel bridge girders is to insulate the steel girder on all three sides.

To see the effect of insulation the steel section on the fire response of steel bridge girders, four insulation thicknesses are applied; 6.4 mm, 12.7 mm, 19 mm, and 25.4 mm. The temperature profile in the girder for these cases under hydrocarbon fire is plotted in Figure 7.23, while the structural response of the steel girder under these cases is plotted in Figure 7.24. It can be seen that using 12.7 mm fire insulation increases fire resistance from 14 to 48 minutes. Furthermore, increasing the insulation thickness to 25.4 mm can increase the fire resistance to 110 minutes. This is due to the presence of insulation that leads to slower raise in web and flanges temperatures. Also, it can be seen the extent of mid-span deflection in Figure 7.24 is much lower as compared to mid-span deflection in Figure 7.22 for the first 10 min of fire exposure. This is due to low thermal gradients generated in the case of insulating the whole section of the girder as compared to high
thermal gradients generated in the case of insulating only the web.

7.5.3 Apply intumescent coating on steel girder section

Using intumescent coatings is another technique to enhance the fire resistance in steel bridge girders. The intumescent coating expands 15-30 times in thickness when exposed to fire, forming a thick layer of foam that insulates the steel structural members from heating. As part of the expansion process, the intumescent coating generates an outer ash-like char layer. With progression of fire exposure, the ash coating erodes, exposing the remaining intumescent coating to form more char. This process continues several times depend on the thickness of the coating (Wong et al., 2010).

To study the effect of applying intumescent coating on enhancing the fire resistance of steel bridge girders, four coatings thickness are used, namely 1 mm, 2 mm, 3 mm and 5 mm. The cross-sectional temperature profile in the girder for theses case under hydrocarbon fire is plotted in Figure 7.25, while the structural response of the steel girder under these cases is plotted in Figure 7.26. It can be seen that applying 2 mm intumescent coating increases fire resistance from 14 to 57 minutes. Further, increasing the coating thickness to 5 mm can increase the fire resistance to 120 minutes. Results from Figure 7.26(b), indicate that web buckling occurs when coating thickness is 1 mm, while using thicker coating results in no web buckling during fire exposure. The low temperature progression in the web and flange during fire exposure resulting from apply intumescent coating, decreases the thermal gradients across the girder section. Therefore, failure of the girder occurs through flexural yielding.
### Table 7.1: High-temperature material properties of fire insulation used in analysis

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<th>Temperature</th>
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### Table 7.2: High-temperature material properties of intumescent coating used in analysis based on Krishnamoorthy and Bailey (2009)

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### Table 7.3: Summary results of test parameters from parametric studies

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<td>Support zone (4.0m)</td>
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Table 7.3 (cont’d)

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<tr>
<td>37</td>
<td>Insulation thickness (only on web-2 sides)</td>
<td>6.4 mm</td>
<td>Load level=30%, Hydrocarbon fire, D/tw =50</td>
<td>19</td>
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<tr>
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<td>12.7 mm</td>
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<td>19 mm</td>
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<td>43</td>
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<td></td>
<td>25.4 mm</td>
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<td>53</td>
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<td>Insulation thickness (entire section-3 sides)</td>
<td>6.4 mm</td>
<td>Load level=30%, Hydrocarbon fire, D/tw =50</td>
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<tr>
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<td>12.7 mm</td>
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<td>25.4 mm</td>
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<td>5 mm</td>
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Figure 7.1: Time-temperature curves for different fire scenarios used in parametric studies

(a) Single spans

(b) Continuous span

Figure 7.2: Axial and rotational restraint scenarios in fire exposed steel bridge girder
Figure 7.3: Longitudinal and transverse sections of a typical steel bridge selected for analysis
Figure 7.4: Progression of cross-sectional temperatures in steel bridge girder under different fire scenarios
Figure 7.4 (cont’d)

(c) External fire

(d) Design fire
Figure 7.5: Progression of thermal gradients along the depth of bridge girder section under different fire scenarios
Figure 7.6: Effect of fire scenario on the response of steel bridge girders
Figure 7.7: Predicted failure modes in a steel bridge girder from ANSYS under different fire scenarios
Figure 7.8: Effect of load level on the response of steel bridge girders
Figure 7.9: Illustration of localized burning scenarios in steel bridge girder
Figure 7.10: Effect of local burning scenario on the response of steel bridge girders
Figure 7.11: Predicted failure modes in a steel bridge girder from ANSYS for different fire exposure scenarios.
Figure 7.12: Progression of cross-sectional temperatures in steel bridge girder under hydrocarbon fire for different web slenderness
Figure 7.13: Effect of web slenderness on the response of steel bridge girders
Figure 7.14: Effect of stiffeners spacing on the response of steel bridge girders
Figure 7.15: Effect of span length on the response of steel bridge girders
Figure 7.16: Axial displacement of steel bridge girder under different fire scenarios
Figure 7.17: Effect of axial restraint conditions on the fire response of steel bridge girders
Figure 7.18: Effect of rotational restraint on the fire response of steel bridge girders
Figure 7.19: Effect of axial and rotational restraint on the fire response of steel bridge girders
Figure 7.20: Predicted failure modes in a steel bridge girder from ANSYS for different restraint conditions.
Figure 7.21: Progression of cross-sectional temperatures in steel bridge girder under hydrocarbon fire for different fire insulation thickness on web
Figure 7.21 (cont’d)

Fire insulation on web (2 sides)
19mm thick insulation
- Hydrocarbon fire
- Bottom flange
- Web-mid depth
- Top flange
- Slab-bottom
- Slab-mid depth
- Slab-top

Fire insulation on web (2 sides)
25mm thick insulation
- Hydrocarbon fire
- Bottom flange
- Web-mid depth
- Top flange
- Slab-bottom
- Slab-mid depth
- Slab-top
Figure 7.22: Enhancing fire resistance of a steel bridge girder through applying fire insulation on the web (2 sides)
Figure 7.23: Progression of cross-sectional temperatures in steel bridge girder under hydrocarbon fire for different fire insulation thickness on steel section
Figure 7.23 (cont’d)

Fire insulation on steel section (3 sides)
19mm thick insulation

- Hydrocarbon fire
- Bottom flange
- Web-mid depth
- Top flange
- Slab-bottom
- Slab-mid depth
- Slab-top

Fire insulation on steel section (3 sides)
25mm thick insulation

- Hydrocarbon fire
- Bottom flange
- Web-mid depth
- Top flange
- Slab-bottom
- Slab-mid depth
- Slab-top
Figure 7.24: Enhancing fire resistance of a steel bridge girder through applying fire insulation on the entire steel section (3 sides)
Figure 7.25: Progression of cross-sectional temperatures in steel bridge girder under hydrocarbon fire for different intumescent coatings on steel section
Figure 7.25 (cont’d)
Figure 7.26: Enhancing fire resistance of a steel bridge girder through applying a coat of intumescent on the entire steel section (3 sides)
7.6 Summary

This chapter presents the influence of various factors on the fire response of steel bridge girders. The studied parameters include: fire scenario, load level, exposure scenario (localized burning), span length, web slenderness, stiffeners, and restraint stiffness (axial and rotational). Results from parametric studies indicate that typical steel girders used in bridges can experience failure in less than 20 minutes under hydrocarbon fire exposure. The time to failure in fire exposed steel girders is highly influenced by fire scenario (severity), exposure scenario, load level, web slenderness, and span length, while mode of failure is influenced by exposure scenario (localized burning) and web slenderness. Under hydrocarbon fire exposure, steel bridge girders fail through flexural yielding when web slenderness is less than 40, however failure mode shifts to web shear buckling when web slenderness in girders greater than 50. Failure due to web shear buckling appears to be more aggressive than flexural yielding and results in slightly lower fire resistance. Results from parametric studies also show that presence of stiffeners and restraint conditions have no influence on enhancing the fire resistance of steel bridge girders. However, increasing axial and rotational restraint stiffness enhances the flexural behavior of the fire exposed steel bridge girder.

Results from the parametric studies are utilized to develop a strategy to enhance the fire resistance of steel bridge girders. The strategy is developed based on overall understanding on the behavior of steel bridge girders under fire conditions and also based on the dominant failure limit states that develop in girders as seen through parametric studies. The most effective way to enhance fire resistance of steel bridge girders is through provision of external fire insulation. Fire insulation can be applied in different configurations and various thicknesses. The fire insulation can be applied only on the
web plate and this is based on the results that shown that slender web in steel bridge girders are highly vulnerable to temperature effects and result in web shear buckling failure. Applying 25 mm thick fire insulation on the web in steel bridge girder can enhance fire resistance from 14 minutes to 53 minutes. Furthermore, applying insulation thickness greater than 12 mm on the web changes the failure mode from web shear buckling to flexural yielding. Therefore, In order to enhance the fire resistance beyond an hour rating, fire insulation is to be applied on three sides of the steel girder section. This configuration enhances fire resistance up to 110 minutes for 25 mm insulation thickness. Another feasible way to apply fire proofing on steel bridge girder is through applying intumescent coatings on the external surface area of steel girder section that is exposed to fire (3-sides). Results from analysis indicate that applying intumescent coating with thickness of 5 mm can result in 2 hours fire resistance.
CHAPTER EIGHT

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

Response of steel bridge girders under fire condition is investigated to develop an understanding on the behavior of steel bridge girders under transient heating conditions. As part of experimental studies, fire resistance tests were carried out on three typical steel bridge girders. The main variable in the fire tests were load level, web slenderness and spacing of stiffeners. The experimental studies also, included high-temperature tensile strength, residual strength, and creep tests on high-strength low-alloy A572 steel commonly used in bridge applications. As part of numerical studies, a finite element based numerical model is developed to trace the response of steel bridge girders under fire conditions. The critical factors, namely: material and geometrical nonlinearities, high-temperature material properties, non-linear interactions between concrete slab and steel girder, and fire induced restraint forces are accounted for in the finite element model. The finite element model is validated by comparing predictions from the model with those measured during fire tests. The validated finite element model is applied to conduct parametric studies to quantify the influence of various factors on the fire response of steel bridge girders.

Also, as part of numerical studies, a methodology for evaluating the residual
capacity of fire exposed steel bridge girders is also developed. This methodology involves three stages of analysis, namely at ambient conditions, during exposure to fire, and then after cooling to room temperature. The validated finite element model is utilized to simulate three stages of analysis by considering relevant material properties for each stage. Results from analysis are utilized to develop a simplified approach for evaluating the residual capacity of steel bridge girders after fire exposure.

Results from the fire tests and parametric studies are utilized to develop a strategy, namely applying fire insulation for enhancing the fire resistance of steel bridge girders.

8.2 Key Findings

Based on the information presented in this study, the following key conclusions are drawn:

1. Fires can pose a significant hazard to steel bridges. In a severe fire incident, the time to failure of steel girders can be less than 30 minutes, which gives very little time for firefighters to respond. At present there are no specific fire resistance provisions in bridge design codes and standards to enhance structural fire safety of bridges. Also, there is no approach on evaluating the residual capacity of fire exposed steel bridge girders.

2. Currently, there is lack of data and understanding on the behavior of steel bridge girders under fire conditions. Information and fire protection specifications available for structural members in buildings might not be directly applicable to structural members in bridges since they behave differently due to differences in
resulting fire scenarios, load level, support conditions, and sectional characteristics.

3. Fire resistance tests carried out on three typical steel bridge girders under ASTM E119 fire, and realistic loading conditions indicate:
   
   - Steel bridge girders under sever fire conditions can experience failure in about 30 to 40 minutes. The failure time and mode of failure is highly influenced by web slenderness and fire intensity.
   - Rolled steel girders having web slenderness of about 50 do not experience web buckling under fire conditions. Thus, the failure in these girders is through yielding of bottom flange and in flexural mode. However, built-up steel plate girders, with web slenderness above 100, experience failure through web shear buckling under fire conditions.

4. Results from material property tests on high-strength low-alloy steel indicate that temperature dependent strength and stiffness reduction factors of A572 steel follow the same trend as that of carbon steel, but with minor variations. However, there are major differences in strength and stiffness recovery after high-temperature exposure. A572 recovers almost 100% of its room temperature yield strength when heated to temperature up to 600°C, regardless the method of cooling, while the extent of strength reduction beyond 600°C is dependent on temperature and method of cooling. The extent of creep deformations at a given temperature for A572 steel, increases with stress level and the creep deformations can be substantial when the stress level exceeds 50% of room temperature yield stress.
5. The proposed finite element model is capable of tracing the response of steel bridge girders under simultaneous loading and fire conditions, as well as in predicting residual capacity of post-fire exposed steel bridge girder. This model accounts for all critical parameters including material and geometrical nonlinearities, high-temperature material properties, composite interaction between concrete slab and steel girder, and fire induced restraints.

6. A methodology for evaluating the residual capacity of fire exposed steel bridge girders is proposed. The maximum fire temperature (and associated temperature in steel) is the most critical factor that influences the residual capacity of a fire exposed bridge girders. A girder exposed to typical “external” (low intensity) fire conditions, with maximum fire temperatures reaching to 600-700°C, retains about 70 to 75% of its flexural capacity on cooling. The proposed simplified approach provides simple and rational methodology for predicting residual capacity of steel bridge girder following fire exposure.

7. Results from parametric studies on the critical factors influencing the response of steel bridge girders under fire conditions indicate the following:

- Typical steel girders can experience failure in less than 20 minutes under hydrocarbon fire exposure. The fire resistance and failure mode is highly influenced by the fire scenario, exposure scenario, web slenderness, load level, and span length. Other factors such as presence of stiffeners and restraint conditions have no significant influence on fire resistance of steel bridge girders. However, increasing axial and rotational restraint stiffness enhances the flexural behavior of the fire exposed steel bridge girder.
• Under hydrocarbon fire exposure, steel bridge girders fail through flexural yielding when web slenderness is less than 50, however failure mode changes to web shear buckling when web slenderness in girders is greater than 50.

8. The fire resistance of steel bridge girders can be enhanced through applying fire insulation. Different configurations of insulation on steel girder can be adopted to achieve up to 2 hours fire resistance, which can minimize fire hazard in steel bridges to a great extent.

8.3 Recommendations for Future Research

While this study has developed fundamental understanding of the fire response of steel bridge girders, further research is required to extend the principles to other practical situations that are present in steel bridges. The following are some of the key recommendations for further research in this area:

• Temperatures in bridges during a fire event are dependent on the type and amount of fuel present. This study focused mainly on a uniform fire exposure along girder span, while limited cases of fire exposure scenarios are considered in analysis. Numerical studies can be extended to include the effect of localized burning on the behavior of steel bridge girders.

• In this study, the numerical analysis was carried out on a typical isolated steel bridge girder. This can be expanded by exposing entire bridge structure including all girders, bracing, slab, pairs and abutment, to fire. Also, numerical analysis carried out in this study was on straight steel bridge girder. Skewed,
curved, and other type of bridges such as steel hanger-pin bridge girders can be considered in the analysis utilizing the approach developed here.

- Residual capacity analysis in this study has been done under static load and focused on the yield strength of steel; therefore fracture toughness was not accounted for. Residual capacity analysis can be extended to consider reduction in the fracture toughness following fire exposure.

- Behavior of steel bridge girders under fire conditions is quite complex, therefore, further research on web shear buckling and local buckling, including fire tests and numerical modeling, are recommended.

- Development of detailed provisions for specific fire resistance requirement in bridges is recommended.

8.4 Research Impact

Fire in bridges can lead to significant economic and public losses. However, at present there are no specific fire resistance provisions in bridge design codes and standards. This is unlike in building codes and standards that specify adequate fire resistance provisions to structural members to maintain structural stability and integrity in the event of a fire. Also, there are no approaches to evaluate the residual capacity of steel bridge girders after fire exposure.

Results from this work can be utilized to develop strategies for enhancing the fire resistance of steel bridge girders through applying fire insulation. These materials (fire proofing) are available in marketplace and can be applied in factory after fabrication of steel bridge girders or on site for existing bridges. Results from numerical analysis show that these strategies can provide up to 2 hours fire rating. Incorporating these strategies in
bridge codes and standards will contribute to reduced loss of life, property damage, and traffic disturbing in fire incidents.

The proposed approach for evaluating residual capacity can be applied to evaluate residual capacity of steel bridge girders after fire exposure. This approach provides a quick assessment of residual capacity of fire exposed structural members before routing the traffic on the bridges after fire exposure. Such an assessment can also help in making proper engineering judgement for developing strategies for retrofitting structural members in bridges.
Appendix A

Design of Steel Girder G3 According to AASHTO

Three steel girders (G1, G2, and G3) are designed according to AASHTO specifications to carry out fire resistance tests. All of the girders are designed as composite assembly (steel girder-concrete slab). The steel girder G1 comprised of a compacted hot rolled section, satisfied all the failure limit states based on AISC provisions and it is designed only for composite action (shear studs) and bearing stiffeners. The other two girders (G2 and G3) are built-up sections and designed to satisfy all the failure limit states specified by AASHTO for plate girders. Since girders G2 and G2 are identical except for the stiffeners spacing, therefore, the design procedure with full calculation details for girder G3 (see Figure A.1) is illustrated in this section.

Design for Flexure

Check flange dimensions per Article 6.10.2.2

\[
\frac{b_f}{2t_f} \leq 12.0 \rightarrow \frac{7}{(2)(0.5)} = 7 < 12.0 \quad OK
\]

\[
b_f \geq \frac{D}{6} \rightarrow 7 > \frac{23}{6} = 3.833 \quad OK
\]

\[
t_f \geq 1.1t_w \rightarrow 0.5 > (1.1) \left(\frac{3}{16}\right) = 0.206 \quad OK
\]

Check the compactness of the composite section per Article 6.10.6.2

The specified minimum yield strengths of the flanges do not exceed 70.0 ksi.

\[
F_{yf} \leq 70 \text{ ksi} \rightarrow 50 < 70 \quad OK
\]
The web satisfies the requirement of Article 6.10.2.1.1 which is:

\[ \frac{D}{t_w} \leq 150 \rightarrow \frac{23}{0.1875} = 122.67 < 150 \quad OK \]

Where:

D = Clear depth of the web (in.).

t_w = Thickness of the web (in.).

The section satisfies the web slenderness limit:

\[ \frac{2D_{cp}}{t_w} \leq 3.76 \sqrt{\frac{E}{F_{yc}}} \]

Where:

D_{cp} = Depth of the web in compression (in.).

F_{yc} = Yielding stress of compression flange (ksi).

E = Elastic modulus (ksi).

D_{cp} depends on the location of neutral axis.

Tension force (Ts) = As Fy \( \rightarrow \) [2(7)(0.5)+(23)(3/26)](50) = 565.625 Kips

Compression force (C) = 0.85(fc)(ts)(beff) = 0.85(4)(5.5)(32) = 598.4 Kips

\[ a = \frac{A_sF_y}{0.85f_c b_{eff}} \rightarrow a = \frac{565.625}{0.85(4)(32)} \rightarrow a = 5.2'' \]

Since a < ts \( \rightarrow \) Neutral axis is in the slab \( \rightarrow \) D_{cp} = zero
The flanges satisfy the following ratio:

\[ \frac{I_{yc}}{I_{yt}} \geq 0.3 \quad \text{\rightarrow symmetric section \rightarrow 1.0 > 0.3 \ OK} \]

Where:

- \( I_{yc} \) = Moment of inertia of the compression flange about vertical axis in the plane of the web (in\(^4\)).
- \( I_{yt} \) = Moment of inertia of the tension flange about vertical axis in the plane of the web (in\(^4\)).

The section is compact composite section as per 6.10.6. and must satisfy the requirements of 6.10.7.

Check the strength limit state for composite section per Article 6.10.7

\[ M_u + \frac{1}{3} f_t S_{xt} \leq \phi_f M_n \]

Where:

- \( \phi_f \) = Resistance factor for flexure specified in Article 6.5.4.2 =1.0.
- \( f_t \) = Flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi), equal to zero for continuously braced flanges.
- \( M_n \) = Nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in.).
$M_a =$ Bending moment about the major-axis of the cross-section determined as specified in Article 6.10.1.6 (kip-in.).

$M_{yt} =$ Yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.).

$S_{xt} =$ Elastic section modulus about the major axis of the section to the tension flange taken as $M_{yt}/F_{yt}$ (in.$^3$).

If $D_p \leq 0.1D_t$, then:

$M_n = M_p$

Otherwise:

$M_n = M_p \left(1.07 - 0.7 \frac{D_p}{D_t}\right)$

Where:

$D_p =$ Distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.).

$D_t =$ Total depth of the composite section (in.).

$M_p =$ Plastic moment of the composite section determined as specified in Article D6.1 (kip-in.).

In this case, $D_p = a ightarrow D_p = 5.2''$

$D_p \leq 0.1D_t ightarrow 5.2 > 0.1 \times (29.5) = 2.95$, then:

$M_n = M_p \left(1.07 - 0.7 \frac{D_p}{D_t}\right)$

$M_p = 0.85 f_c a b_{eff} \left(\frac{d}{2} + t_s - \frac{a}{2}\right) \rightarrow = 0.85(4)(5.2)(32) \left(\frac{24}{2} + 5.5 - \frac{5.2}{2}\right)$
\[ M_p = 8429.824 \text{ (kips in)} \]

\[ M_n = 8429.824 \left( 1.07 - 0.7 \frac{5.2}{29.5} \right) \]

\[ M_n = 7980.0 \text{ (kips in)} \]

Since \( f_l = \text{zero} \) then ---\( M_u \leq \phi_f M_n \)

From analysis, ultimate shear capacity for steel girder G3 = 105 kips this result in

\( M_u = 7560 \) (kip.in)

\[ M_u \leq \phi_f M_n \rightarrow 7560 < (1)(7980) \text{ OK} \]

Note: For the concrete slab minimum steel reinforcement for temperature and shrinkage was used in the bottom of the concrete slab.

**Design shear studs per Article 6.10.10**

At the strength limit state, the minimum number of shear connectors, \( n \), over the region under consideration shall be taken as:

\[ n = \frac{P}{Q_r} \]

Where:

\( P = \text{Total nominal shear force between the point of maximum moment and zero moment, determined as specified in Article 6.10.10.4.2 (kip).} \)

\( Q_r = \text{factored shear resistance of one shear connector determined from Eq. 6.10.10.4.1-1 (kip).} \)
\[ Q_r = \phi_{sc} Q_n \]

Where:

\( Q_n \) = Nominal shear resistance of a single shear connector determined as specified in Article 6.10.10.4.3 (kip).

\( \phi_{sc} \) = Resistance factor for shear connectors = 0.85 specified in Article 6.5.4.2.

\[ P \text{ is taken as the lesser of:} \begin{cases} 
A_s F_y \to (11.3125)(50) = 565.625 \text{ kips control} \\
0.85 f_c b_{eff} t_s \to (0.85)(4)(32)(5.5) = 598.4 \text{ kips}
\end{cases} \]

\[ P = 565.625 \text{ kips} \]

\[ Q_n = 0.5 A_{sc} \sqrt{f_c E_c} \leq A_{sc} F_u \]

Where:

\( A_{sc} \) = Cross-sectional area of a stud shear connector (in\(^2\)).

\( E_c \) = Modulus of elasticity of the deck concrete determined as specified in Article 5.4.2.4 (ksi).

\( F_u \) = Specified minimum tensile strength of a stud shear connector determined as specified in Article 6.4.4 (ksi).

\[ Q_n = 0.5 A_{sc} \sqrt{f_c E_c} \leq A_{sc} F_u \]

\[ E_c = 1820 \sqrt{f_c} \text{ for normal weight concrete with } w_c = 0.145 \text{ kcf} \]

\[ E_c = 1820 \sqrt{4} \to E_c = 3640 \text{ ksi} \]
Using shear stud with Ø3/4" and Fy=50 ksi and Fu=60 ksi

\[
Q_n = \min \left\{ \frac{0.5 (0.4418) \sqrt{(4)(3640)}}{(0.4418)(60)} \right\} \rightarrow = 26.65 \text{ kips} \quad \text{control}
\]

\[
Q_r = 0.85 \times 26.5 = 22.525 \text{ kips}
\]

\[
n = \frac{P}{Q_r} \rightarrow n = \frac{565.625}{22.525} = 25 \text{ shear studs for half of the span}
\]

\[n = 50 \text{ shear studs for the entire span}\]

Shear studs spacing = L/50 ----> \(= (12)(12)/44 = 2.88"\)

Check traverse spacing, cover and penetration per Articles 6.10.10.1.3 and 6.10.10.1.4

\[
\text{minimum spacing:} \begin{cases} 
\text{traverse spacing} = 4\phi \rightarrow = 3" \\
\text{vertical cover} = 2" \\
\text{penetration into the concrete slab} = 2" \\
\text{longitudinal spacing} = 4\phi \rightarrow = 4.5" \text{ according to AISC}
\end{cases}
\]

Shear stud Ø3/4" are used in double rows with 4.5" longitudinal spacing and 3.5" traverse spacing

**Design for shear**

Nominal shear capacity of the stiffened web 6.10.9.3:

At the strength limit state, straight and curved web panels shall satisfy:

\[V_u \leq \phi_v V_n\]

Where:

\[\phi_v = \text{Resistance factor for shear} = 1.0, \text{ specified in Article 6.5.4.2.}\]
\( V_n = \text{Nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively (kip).} \)

\( V_u = \text{Shear in the web at the section under consideration due to the factored loads (kip).} \)

\[
I f \quad \frac{2D t_w}{(b_{fc} t_{fc} + b_{ft} t_{ft})} \leq 2.5 \quad \rightarrow \quad V_n = V_p \left[ C + \frac{0.87(1 - C)}{\sqrt{1 + \left(\frac{a}{D}\right)^2}} \right]
\]

\[
\text{Otherwise} \quad V_n = V_p \left[ C + \frac{0.87(1 - C)}{\sqrt{1 + \left(\frac{a}{D}\right)^2 + \left(\frac{a}{D}\right)}} \right]
\]

\( V_p = 0.58 F_{yw} D t_w \)

Where:

\( a = \text{Transverse stiffener spacing (in.).} \)

\( V_n = \text{Nominal shear resistance of the web panel (kip).} \)

\( V_p = \text{Plastic shear force (kip).} \)

\( C = \text{Ratio of the shear-buckling resistance to the shear yield strength.} \)

The ratio, \( C \), shall be determined as specified below:

\[
I f \quad \frac{D}{t_w} \leq 1.12 \sqrt{\frac{E_k}{F_{yw}}} \quad \text{then} \quad \rightarrow \quad C = 1.0
\]

\[
I f \quad 1.12 \sqrt{\frac{E_k}{F_{yw}}} < \frac{D}{t_w} \leq 1.4 \sqrt{\frac{E_k}{F_{yw}}} \quad \text{then} \quad \rightarrow \quad C = \frac{1.12}{D} \sqrt{\frac{E_k}{F_{yw}}}
\]
If \( \frac{D}{t_w} > 1.4 \sqrt{\frac{E_k}{F_{yw}}} \) then \( C = \frac{1.57}{(\frac{D}{t_w})^2} \left( \frac{E_k}{F_{yw}} \right) \)

\( k = \text{Shear} - \text{buckling coefficient} \)

\[ k = 5 + \frac{5}{\left( \frac{a}{D} \right)^2} \]

\[ \frac{2Dt_w}{(b_{fc}t_{fc} + b_{ft}t_{ft})} \leq 2.5 \rightarrow \frac{2(23)\left( \frac{3}{16} \right)}{[(7)(0.5) + (7)(0.5)]} = 1.23 < 2.5 \rightarrow V_n \]

\[ V_p = V_p \left[ C + \frac{0.87(1-C)}{\sqrt{1 + \left( \frac{a}{D} \right)^2}} \right] \]

\( k = 5 + \frac{5}{\left( \frac{a}{D} \right)^2} \rightarrow k = 5 + \frac{5}{(1.5)^2} \rightarrow k = 7.22 \)

\[ \frac{D}{t_w} > 1.4 \sqrt{\frac{E_k}{F_{yw}}} \rightarrow \left( \frac{23}{3} \frac{3}{16} \right) = 122.67 > 1.4 \sqrt{\frac{(29000)(7.22)}{50}} = 90.6 \rightarrow C \]

\[ C = \frac{1.57}{(122.67)^2} \left( \frac{(29000)(7.22)}{50} \right) \rightarrow C = 0.437 \]

\[ V_p = 0.58(50)(23)\left( \frac{3}{16} \right) \rightarrow V_p = 125.1 \text{ kips} \]

\[ V_{cr} = CV_p \rightarrow 0.437(125.1) \rightarrow V_{cr} = 54.7 \]
 Traverse stiffeners 6.10.11.1.1:

The width (bt) of each projecting stiffener element shall satisfy:

\[
 b_t \begin{cases} 
 \geq 2.0 + \frac{D}{30} \\
 \geq \frac{b_t}{4} \\
 \leq 16t_p 
\end{cases}
\]

\[ b_t \geq 2.0 + \frac{23}{30} \rightarrow b_t \geq 2.76 \]

\[ b_t \geq \frac{7}{4} \rightarrow b_t \geq 1.75 \]

Try double stiffeners \( b_t = 6.0 \) (3" for each side of the web), and \( t_p = \frac{3}{8} " \)

\[ b_t \leq 16t_p \rightarrow 3.5 < 16\left(\frac{3}{8}\right) = 6 \text{ OK} \]

Check moment of inertia of the stiffener

\[ I_t \geq I_{t1} & \& I_{t2} \]

\[ I_{t1} = b \ t_w^3 I \]

Where:
\[ J = \frac{2.5}{(a/D)^2} - 2.0 \geq 0.5 \Rightarrow \frac{2.5}{1.5^2} - 2.0 = -0.9 < 0.5 \Rightarrow J = 0.5 \]

\[ b = \text{smaller of } \begin{cases} a \rightarrow 34.5" \\ \frac{a}{D} \rightarrow 23" \end{cases} \Rightarrow b = 23" \]

\[ I_{t_1} = b t_w^3 J \Rightarrow (23)(\frac{3}{16})^3 (0.5) \Rightarrow I_{t_1} = 0.076 \ (in^4) \]

\[ I_{t_2} = \frac{D^4 P_t^{1.3}}{40} \left( \frac{F_{yw}}{E} \right)^{1.5} \]

\[ P_t = \text{larger of } \begin{cases} \frac{F_{yw}}{F_{cr}} \\ 1.0 \end{cases} \]

\[ F_{cr} = \frac{0.31E}{b_t^2 t_p^2} \leq F_{ys} \Rightarrow \frac{(0.31)(29000)}{(6/0.5)^2} \leq 50 \Rightarrow 250 \text{ ksi} > 50 \text{ ksi} \Rightarrow F_{cr} = 50 \text{ ksi} \]

\[ P_t = \text{larger of } \begin{cases} 50 \\ 1.0 \end{cases} \Rightarrow P_t = 1.0 \]

\[ I_{t_2} = \frac{23^4 1.0^{1.3}}{40} \left( \frac{50}{29000} \right)^{1.5} \Rightarrow I_{t_2} = 0.5 \ (in^4) \]

\[ I_t = \frac{t_p b_t^3}{12} \Rightarrow \frac{3}{8} (6)^3 \Rightarrow I_t = 6.75 \ (in^4) \]

Since \( I_{t_2} > I_{t_1} \) then the following should be satisfied:

\[ I_t \geq I_{t_1} + (I_{t_2} - I_{t_1}) \left( \frac{V_u - \phi_v V_{cr}}{\phi_v V_n - \phi_v V_{cr}} \right) \]
$I_t \geq 0.076 + (0.5 - 0.076) \left( \frac{105 - (1)(54.7)}{(1)(93.9) - (1)(54.7)} \right) \rightarrow 6.75 > 0.6 \text{ OK}$

Use double stiffeners with $b_t = 6 \text{ in}$ and $t_p = \frac{3}{8} \text{ in}$

Bearing stiffeners 6.10.11.2.2:

**Bearing stiffener under the actuator (see Figure A.2)**

*The width, $b_t$ of each projecting stiffener element shall satisfy:*

*Try bearing stiffeners with $b_t = 6.0 \text{ in}$ and $t_p = \frac{5}{8} \text{ in}$*

\[ b_t \leq 0.48t_p \frac{E}{F_{ys}} \rightarrow 6 < 0.48 \left( \frac{5}{8} \right) \sqrt{\frac{29000}{50}} = 7.22 \text{ OK} \]

The factored bearing resistance for the fitted ends of bearing stiffeners shall be taken as:

\[ (R_{sb})_r = \varphi_b (R_{sb})_n \]

Where:

$(R_{sb})_n = \text{Nominal bearing resistance for the fitted ends of bearing stiffeners (kip)}$.

$\varphi_b = \text{Resistance factor for bearing =1.0, specified in Article 6.5.4.2.}$

$A_{pn} = \text{Area of the projecting elements of the stiffener outside of the web-to-flange fillet}$

$\text{welds but not beyond the edge of the flange (in}^2\text{).}$

$F_{ys} = \text{Specified minimum yield strength of the stiffener (ksi).}$

\[ (R_{sb})_n = 1.4A_{pn}F_{ys} \]

\[ A_{pn} = l_{pn}t_p \]

\[ l_{pn} = b_t - 2S_{max} - t_w \]
\[ S_{\text{max}} = \text{max welding size of fillet weld} \]

\[ t_w = \frac{3}{16} < \frac{1}{4} \implies \begin{cases} S_{\text{min}} = \frac{1}{8} \text{ in} \\ S_{\text{max}} = \frac{3}{16} \text{ in} \end{cases} \]

\[ l_{pn} = 6 - 2\left(\frac{3}{16}\right) - \frac{3}{16} \implies l_{pn} = 5.44 \text{ in} \]

\[ A_{pn} = (5.44)\left(\frac{5}{8}\right) \implies A_{pn} = 3.4 \text{ in}^2 \]

\( (R_{sb})_n = 1.4(3.4)(50) \implies (R_{sb})_n = 238 \text{ kips} \)

\( (R_{sb})_r = 1.0 \times 238 \implies (R_{sb})_r = 238 \text{ kips} > 2V_u = 210 \text{ kips (from analysis) OK} \)

**Bearing stiffener at the supports**

Try bearing stiffeners with \( b_t = 6.0 \text{ in and } t_p = \frac{3}{8} \text{ in} \)

\[ A_{pn} = (5.44)\left(\frac{3}{8}\right) \implies A_{pn} = 2.04 \text{ in}^2 \]

\( (R_{sb})_n = 1.4(2.04)(50) \implies (R_{sb})_n = 142.8 \text{ kips} \)

\( (R_{sb})_r = 1.0 \times 142.8 \implies (R_{sb})_r = 142.8 \text{ kips} > V_u = 105 \text{ kips (from analysis) OK} \)

**Axial resistance of bearing stiffeners (at support and at actuator) has to be checked per 6.10.11.2.4**

The factored resistance of components in compression, \( P_r \), shall be taken as:

\[ P_r = \varphi_c P_n \]

Where:

\( P_n = \text{Nominal compressive resistance as specified in Articles 6.9.4 or 6.9.5, as applicable (kip)} \)

\( \varphi_c = \text{Resistance factor for compression =0.9 as specified in Article 6.5.4.2} \)

Only consider flexural buckling for bearing stiffeners based on 6.9.4.1.1
\[ P_n = \begin{cases} 0.658 \left( \frac{P_o}{P_e} \right) P_o & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ 0.877 P_e & \text{if } \frac{P_e}{P_o} < 0.44 \end{cases} \]

\[ P_e = \frac{\pi^2 E}{(kl)^2} A_g \]

\[ P_o = Q F_y A_g \]

Where:

- \( A_g \) = Gross cross-sectional area of the member (in\(^2\)).
- \( F_y \) = Specified minimum yield strength (ksi).
- \( P_e \) = Elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional buckling or flexural-torsional buckling, as applicable (kips).
- \( P_o \) = Equivalent nominal yield resistance (kips).
- \( Q \) = Slender element reduction factor determined as specified in Article 6.9.4.2. \( Q \) shall be taken equal to 1.0 for bearing stiffeners.
- \( k_l \) = Effective length for bearing stiffener.
- \( r \) = Radius of gyration.

Note: to compute \( A_g \), \( I \), and \( r \), the effective section is to be used (see Figure A.3)
Bearing stiffener at the actuator

\[ A_y = 2 \left( \frac{5}{8} \right) (3) + \left( \frac{3}{16} \right) (2(1.6875) + \frac{5}{8}) \]

\[ A_y = 4.5 \text{ (in}^2\text{)} \]

\[ l_s = \frac{\left( \frac{5}{8} \right) (6 + \frac{3}{16})^3}{12} \rightarrow l_s = 12.337 \text{ (in}^4\text{)} \]

\[ r_s = \sqrt{\frac{12.337}{4.5}} \rightarrow r_s = 2.74 \text{ (in)} \]

\[ P_e = \frac{\pi^2(29000)}{\left( \frac{0.75(23)}{2.74} \right)^2 4.5} \]

\[ P_e = 32496.24 \text{ kips} \]

\[ P_o = 1.0(50)(4.5) \rightarrow P_o = 255 \text{ kips} \]

\[ \frac{P_e}{P_o} = \frac{32496.24}{255} = 127.4 > 0.44 \]

\[ P_n = \left[ 0.658\left( \frac{1}{127.4} \right) \right] 255 \rightarrow P_n \]

\[ = 254 \text{ kips} \]

\[ P_r = 0.9(254) = 228.6 \text{ kips} \]

\[ P_r = 228.6 > 210 \text{ kips at actuator} \]

OK

Bearing stiffener at the supports

\[ A_y = 2 \left( \frac{3}{8} \right) (3) + \left( \frac{3}{16} \right) (2(1.6875) + \frac{3}{8}) \]

\[ A_y = 2.953 \text{ (in}^2\text{)} \]

\[ l_s = \frac{\left( \frac{3}{8} \right) (6 + \frac{3}{16})^3}{12} \rightarrow l_s = 7.403 \text{ (in}^4\text{)} \]

\[ r_s = \sqrt{\frac{7.403}{2.953}} \rightarrow r_s = 1.583 \text{ (in)} \]

\[ P_e = \frac{\pi^2(29000)}{\left( \frac{0.75(23)}{1.583} \right)^2 2.953} \]

\[ P_e = 7117.8 \text{ kips} \]

\[ P_o = 1.0(50)(2.95) \rightarrow P_o = 147.65 \text{ kips} \]

\[ \frac{P_e}{P_o} = \frac{7117.8}{147.65} = 48.2 > 0.44 \]

\[ P_n = \left[ 0.658\left( \frac{1}{48.2} \right) \right] 147.65 \rightarrow P_n \]

\[ = 146.4 \text{ kips} \]

\[ P_r = 0.9(146.4) = 131.74 \text{ kips} \]

\[ P_r = 131.74 > 105 \text{ kips at support} \]

OK

OK

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Design of welding (see Figure A.4)

\[ t_w = \frac{3}{16} < \frac{1}{4} \rightarrow \begin{cases} \frac{1}{8} \text{in} \\ \frac{3}{16} \text{in} \end{cases} \]

Use fillet welding size \((S) = 3/16"\)

\[ L_w = D - 2S_w - 2S \]

\[ S_w \geq 4t_w \rightarrow \frac{3}{16} \rightarrow S_w \geq 0.75\text{(in)} \]

\[ S_w \leq \text{lesser of} \begin{cases} \frac{6t_w}{4} \text{(in)} \end{cases} \]

\[ S_w \leq \text{lesser of} \begin{cases} \frac{6t_w}{4} \rightarrow 1.125 \text{(in)} \end{cases} \]

\[ 0.75 \text{(in)} \leq S_w \leq 1.125\text{(in)} \]

use \(S_w = 1.0\) (in)

\[ L_w = 23 - 2(1.0) - 2\left(\frac{3}{16}\right) \rightarrow L_w = 20.8 \text{(in)} \]

\[ R_r = 0.6\phi_{e2}F_{exx} \quad \text{factored shear resistance of the weld metal} \]

\[ R_r = 0.6(0.8)(70) \rightarrow R_r = 33.6 \text{ ksi} \]

\[ \mu = R_r(0.707)(S) \quad \text{shear resistance of the fillet weld} \]

\[ \mu = (33.6)(0.707)\left(\frac{3}{16}\right) \rightarrow \mu = 4.45 \frac{kips}{\text{in}} \]
Total factored shear resistance of thye welding = 4(4.45)(20.8)

\[ = 370.2 \text{kips} > 210 \text{kips (at actuator)} \text{ and } 105 \text{kips (at supports)} \text{ OK} \]

Welding details and fabrication of steel girders used in fire tests is shown in Figure A.5.
Figure A.1: Layout of steel girder G3 used in fire test

Figure A.2: Steel section with bearing stiffener
Effective section at actuator per 6.10.11.2.4

Effective section at support per 6.10.11.2.4

Figure A.3: Effective section of bearing stiffeners

Figure A.4: Welding details for bearing stiffener
Figure A.5: Welding details and fabrication of steel girders used in fire tests
Appendix B

Governing Equations for Structural Analysis

The main equations governing the force-displacement history in a structural member (system) can be obtained by minimizing the total potential energy in the member. The total potential energy ($\pi$) for the steel girder that is shown in Figure B.1 can be expressed as:

$$\pi = \frac{1}{2} \int_v \varepsilon_x \sigma_x \, dv - \int_L \nu^T q \, dx \quad \text{.................................................................} \quad [B.1]$$

where;

$\sigma_x$ = normal stresses,  $\varepsilon_x$ = normal strain ($\varepsilon_{\text{mechanical}}+\varepsilon_{\text{thermal}}+\varepsilon_{\text{creep}}$),  $v$ = the displacement at any point, and $q$ = the applied surface traction (distributed load).

In the displacement based finite element approach, the displacement is assumed to have unknown values only at the nodal points, so that, the variation within any element is described in terms of the nodal values by means of interpolation (or shape) functions. The steel girder is discretized using typical beam element shown in Figure B.2, the displacement (deflection) of the member can be expressed as:

$$v = \begin{bmatrix} N_1 & N_2 & N_3 & N_4 \end{bmatrix} \begin{bmatrix} v_1 \\ \theta_1 \\ v_2 \\ \theta_2 \end{bmatrix} \Rightarrow v = N.d_e \quad \text{.................................................................} \quad [B.2]$$

where;

$N_1$, $N_2$, $N_3$, and $N_4$ are interpolation functions, which are also termed the shape functions and are given by:
\[ N_1 = 1 - \frac{3x^2}{L^2} + \frac{2x^3}{L^3} \] ................................................................. [B.3a]

\[ N_2 = \frac{x}{L} - \frac{2x^2}{L^2} + \frac{x^3}{L^3} \] ................................................................. [B.3b]

\[ N_3 = \frac{3x^2}{L^2} - \frac{2x^3}{L^3} \] ................................................................. [B.3c]

\[ N_4 = -\frac{x^2}{L^2} + \frac{x^3}{L^2} \] ................................................................. [B.3d]

\( v_1, \Theta_1, v_2, \Theta_2 \) are vertical displacements and rotations at each nodes, while \( d_e \) is vector of nodal displacements (degree of freedom) of the element.

Stress in the element can be expressed in terms of bending moment as follows:

\[ \sigma = \frac{M_y}{I} \] ................................................................. [B.4]

Stress is related to the strain by Hooke’s law, as follows:

\[ \sigma = E_{(T)} \varepsilon \] ................................................................. [B.5]

under fire conditions, modulus of elasticity degrades with temperature raise in the member. Therefore the modulus of elasticity \( E_{(T)} \) is a function of temperature (fire exposure time).

The total potential energy of the continuum (or any elastic structure) will be the sum of the energy contributions of the individual elements. Thus,

\[ \pi = \sum \pi_e \] ................................................................. [B.6]

where, \( (\pi_e) \) represents the total potential energy of elements \( (e) \).

For simplification, the steel girder is discretized to one element. Therefore, the total potential energy in the member can be rewritten as:
\[ \pi_e = \frac{1}{2} \int_{L} \int_{A} \frac{M^2}{E(T)} I^2 y^2 \, dA \, dx - \int_{L} \left[ N^T \right]^T [d]^T q \, dx \] \quad \text{[B.7]} \]

where, \( M = E(T) I \frac{d^2 y}{dx^2} \) and \( \int y^2 \, dA = I \) \quad \text{[B.8]}

\[ \pi_e = \frac{1}{2} E(T) I \int_{L} \left( \frac{d^2 y}{dx^2} \right)^2 \, dx - \int_{L} [N]^T [d]^T q \, dx \] \quad \text{[B.9]}

\[ \frac{d^2 y}{dx^2} = \begin{bmatrix} d_1^2 \frac{d^2}{dx^2} N_1 & d_2^2 \frac{d^2}{dx^2} N_2 & d_3^2 \frac{d^2}{dx^2} N_3 & d_4^2 \frac{d^2}{dx^2} N_4 \end{bmatrix} [d] \Rightarrow [B][d] \quad \text{[B.10]} \]

\[ \pi_e = \frac{1}{2} E(T) I \int_{L} [B]^T [d]^T [B] \, dx - \int_{L} [N]^T [d]^T q \, dx \] \quad \text{[B.11]}

Minimization of potential energy of element (e) can be done by differentiating the total potential energy with respect to nodal displacements (\( d^e \)) in the element and as follows:

\[ \frac{\partial \pi_e}{\partial d_e} = \frac{1}{2} E(T) I \int_{L} [B]^T [B] \, dx - \int_{L} [N]^T [d]^T q \, dx = 0 \] \quad \text{[B.12]}

\[ K_e d_e - F_e = 0 \] \quad \text{[B.13]}

where:

\( F_e \) = the equivalent nodal forces for the element (\( F_e^n \)). For restraint members under fire condition, \( F_e \) will be the summation of the nodal forces and the fire induced restraint forces (\( F_{e \text{th}} \)).

\( K_e \) = termed the element stiffness matrix, given by

\[ K_e = \int_{L} [B]^T E(T) I [B] \, dx \] \quad \text{[B.14]}

[B] matrix can be obtained by differentiating the shape functions two times with respect to x and as given in Eq. [B.10] and this result in:
\[ B = \begin{bmatrix} \frac{6}{L^2} + \frac{12x}{L^3} & - \frac{4}{L} + \frac{6x}{L^2} & \frac{6}{L^3} - \frac{12x}{L^2} & - \frac{2}{L} + \frac{6x}{L^2} \end{bmatrix} \]

\[ K_e = E_{(x)_L} \begin{bmatrix} \frac{12}{L^3} & \frac{6}{L^2} & - \frac{12}{L^2} & \frac{6}{L^2} \\ \frac{6}{L^2} & - \frac{L}{L^3} & - \frac{L}{L^3} & \frac{2}{L^2} \\ - \frac{L}{L^3} & - \frac{L}{L^3} & - \frac{L}{L^3} & - \frac{L}{L} \\ \frac{6}{L^3} & \frac{2}{L^2} & - \frac{6}{L^2} & \frac{4}{L} \end{bmatrix} \]

and

\[ F_e = \int_0^L \begin{bmatrix} N_1 \\ N_2 \\ N_3 \\ N_4 \end{bmatrix} q \, dx \]

where, \( q = \) the applied surface traction (distributed load)

The summation of the terms in Eq. [B.14] over all the elements (in case of using more than one element), when made equal to zero, results in a system of equilibrium equations for the complete continuum. These equations are then solved by applying any standard technique to obtain the nodal displacements (Hinton and Owen, 1980).
Figure B.1: Typical steel bridge girder with boundary conditions

Figure B.2: Typical beam element
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