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Assessment of Bridge-Structures Under Fired Impact: A Case Study Approach

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ASSESSMENT OF BRIDGE-STRUCTURES UNDER FIRE IMPACT: A CASE
STUDY APPROACH
BY
JAN SCHUMACHER

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
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MASTER OF SCIENCE THESIS
OF
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ABSTRACT

This master thesis deals with the behavior of steel girder highway bridges under small vehicle fires. Fire at and below highway bridges are a common problem for bridge structures. Unlike in tunnels or buildings the consideration of fire hazard is no mandatory aspect of bridge design. However, the high amount of fire incidents on highways and the important function that bridges have in the infrastructure resulted in a few case studies to explain the failure process of a bridge. All of these case studies deal with collapses of the bridge structure caused by gasoline tanker fires.

The behavior of steel girder bridges under small vehicle fires without a collapse of a bridge is not included in any case study yet. This thesis shall establish understanding of the post-fire behavior of a steel girder under a vehicle fire with several boundary conditions. A special topic in this thesis is the analysis of a micro-crack included in the steel girder. Micro-cracks can be caused by fatigue or by the fabrication of the girder among other things. This is also a first attempt to consider the condition of a highway bridge in the analysis of a steel girder bridge under a fire impact. The fire impacts used in this thesis contains temperatures less than hydrocarbon fires (maximum temperature of 1100°C), that are used to represent gasoline tanker fires, but higher temperatures than external fires which are not caused by fuel or gas (maximum temperature of 600°C - 700°C).

The case study provided in this thesis is carried out using the finite element software Abaqus. The structural model of the chosen hypothetical highway bridge is a two span beam. To simplify the numerical model just one steel girder connected to a concrete deck instead of a complete bridge is used. This offers detailed modeling of the structure and the consideration of many possible mechanisms of the structure. The numerical model considers a variation of the fire impact, the

fire location, horizontal restrains and the number of spans of the bridge.

The results of the case study can be transferred to a complete highway bridge.

The results can also be transferred to different static models of bridges.

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CHAPTER 1

Introduction and research question

Bridges are an important part of the infrastructure all around the world. They are often located at highly frequented parts of the road system, making it possible to pass rivers, cross streets and pass valleys. It is well-known that bridges are vulnerable structures. Because of their function and the very high building costs, bridges are built to last for several decades [16]. Engineers dedicate themselves to design bridges that promise a long lifetime.

Due to the fact that bridges are very vulnerable to dynamic and static fatigue the European Code provides a set of rules about how to consider the influence of traffic load, wind, snow, earthquakes and collisions especially for bridges [17, 18, 19, 20, 21]. Although many fire incidents happened in the last years fire hazards for bridges are not amongst the regulated cases covered by the European Code and thus are not necessarily part of the considerations while designing a bridge [16, 22]. Fire protection is an important factor in many fields of civil engineering. The DIN EN 1991-2 indicates the fire protection for buildings. There are many fire regulations to follow while planning and designing a building.

Fire hazard prevention measures for bridges are not mentioned in any of the DIN NORM regulations, although the type of the fire and the possible effects on a bridge differ greatly from the ones on buildings [9]. The typical types of bridge fires are hydrocarbon and petrol fires. These fire events can reach much higher temperatures than most building fires. Hydrocarbon fires can reach 1100°C - 1200°C. Also the typical temperature profiles of bridge fires are different to building fires. Typical building fires need some time to develop and to reach the maximum temperature. So there is a good chance that the fire brigade can

fight the fire before the maximum temperature is reached. Hydrocarbon-fires and gasoline-fires reach their maximum temperature very fast. So the fire is often fully developed before the first attempts to extinguish it have been made. Fire on or under bridges can have many different reasons. The most common reason are collisions of vehicles. There are two different kinds of collisions: Either two vehicles crash into each other or a vehicle crashes into parts of the bridge, for example columns or guide boards. Also the kind of the vehicle is an important factor for the size of a fire. Some of the major fire incidents of the last 15 years were caused by gasoline tanker crashes [22].

Another reason for fire incidents on bridges can be electrical problems, which happened e.g. at the 'Big Four Bridge in Louisville' in 2008. Also vandalism and sabotage can lead to fire incidents. Another possible reason for a bridge fire is a flash-over from a fire next to a bridge.

The type of the parts vulnerable to fire in buildings and bridges differ greatly. The girders of a bridge have to span a bigger length. Because of that they are often much higher than girders in buildings. By losing strength, the web of a big girder is more vulnerable to buckle [9]. The condition of the bridge after the fire can also vary highly. The fact that fire hazards for bridges are an under-researched topic leads to a lack of information regarding the post-fire strength of the different parts of a bridge [9, 23]. Good knowledge of the post-fire strength is necessary in order to develop the right repair strategies after a fire incident. Unlike buildings there are no fire protections on bridges such as sprinkler systems or fire extinguishers [9].

An important, yet under-researched factor that determines the behavior of a bridge during a fire event is the pre-fire condition of the bridge. It is difficult to research the effects of the condition of the bridge. This aspect affects mostly

local failures. During the long lifetime of a bridge micro or even bigger cracks are no exception. The behavior of these cracks during a fire event has not yet been included in any case study. Therefore, this master thesis deals with a variation of small vehicle fires and the post fire condition of a steel girder as part of a highway bridge. The influence of the different boundary conditions on the serviceability of the bridge after the fire event is an important part of this thesis. Also the behavior of already existing micro cracks in the steel girder during the fire event is part of the research. The aim is to find conclusive answers to the question of how fire events affect the structure of a bridge by using an inclusive research method that might offer the possibility to generalize.

The thesis is divided in different parts. First, the state of the art of the topic and the importance of the study will be presented followed by the general methodology of the case study. Chapter 2 discusses the general behavior of bridges during fire events. A special aspect of the master thesis is the fracture analysis in a finite element software. Therefore, the theoretical background of micro cracks in steel girders is discussed in chapter 3. Another part of the thesis explains all the important aspects of the case study, in order to estimate the results of the case study correctly. This also enables the transfer of this case study to similar problems. The main part of the thesis are the results of the case study and the conclusion of the results. The thesis concludes by revisiting the results and connecting them in order to identify the major findings.

1.1 Previous fire incidents on bridges and state of the art

There have been some major fire incidents of bridges in the last 15 years. Most of the well-known fire incidents led to a total collapse of the bridge. Total collapses induced by fire events trigger bigger public attention. But also smaller fire events can lead to enormous damage of the bridge without resulting in a complete

collapse. These fire incidents are more difficult to list because they occur more often but create less public interest.

Among other authors, Maria E. Moreyra Garlock discussed some major bridge fire incidents in the USA in her papers (Review and Assessment of Fire Hazard in Bridges (2010) and Fire Hazard in bridges: Review, assessment and repair strategies (2011)) [22, 24]. Her research states that many of the big bridge fires are caused by gasoline tankers. A gasoline fire on July 15th, 2009 lead to the collapse of a bridge that crossed the Interstate 75 near Detroit, Michigan. The tanker that crashed into the bridge carried approximately 46,800 liter of gas and 14,400 liter of diesel [22]. The bridge structure collapsed completely within 30 minutes.

A similar accident happened in Oakland, California on April 29th, 2007. A gasoline tanker carried 30,960 liter of gasoline and overturned on the I-80 - I-880 interchange. The very high temperature of approximately 1100°C lead to the collapse of some of the steel girders of the bridge. The bridge structure resisted 22 minutes before it collapsed [22].

The third bridge fire Maria E. Moreyra Garlock et al discussed in their paper Review and Assessment of Fire Hazard in Bridges (2011), occurred on July 28th, 2006 in Arizona at the Bill Williams River Bridge. A tanker truck overturned while on top of the bridge and 27,360 liter of gasoline caught fire. The fire was located on and also under the bridge. Most parts of the bridge survived the fire but major repairs were necessary.

Recently, similar bridge fires have occurred. On July 1st, 2015 a truck overturned and 36,000 liter of ethanol caught fire. The bridge fire took place in Columbus, Ohio. The fire department had the fire under control one hour after the gasoline started burning. The bridge structure did not collapse but damage of approximately \$1 million was caused [1]. The bridge fire in Columbus illustrates

a common scenario: The fire usually affects only one span of a multi-span-bridge.

The bridge structure didn't collapse because of the fire, but major damage was caused. The high resistance of the bridge depends greatly on the material of the bridge which in this case is concrete. Concrete resists high temperatures much better than steel girders do.

Because of a few case studies there is a the general lack of information about the bridge performances under fire impact [22]. There are two types of case studies: The first type of case studies is made about real fire incidents in the past. The scientists try to evaluate special fire incidents and the reason for the collapses or damage of the affected bridges [25].

The second type of case studies are about hypothetical bridges. The material and the structural system is chosen by the author and analyzed according to an estimated impact of fire. Kodur et al. (2013) chose to analyze a theoretical steel girder instead of a whole bridge [16]. This focuses the analysis on the specific effects of fire on chosen parts. Also the amount and the duration of the fire impact can be pre-defined by the scientists [16, 9].

Although different fire curves are included in these case studies, almost all of them are based on an accident of a gasoline tanker or something similar. Small vehicle fires without a subsequent collapse of the bridge have been neglected so far. The high amount of vehicle fires and the high percentage of passenger cars on the highways point out the importance of smaller vehicle fires.

The 'National highway traffic safety administration' gives an overview of all fatal vehicle crashes for every year, including the average number of accidents of vehicles with bridge elements with and without following fire incidents [26]. It indicates that an average of 37 bridge collisions with following fire incidents occur every year in the US. But collisions are just one of many possible reasons for fire

incidents at bridges. It is difficult to get a reliable number of fire incidents under or on bridges [26]. Nonetheless, several authors have mentioned specific numbers in their respective publications. Payá-Zaforteza and M.E.M. Garlock collected the information in a survey of 18 US-American states in the year 2011 [9]. The survey collected data related to 1746 bridge failures. The result is that 54 of these bridge collapses were caused by fire. This means three times more collapses were caused by fire than by earthquakes, even though states with high seismic risks such as California were part of the survey [9].

Maria Garlock et al. (2011) included the survey of 18 US-American states and increased their data base by using the survey by Battelle [27, 22]. The survey of Battelle stated that the aggregated number of vehicle fires on highways in the USA is 376,000 per year. In total, these fire incidents lead to 1.28 billion-dollar worth of property losses and 570 civilian deaths [27]. The survey gives no indication of how many bridges are affected by vehicle fires. But the high amount of vehicle fires on highways and the equally high number of bridges on highways show the hazard that vehicle fires pose to bridges.

Kumalasari Wardhana and Fabian C. Hadipriono deal with this topic in their paper, 'Analysis of recent bridge failures in the United States' [28]. The paper is based on different reports of US-American departments. But neither the National Bridge Inventory nor the New York Department of Transportation (NYDOT) released a list with all bridge failures that included the reasons of failure. The paper by Kurnalasari Warhana and Fabian C. Hadipriono is based on a survey of bridge failures from 1989 to 2000 found in the NYDOT database [28]. They determined that 16 of the 503 bridge failures are caused by fire.

Fires usually occur as a result of unrelated incidents, such as earthquakes and collisions, than may have an impact on bridges. Materials for bridges can be wood,

steel, concrete or a combination of these materials. Wood is the only flammable material, meaning even if the fire source is not part of the wooden structure it will still continue to burn once caught fire. Because there are no fire safety measures, the fire is usually highly developed when fire fighting forces arrive at the place of the incident. Thus, in many cases wooden bridges that have caught fire burn to the ground. Bridges or parts of bridges that are made of concrete or steel are not flammable. Depending on the boundary conditions, the characteristics of the bridge after the fire event can vary highly.

The review of the state of the art shows the importance of this topic, but also a lack of research and information about small vehicle fires and the post fire behavior of bridge structures loaded by these fires, thus illustrating the necessity of this thesis.

1.2 Numerical methods

Choosing the right numerical method for the intended purpose is crucial for every case study. The numerical model has to be well-thought-out. All important mechanical and thermal mechanisms and all boundary conditions have to be included in the numerical model.

As mentioned before, the lack of information is reflected by the many individual and unconnected case studies. These case studies differ according to their characteristics and boundary conditions. Nonetheless, the methodological procedure is similar for every case study. Since the exact fire event is unknown in many case studies, different possible fire incidents have to be considered [23].

To evaluate the residual strength of the bridge parts, a finite element based software is needed. In this context, the software programs 'Ansys' or 'Abaqus' are the most frequently used [23, 9]. This case study is based on the finite element software Abaqus which offers the possibility to analyze the steel girder under different

boundary conditions. Abaqus was chosen because it has all necessary analyzing instruments.

Some parts of the numerical model can only be used by Abaqus and are not transferable but most of the important parts of the numerical model are universally valid. The analysis includes two kinds of problems: thermal and mechanical. Two different sets of discretization models are needed one for the thermal and another for the mechanical strength analysis.

The basics of the discretization for the thermal analysis is independent from the finite element software which is used. Heat transfer calculations are needed to evaluate the temperature in the structure. The elements have to be three-dimensional elements with a thermal conduction capability and they need a single degree of freedom: the temperature [23, 9]. The elements have to be able to be integrated in a three dimensional steady-state or transient thermal analysis [29, 30]. The surface of the elements should be able to simulate the convection and radiation of a real surface. These elements need, among others, the emissivity factor and the convection co-efficient as input values. These values can vary along the cross-section of the members [23].

The mechanical strength analysis requires a different type of discretization of the elements. The discretization of the elements depends not only on the type of the analysis but also on the materials. Steel and concrete need different types of elements. Steel elements need six degrees of freedom and three translations in the x-, y- and z- direction [29, 30]. With these degrees of freedom local buckling in flanges and webs and lateral torsional buckling of girders is displayable. The elements with these characteristics are well-suited for large rotation, large strain and non-linear problems [23].

Concrete elements perform in a different way than steel structures do and

the discretization of concrete elements for the structural analysis differ from steel structures. Concrete slabs in bridge structures have basically three degrees of freedom: translations in x,y and z directions. The characteristics of the elements for the numerical model shall allow the numerical model to represent cracking of concrete in tension, crushing of concrete in compression, creep and large strains. The boundary conditions such as the support conditions vary in every case and are very important for a successful analysis.

Three stages of the analysis are necessary to evaluate the residual strength. The first step is to analyze the bridge in ambient conditions. This is necessary in order to determine the results of the next steps of the analysis. At the first step, a mechanical strength analysis is needed. The normal properties of the materials are also valid in this case - such as the typical stress-strain model of steel [23].

The goal of the second step of the analysis is to determine the performance of the bridge during the exposure to the fire impact. In this step of the analysis the thermal and the stress analysis are required. Although all impacts on the bridge occur at the same time; by using the software, the thermal analysis takes place first. The results of the thermal analysis are applied as thermal-body-load on the structural model. The thermal-body-loads are consistent along the structure [23]. The properties of steel and concrete change during the heating, which is described in detail in chapter 2.1.

The third and last step of the analysis is to determine the fire-exposed steel girder after cooling [23]. The characteristics of the materials after cooling are unlike the properties before the fire occurred. The change of the properties depends highly on the maximum temperature during the fire event. Chapter 2.1 states that the configuration of the materials can change when reaching certain temperatures. The yield strength of carbon steel decreases by $0.3 \frac{\text{MPa}}{^{\circ}\text{C}}$ when the temperature of the steel

reaches 600°C or more during the heating progress [31]. The residual compressive strength of concrete is 10 % less than the minimum compressive strength during the fire event. To model these changed properties in the finite element software is complicated, due to the fact that the maximum temperature of every node of the model varies and thus the properties for the different areas of the structure vary as well.

It is important to define failure criteria by analyzing bridges with finite element software. For bridges with big steel girders the buckling of the web often is a failure criterion. The resistance of the web of a steel girder depends on the stiffness of the vulnerable parts. The fire impact leads to a reduction of the stiffness of the fire-exposed parts [23]. How much of the web-stiffness is left, needs to be analyzed. It is possible that the girder becomes unstable under its own weight or if it is loaded by traffic loads after a fire incident [9]. Another possible failure criterion is the appearance of fractures. This happens if the ultimate strain of $\epsilon_u = 0.2\%$ is attained [9]. This mode can be checked by using the module; 'Damage for Traction Separation Laws' in Abaqus or by checking the strain results of the stress analysis [9]. The impact of the condition of the bridge on the failure due to local fractures is still unclear.

CHAPTER 2

Performance of bridges under high temperatures

The performance of a bridge during a fire event is influenced by many different aspects. The characteristics of the fire have the most obvious impact on the behavior of a bridge [22]. The quality and the quantity of the fire event affect the performance of the bridge significantly. The maximum temperature and the time of the fire can vary highly depending on the amount and the type of the fire load.

The behavior of the bridge is not only related to the impact of the fire, but also to the characteristics of the bridge, such as the geometrical features or the material of the bridge. One of the most important aspects is the structural system and the span length of the bridge [22]. Statically determined and indeterminate systems behave very differently upon fire impacts. Therefore, statically determinate and indeterminate systems are included in the numerical model of this thesis. Big bridges are mostly build as statically indeterminate systems which has multiple advantages. The structural systems differ a lot and so do the internal forces of the bridge. Also the length of the main parts influences the amount of the deformation a lot. In addition to the length of the spans, the number of lanes is also a critical factor for the behavior of the bridge [22].

The material of the bridge is an important factor of the performance during a fire event. It is obvious that a bridge which is made of wood performs in a different way than concrete or steel bridges do. The recent fire incidents clearly show that fire impacts often burn wood bridges down. Steel and concrete bridges on the other hand often retain residual strength after fire events [26].

All these aspects illustrate the complexity of trying to develop general rules for bridges regarding fire events. Due to that, individual case studies have to be

developed for every single project. This leads to a huge expenses of time and money. V.K.R. Kodur and M.Z. Naser tried to investigate an 'Importance factor for design of bridges against fire hazard' in 2013 [16]. The idea was to classify bridges in relation to the fire hazard. As already mentioned, many aspects affect the behavior of a bridge in a fire event. Kodur and Naser created a rating system for different characteristics of bridges that are important for the behavior during fire events. The factors are grouped in five classes.

The first class includes geometrical features, material properties and design characteristics. These include e. g. the age of the bridge, although it is questionable whether the age can estimate the exact condition of the bridge. The second class deals with the fire hazard likelihood. Since vehicle fires are one of the main sources of fire hazards for bridges, the third class refers to the traffic demand of the bridge. Class 4 is about economic consequences. This class includes for example the expected time and cost that is needed to repair the bridge. The fifth and last class includes the expected losses such as life/ property losses and environmental damages [16].

Although the classification system of V.K.R. Kodur and M.Z. Naser has its faults especially regarding the fact that some categories are rather difficult to identify and classify, it remains a solid first attempt to generalize safety measures for bridges in order to prevent fire damages.

2.1 Materials fire behavior

The properties of structural steel vary highly according to its temperature. The correlation of the property and the temperature is non-linear and different for the respective features of the structural steel. In order to analyze the impact a fire has on steel, several temperature-dependent characteristics are considered. For the stress analysis the strength-strain-relationship of the structural steel is needed.

The conductivity, the specific heat and the expansion of the steel are important properties for the transient heat transfer analysis. The density of the steel is the only property which is not temperature related.

The stress-strain curve of steel can be divided into an elastic and a partially plastic part. The partially plastic part of the curve includes the yielding, the strain hardening and the necking phase. The general stress-strain curve is shown in figure (1).

Figure 1. General stress-strain curve of carbon steel [2]

The characteristic stress-strain curve of structural steel depends on the temperature. The stress-strain curves in Figure (2) indicate that the yield-strength of the material decreases but the maximum strain is on the same level at all temperatures. At higher temperatures, the characteristic properties of the stress-strain curve are non-existent anymore [3]. The upper yield-peak and the plastic plateau are gone and the plastic coating occurs during a longer strain section. This behavior of structural steel under high temperatures leads to a variable modulus of elasticity. The modulus of elasticity at ambient temperature is constant as long as the yield-strength-peak is not reached.

Figure 2. Variation of stress-strain correlation of structural steel at different temperatures [3]

The different parts of the stress-strain curve are not clearly specifiable under high temperatures. Because of the variability of the modulus of elasticity under high temperatures, the comparison of the modulus of elasticity at different temperatures has to take place in the first section of the stress-strain curve. The correlation of the Young-modulus of the structural steel and the temperature is shown in figure (3).

Figure 3. Variation of the Young-Modulus of structural steel at different temperatures [4]

This diagram shows the decreased values for the Young-modulus of steel at temperatures from 20°C to 1200°C. Structural steel starts to melt beyond 1200°C. These diagrams indicate how risky fire events for steel members in bridges are. Under certain boundary conditions, gasoline fires can reach temperatures of 1200°C and more. The temperatures in the steel members depend on the boundary conditions and are not generally calculable. A temperature of 1000°C leads to a yield-strength of under 10 %.

Steel shows plastic behavior after the yield strength of the material is reached. The relationship of the yield strength and the temperature is shown in figure (4). The yield point of steel starts to decrease at 400°C. At 1200°C the yield strength is equal to zero, but the values are already very low at 800°C.

Figure 4. Variation of the yield strength of structural steel at different temperatures [4]

All these different aspects affect the performance of the steel members in a bridge. Even when the live loads of a bridge usually do not exist during a fire event the dead loads can cause failure of steel members. Also the properties of structural steel that are important for the transient heat transfer analysis, depend highly on the temperature. The relationship of the conductivity and the temperature for carbon steel is defined by the Eurocode 3, part 1-2. The conductivity decreases when temperatures increase and is limited at 800°C to $27.3 \frac{W}{m \cdot K}$. The correlation

of the conductivity of carbon steel in dependency of temperature is shown in figure (5).

Figure 5. Conductivity of carbon steel in dependency of temperature [4]

In contrast to the steady state analysis the specific heat of the structural steel is needed in the transient heat analysis. The correlation of the specific heat of carbon steel and high temperatures are defined by different functions for the several temperatures; the curve is shown in figure (6). The maximum of the specific heat is at 735°C.

Figure 6. Specific heat of carbon steel in dependency of temperature [4]

Simple bridges are normally constructed in a way, that thermal expansion does not lead to additional stresses. Expansion joints have been included in the bridge structure in order to absorb expansion movements of the bridge generated

by seasonal temperature differences. The expansion caused by fire is much higher, thus the limit of the expansion joints is reached and the structural system of the bridge changes. Details of elastomer bearings are discussed in chapter 4.5.1. The correlation of the expansion of the structural steel and the temperature is indicated through a linear function and a constant value of 600°C to 735°C. The relationship of the thermal expansion and the temperature is shown in figure (7).

Figure 7. Thermal expansion of carbon steel in dependency of temperature [4]

Next to steel, concrete is the most used material in the construction of bridges. Concrete is mainly used in railroad slabs, especially for highway bridges. Concrete structures react completely different to fire than steel members. Concrete has a higher resistance to fire than steel or wood [6]. Concrete is nonflammable and does not conduct heat like steel does. Concrete consists of a mixture of different materials. Alongside the basic materials such as water, cement and aggregate, there are many different concrete additives, such as superplasticizer.

Very high temperatures can change the chemical composition of the concrete. When concrete members are exposed to fire and the temperature reaches 100°C, the pressure in the inside of the material increases [6] and the uncombined water in the structure vapors, but the properties of the material are almost constant. The cement components in the concrete change when the temperature increases

to more than 400°C [32].

The hydroxide in the cement starts to dehydrate and leads to a significant loss of strength and also to a reduction of the modulus of elasticity [5, 6]. The significant loss of strength amounts to 30% – 40% [33]. The 'Technical Note - NO. 102' (2011) analyzes the effects of high temperature fire on concrete components [33]. In addition to the reduction of the compressive strength, various degrees of spalling and reduction of the modulus of elasticity, micro-cracking within the concrete micro-structure, color changes, loss of bond between concrete and steel and a possible loss of residual strength of steel reinforcement and possible loss of tension in prestressing tendons are mentioned [33].

At temperatures higher than 1200°C various components of concrete start to melt. But such temperatures are rare even at gasoline fires. The chemical processes caused by the thermal stress change the properties of the concrete structure. After cooling, the structure remains changed and the performance of the concrete member is not predictable anymore.

Similar to the behavior of structural steel, the behavior of concrete members is nonlinear during heating. During a fire event concrete slabs of bridges are mostly stressed on one side of the member. The temperature difference of two borders of the cross section leads to bending and additional deflections. The stress-strain-diagram changes under high temperatures. Concrete is a brittle material under normal temperatures. When the temperature rise, the maximum stress decreases but the strain increases. The material becomes more ductile under increasing temperatures and the deflections become bigger; this behavior is different to the behavior of structural steel. Mr. C Sangluaia et al. developed a diagram based on the European Code 3 part 1-2 [4].

As mentioned before, concrete is less heat-conducting than steel. Usually

concrete slabs of a bridge are very deep. When unidirectional thermal stress occurs, the temperature differences along the cross-section of the concrete members are very high. The European Code 1 part 1-2 offers diagrams to ensure the fire safety of buildings [18]. These diagrams describe the temperature profile of a concrete member. The temperature profile depends on the geometric form of the concrete member and on how many parts of the shape are affected by flames. There are many different diagrams provided because of the different boundary conditions. Unidirectional thermal stress is the most common fire hazard for concrete slabs in bridge design. A possible diagram that analyzes a concrete slab under fire safety aspects is shown in figure 8. The labels of the lines in the profile identify the different fire safety levels of the European Code. The letter 'R' indicates the fire resistance of the concrete and the number indicates the time in minutes of how long this member resists the fire under ambient conditions.

Figure 8. Temperature profile under one-sided fire impact [7]

The concrete slab is only included in the transient heat transfer analysis but not in the stress analysis in this case study. It is common in highway bridge design to plan for the steel girders to bear the dead loads of the entire bridge although this depends on the strategy of the construction of the bridge. If the bridge is built in place and the steel girders are placed first and the concrete hardens afterwards without additional construction support, the steel girders are loaded with the dead load of the concrete slab in addition to their own dead load. The structure bears the live loads as a composite member but the dead loads are carried by the steel girder. For the heat transfer analysis the concrete slab is important. The temperature of the upper flange of the steel girder depends on the behavior of the concrete slab during fire.

Therefore the conductivity of the concrete is needed. The European Code specifies two curves for the conductivity of normal weight concrete. These two curves offer the upper and the lower limit of the conductivity. The exact value can be determined by every country separately. The two curves are shown in figure (9).

Figure 9. Thermal conductivity of normal weight concrete in dependency of temperature [7]

The specific heat of concrete in average is higher than the specific heat of steel. The value of the specific heat increases during higher temperatures until it

reaches the maximum value of $1100 \frac{J \cdot K}{kg}$ at $400^{\circ}C$. This is indicated in figure (10).

Figure 10. Specific heat of normal concrete in dependency of the temperature [7]

2.2 Post-fire assessment and repair

The damage of the bridge caused by a fire has to be calculated with a finite element based software. The results of the analysis have to be evaluated in order to develop repair strategies. The aim of the repair strategies is to guarantee the service ability of the bridge [24]. Structural steel is vulnerable to very high temperatures. Abrupt heating and cooling can change some of the properties of structural steel [34]. Engineers have tried to analyze the impact of a fire event on a bridge by using a finite element based software and through visual evaluation. Non-destructive and destructive testing is required as well [34].

The problem of non-destructive and destructive testing techniques after fire incidents is the classification of the results. Usually there is no data of a special part of a bridge before the fire occurs. The properties of a special steel grade vary for every steel member, so the exact properties of a steel member, and thus the exact properties of every member cannot be generalized. Instead, additional testing is required. A solution for this lack of data could be to test some parts of the steel members outside of the fire zone [34].

The easiest way to evaluate the condition of a bridge after a fire is a visual evaluation. Some issues such as damaged steel connections can be detected without further investigation [24]. The repair strategy depends on the material and the intensity of the damage. Given that concrete has a superior fire resistance in contrast to other construction materials such as timber and steel, it may be efficient to use in-situ repair options. Steel and timber constructions have to be replaced often after a big fire event [33].

To repair damaged concrete structures, different repair materials are available. Small damages such as cracks can be repaired by patching products or epoxy. These products are approved if they meet the requirements of the 'Specifications for Concrete Patching Materials' [35]. Similar requirements are given for the force of sealers. Sealers have to meet the requirements of the 'Specifications for Concrete Sealers'. In case of big damage to the structure, patching products are not usable and concrete has to be added. It is important that the new concrete conforms to the original material. The detailed mix design of the new concrete has to be approved by the engineer in charge [35]. It is also possible to re-establish the bond of the steel and the concrete with concrete bonding agents.

Reinforcing or pre-stressed steel can also be replaced and the replacement has to meet the requirements of G30.18 and accordingly ASTM 416 [35]. Also, the procedure for the restoration of reinforced and pre-stressed concrete girders is determined by the 'Repair Manual for Concrete Bridge Elements' of Alberta Infrastructure and Transportation [35]. The aim of a repair strategy is usually to restore the original condition of the bridge. As mentioned before, the repair strategy is designed according to the extent of the damage. A successful repair comprises of different steps. The first step is the removal of unsound concrete by using basic hand tools [35, 36]. This repairs has to be undertaken very carefully in

order to conserve the sound areas of the bridge. After that, the reinforced and pre-stressed steel has to be checked and in case of damage it has to be straightened or replaced. Additional steel might have to be inserted. The splicing and re-stressing of pre-stressed strands is again an important and rather difficult step needed in order to re-establish the original condition of the bridge [36]. The 'Repair Manual for Concrete Bridge Elements' provides a guideline for this procedure [35]. After the reinforced and pre-stressed steel is repaired and the concrete restoration area is brush-blasted, the mentioned repair strategies can take place. New installed concrete needs formwork for 28 days to develop its properties. After the new concrete is installed smaller issues can be fixed. The crack repair areas should be grounded smoothly in order to match the surrounding concrete and achieve a uniform surface [35].

After the repairs is completed, the structure has to be tested to make sure it was successful. As already mentioned, the damage assessment and repair strategies for steel structures are more difficult than for concrete structures. The fire affected steel structure has to be investigated after cooling. When the structure is straight and without any distortions, the steel member is probably still useable [37, 31]. Due to extensive research, the resistance strength of the heated steel member is well-known, but the properties of the steel structure after cooling depend on the boundary conditions. For a precise evaluation of the resistance strength of the steel structure a detailed analysis is needed. Some hardness tests such as Brinell or Vickers have been performed to analyze the structure [38]. The correlation of the hardness numbers and the ultimate tensile strength of steel is given in figure (11).

The result of the hardness test has to be compared to the data given in figure 11. The range of the acceptable difference of the two values is ten percent [8]. But

	Brinell Hardness Number	Vickers Hardness Number	Ultimate Tensile Strength N/mm ²
Grades S355	187	197	637
	179	189	608
	170	179	559
Grades S275	163	172	539
	156	165	530
	149	157	500
Grades S235	143	150	481
	137	144	481
	131	138	461
	126	133	451
	121	127	431

Figure 11. Brinell and Vickers hardness numbers with equivalent ultimate tensile strength values [8]

even if the tested materials pass the test, they might not be reusable [8]. A general guideline as given for concrete structures is not available for steel structures.

Every bridge has to be analyzed individually and a structural engineer has to decide what level of transformation of the steel structure is acceptable [8, 37, 31].

2.3 Typical fire impacts on bridges

Many different reasons can lead to bridge fires, thus many fire curves are possible. As described in chapter 1.1 the most common type of major bridge fires in the last years are hydrocarbon and gasoline fires.

There were many different fire models used in the past. Some of the fire curves, which relate to possible bridge fires are mentioned below. The fire curves are limited to 60 minutes, although the fire events can last much longer. The time the fire lasts depends on different boundary conditions, the most important one is the amount of the flammable material. The described fire incidents in chapter 1.1 show, that the time of the fire event can vary highly. Steel girders of a bridge

structure usually collapse before the 60 minutes, regardless of the type of the fire.

The first fire model to discuss is the 'Cellulosic Fire Curve'. Although the 'Cellulosic Fire Curve' is mainly developed for building fires, it was also included in fire analyses of tunnels and bridges. It is also part of the DIN 4102 in Europe [39]. The curve is displayed in figure (12). Because this fire curve is developed for building fires, it does not develop as fast as the other fire curves, such as hydrocarbon fire curves. After 60 minutes the curve still increases and the maximum has not yet been reached. The maximum temperature in the first 60 minutes is approximately at 945°C [39].

Figure 12. Cellulosic Fire Curve

The most used fire curve for bridge fire incidents is the 'Hydrocarbon Fire Curve'. This fire curve is provided by the European Code 1 part 1-2 [18]. The 'Hydrocarbon Fire Curve' is provided for small petroleum fires, such as car fuel tanks, petrol or oil tankers [39]. The 'Hydrocarbon Fire Curve' develops much faster than the 'Cellulosic Fire Curve'. The maximum temperature of 1100°C is reached after 60 minutes, the 'Hydrocarbon Fire Curve' reaches the maximum temperature of the 'Cellulosic Fire Curve' of 100°C after five minutes. Because of the unlimited availability of oxygen at bridge fires, 'Hydrocarbon Fire Curves' have no cooling phase and the duration of the fire is theoretically unlimited. It

can only be stopped by inflammable material and through active fire protection. The 'Hydrocarbon Fire Curve' is pictured in figure (13).

Figure 13. Hydrocarbon Fire Curve

The regulations in the European Code can be modified by every European country. In the case of the 'Hydrocarbon Fire Curve', most countries adopt the suggestions of the universal European Code. However, the French regulations asked for a modified version of the 'Hydrocarbon Fire Curve' [39]. The result of the request is a similar fire curve with a higher maximum. In the 'Hydrocarbon Fire Curve' the maximum temperature increases from 1100°C to 1300°C. The 'Modified Hydrocarbon Fire Curve' is shown in figure (14). This 'Modified Hydrocarbon Fire Curve' is primarily developed for concrete structures [39]. The idea that the fire load varies due to the material of the bridge is questionable. As described in chapter 2.1 the properties of steel are defined for temperatures up to 1200°C. Higher temperatures can lead to a complete loss of strength of the steel material, the exact temperature depends on the carbon content of the steel.

The 'RABT-ZTV Curve' was developed in Germany. The basis for the 'RABT-ZTV Curve' are many different test programs such as the 'Eureka Project' [40]. In these studies cars and train fires were tested and the results are collected in different databases [41]. The studies were basically made to analyze fire incidents

Figure 14. Modified Hydrocarbon Fire Curve

in tunnels. The boundary conditions vary from tunnel fires to bridge fires, thus they are not perfectly applicable for bridge fire incidents.

The curves for car and train fires differ in the length of the fire event. The maximum temperature of 1200°C is identical for both cases. The 'RABT-ZTV Curve' for cars starts to decrease after 30 minutes, the 'RABT-ZTV Curve' for trains decreases after 60 minutes. The curves are shown in figure (15).

Figure 15. RABT-ZTV Fire Curve

Also the Rijkswaterstaat, Ministry of Transport in the Netherlands, developed a fire model. The 'RWS Fire Curve' is based on a 50,000 liter fuel, oil or petrol tanker fire. The fire load of the tanker fire is estimated to be 300 MW and the

fire event lasts up to 120 minutes. The experiment was carried out by TNO in the Netherlands in 1979 [39]. The maximum temperature of the 'RWS Fire' is 1350°C after 60 minutes. This fire model is also used for tunnel fire studies. The temperatures for bridge fires would be lower because of the possible distribution of the temperatures at a bridge fire. Also the amount of 50,000 liter considered in the experiment is very high. The fire incidences mentioned in section 2.1, included a maximum load of approximately 27,000 liter fuel. The 'RWS Fire Curve' is represented in figure (16).

Figure 16. RWS (Rijkswaterstaat) curve

An experiment was carried out especially for bridge fires in the year 2004 by Richard B. Stoddard [42]. In this experiment a railroad tanker loaded with 108,000 liter methanol was investigated. The research is based on an actual fire in the year 2002. The 'Stoddard's Fire Curve' differs highly from the 'Hydrocarbon Fire Curve' of the European Code. The maximum temperature of 1500°C is very high and the development of the temperature is not as fast as in the 'Hydrocarbon Fire Curve'. The maximum temperature is reached after 30 minutes. The 'Stoddard's Fire Curve' also has a distinct cooling phase. The temperature falls under 540°C after 2 hours [42]. I. Payá-Zaforteza and M.E.M. Garlock compare the 'Stoddard's Fire Curve' with the 'Hydrocarbon Fire Curve' and a standard building fire in

their paper 'A numerical investigation on the fire response of a steel girder bridge' [9]. The result of the comparison was, that the 'Stoddard's Fire Curve' is less severe than the 'Hydrocarbon Fire Curve'. The faster rising temperatures in the 'Hydrocarbon Fire Curve' lead to higher damage [9].

CHAPTER 3

Micro-cracks in steel girders

As mentioned, bridges are an important part of the infrastructure. The variation, of the loading on a bridge lead to a dynamic impact on bridges. Thus bridges are very vulnerable to fatigue cracking. Steel members are also vulnerable to rust as they are exposed to the daily weather and salt or chemicals. Although bridges are designed to last a long time, many bridges already show static or construction problems after a short time. Deficient bridges lead to big financial losses, disturbed traffic and pose a danger to their surroundings.

Knowing this, the US-American states' Departments of Transportation try to monitor their respective bridges. The departments developed a consistent rating system for bridges. The 'National Bridge Inspection Standards' define condition ratings in order to define the condition of an existing bridge [43] which can be described with a code from 0 to 9. The definition of each code is shown in figure (17). The exact calculation of the different 'Bridge Condition Rating Categories' is given by the Federal Highway Administration of the U.S. Department of Transportation [10].

Rating	Condition Category	Description *
9	Excellent	
8	Very Good	No problems noted.
7	Good	Some minor problems.
6	Satisfactory	Structural elements show some minor deterioration.
5	Fair	All primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	Poor	Advanced section loss, deterioration, spalling, or scour.
3	Serious	Loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	Imminent Failure	Major deterioration or section loss present in critical structural components, or obvious loss present in critical structural components, or obvious vertical or horizontal movement affecting structural stability. Bridge is closed to traffic, but corrective action may be sufficient to put the bridge back in light service.
0	Failed	Bridge is out of service and is beyond corrective action.

Figure 17. Bridge Condition Rating Categories [10]

Summary rating results for all bridges in the USA are published online by the 'Federal Highway Administration' [11]. The 'FHWA' categorizes the bridges in this report in:

- Total number of bridges
- Number of structurally deficient bridges
- Number of functionally obsolete bridges
- Number of deficient bridges

Structurally deficient bridges are rated in condition 4 (poor condition) or less. Bridges which are rated as structurally deficient have major issues and have been restricted to light vehicles, require rehabilitation or are completely closed for traffic. Structurally deficient bridges have parts that need to be monitored or repaired [43]. A bridge can be labeled as functionally obsolete if it was built according to outdated formalities. They are not necessarily structurally deficient or unsafe [43]. E.g., a bridge can be labeled as functionally obsolete if the lane widths is less than the actual standards. It is also possible that the bridge is designed for traffic loads that are lower than the traffic loads today. The label 'deficient bridges' combines the number of structurally deficient and the number of functionally obsolete bridges.

The list of the bridges also includes the information about the state and the age of the different bridges. Bridges with a length of 20 feet or less are not part of the 'National Bridge Inspections' (NBI) and thus, are not part of the statistics mentioned below. The tables get updated frequently. The survey provides a total amount of 611,844 bridges in the USA. 58,791 of these bridges are structurally deficient and 84,121 bridges are labeled as functionally obsolete, thus the structurally deficient and functionally obsolete bridges represent 23.4% of all bridges.

The classification of all bridges in the USA, which are part of the NBI, is shown in figure (18).

Figure 18. Total amount of bridges in the 'United States' and their classification (compare [11])

To understand the reasons for the high amount of functionally obsolete and structurally deficient bridges a more detailed statistic is necessary. The subdivision of the bridges in sections of ten years each, clarify the distribution of the bridges with a condition of 4 or less. Figure (19) shows the amounts of the different classifications of bridges more detailed. The total number of bridges is subdivided in sections of 10 years. Diagram (19) indicates that the biggest group of bridges is between 51 to 60 years old. 50% of the bridges are 41 years or older.

Figure 19. Total amount, structural deficient and functionally obsolete bridges under consideration of their age (compare [11])

The figure (20) shows the vulnerability of bridges to fatigue more clearly. The numbers in percent for each period indicate the increasing vulnerability of older bridges. Only 10 % of the new bridges, build from 2006 to 2015 are deficient. The amount of deficient bridges which are 41 to 50 years old is more than double that number. More than 30 % of the bridges, 51 to 60 years old, are deficient. Even 40 % deficient bridges are indicated for bridges which are 81 to 90 years old. The bridges that are older than 90 years old again represent higher values.

Figure 20. Proportional amount of structural deficient, functionally obsolete and deficient bridges by the age of the bridge (compare [11])

The results of the data provided by the 'Federal Highway Administration' show increasing percentages of deficient bridges by increasing age of the bridges. It is also indicated that the volume of new bridge structures vary highly. The fatigue problem will become even worse in the future, when the big number of bridges, built from 1956 to now, become older. Unfortunately the survey does not specify the different types of bridges.

The bad condition of bridges is not only a problem in the USA. In many parts of the industrial world the massive expansion of infrastructure including the amount of bridges took place in the first half of the 20th century [44]. A survey in Europe states that 70% of the steel railway bridges are 50 years old and about 30% of these bridges are more than 100 years old [44]. Unfortunately, it is really difficult

to consider the vulnerability of bridges to fatigue in a numerical model, although these problems are well-known. Every bridge is different and so are the types and the places of fatigue failures. In all case studies so far no fatigue influences such as small fatigue cracks and no cross-section loss caused by rust have been included.

Especially the web-gab area of the girder at the girder-floor beam is vulnerable for fatigue cracking [45]. The size of fatigue cracks varies. However, visible cracks of a length more than 1 cm are not very likely to be present at a possible fire event. These cracks would supposedly have been detected and repaired quickly after they appeared. The probability of smaller cracks, for example cracks with a length of 1 mm in steel beams is higher. These cracks would also grow without a fire, but the fire speeds up that process significantly. The fire impact causes decreasing strength of the steel material thus the crack grows much faster. The 'Federal Highway Administration' of the U.S. Department of Transportation provides a 'Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges' [12]. This manual provides an example of a fatigue crack in the web area of a steel girder [12].

3.1 Fracture analysis

This thesis deals with the growing of an already existing crack. The contact properties of the crack and the steel beam define the growing of the crack. Several scientists developed strategies to analyze cracks.

Alan Arnold Griffith developed the Griffith Critical Energy Release Rate (G_c) to determine the failure stress (σ_f). The basis of Griffith's theory is the definition of the Work (W) given in equation (1).

$$W = \int F dx \quad (1)$$

By multiplying and dividing the equation (1) with the volume (V) the work is

defined in terms of stress and strain. The calculation is shown in equation (2).

$$W = \int F dx * \left(\frac{V}{AL} \right) = \int \sigma d\epsilon * V \quad (2)$$

$$U = W * V = \int \sigma d\epsilon = \int E * \epsilon d\epsilon = \frac{1}{2} * E * \epsilon^2 = \frac{\epsilon^2}{2 * E} \quad (3)$$

Bog McGinty, PhD, describes the bond energy of a material [13]. If the energy, which is introduced in an object, overcomes the bond energy, a crack is going to grow. The calculation of the bond energy, E_{bond} , is given in equation (4). The value γ_s describes the energy required to destroy the atomic bonds per unit of the surface area. The value a represents the length of the crack and B the thickness of the body [13].

$$E_{bond} = 2 * \gamma_s * a * B \quad (4)$$

The strain release energy of an un-cracked body is given by equation (5).

$$U = \frac{\sigma^2}{2 * E} * V \quad (5)$$

A crack would decrease the strain energy release. The impact of the crack on this energy depends on the length a of the crack. Griffith calculated the resulting strain energy release with the equation (6) [13].

$$U = \frac{\sigma^2}{2 * E} * V - \frac{\sigma^2}{2 * E} * B * \pi * a^2 \quad (6)$$

The equation (6) indicates that the energy decreases non-linearly. The length of the crack has a quadratic influence on the strain energy release. Thus the energy loss at short crack lengths is small, but it becomes much more significant at longer lengths [13]. The total energy in the system is now given by the combination of equation (4) and (6), given in equation (7).

$$U = \frac{\sigma^2}{2 * E} * V - \frac{\sigma^2}{2 * E} * B * \pi * a^2 \quad (7)$$

Bog Mc Ginty visualized the bonding energy, the strain energy and the sum of both. The energy values are pictured in dependency of the crack length. The sum energy function has an increasing trend for short crack lengths. Thus the crack is stable if no more energy is added into the material. The energy values start to decrease after a certain crack length. After this point the crack is unstable and it grows without any additional external energy [13].

Differentiating the equation (7) with respect to the crack length a and solving the result for the critical stress σ_s leads to equation (8).

$$\sigma_f = \sqrt{\frac{G_c * E}{\pi * a}} \quad (8)$$

$$G_c = 2 * \gamma_s \quad (9)$$

The critical energy release rate G_c is the input value for the crack analysis in Abaqus. The critical energy release rate can be calculated by solving the equation (8) for G_c (equation (11)). Also the stress intensity Factor K_c (equation (10)), introduced by G. R. Irwin, can be included in the equation for the critical energy release rate G_c [13].

$$K_c = \sigma_f * \sqrt{\pi * a} \quad (10)$$

$$G_c = \frac{K_c^2}{E} \quad (11)$$

CHAPTER 4

Numerical model

The goal of the case study is to analyze a typical highway bridge structure during a fire event. Although highway bridges show similarities, exceptional bridge designs sometimes occur due to special circumstances. For example lakes or railroads next to the highway can require longer span lengths, the height of bridges depends essentially on the landscape of the area the bridge is planned in. The span length and the height are not the only characteristic parameters of a highway bridge. The structural model of the bridge is also very important for the bridge behavior under fire impact. Usually there are no aesthetical requirements for the design of highway bridges. The efficiency of the bridge is much more important. Thus T-beam constructions are the most common shape for highway bridges. Very often the slab is made out of concrete; orthotropic decks made of steel are more common for pedestrian bridges or smaller vehicle streets. The girders of the bridge can be made of steel or concrete. Due to the fact that steel is much more vulnerable to fire impacts than concrete, steel girders are included in the numerical model of the highway bridge.

The typical construction of a T-beam section and its impact on the interaction of the steel girder and the concrete slab are discussed in section 4.2.

4.1 Chosen fire impact

Most of the case studies published in the last years dealt with a worst case scenario. The analyzed fires were most likely caused by fuel tanker accidents. This is due to the high impact and damage of these incidents. Case studies dealing with the worst case scenario aim to calculate the time before failure of the bridge or to reproduce a real bridge collapse to understand the mechanism of the failure.

More likely than the worst case scenario is a fire scenario with a smaller fire impact. These smaller fire incidents can be caused e.g. by normal vehicles instead of fuel tankers. Although vehicle fires lead to lower maximum temperatures than fuel tankers, these fires are also able to cause significant damage to the bridge. Vehicle fires do not last as long as tanker fires due to the limited amount of fuel. While tanker fires usually lead to a collapse of the bridge a survival of the bridge after vehicle fires is more likely.

The hydrocarbon fire curve provided by the 'European Code' does not include a cooling phase. Thus the time period of the fire has to be determined by the author of the case study. This applies to most of the fire curves provided in chapter 2.3. The concentration of the case studies on tanker fires lead to a lack of information about vehicle fires. The 'RABT-ZTV curve' for vehicles is based on experimental data of a burning vehicle in a tunnel. Although the boundary conditions of a tunnel fire vary from the boundary conditions of a bridge fire the 'RABT-ZTV curve' represents a vehicle fire under a bridge best of all fire curves discussed in chapter 2.3. The curve includes a heating phase, a fully developed fire phase and a cooling phase. In this case study different fire data based on the 'RABT-ZTV fire' curve are used. The fire data varies according to the duration of the fully developed fire phase and the maximum temperature. Assuming that the open air conditions of a vehicle fire on a highway affect the temperatures highly, the values of the temperature in the numerical model are lower than the fire data of the 'RABT-ZTV curve'. The first included fire curve has the modification factor $\eta = 0.75$. The cooling phase of the fire curve starts after $t = 15 \text{ min}$. The second fire curve has the same maximum temperature as the first one but the cooling phase starts five minutes later. The third and the fourth fire curve have a modification factor for the maximum temperature of $\eta = 0.8$. The third fire curve takes again fifteen

and the fourth fire curve twenty minutes before the cooling phase starts. The four different fire curves are shown in figure (21). The numbers for the different fire events shown in the figure are used during the further parts of the case study.

Figure 21. Modification curves (1,2,3 and 4) of the RABT-ZTV curve for the numerical model

4.2 Structural model and location of the fire

A typical structural model for a highway bridge is a two span continuous T-beam. The length of the spans has a decisive impact on the behavior of a bridge during fire. It is common that both spans of the bridge have the same length. The middle support of the bridge is located between the two directions of the highway.

In this case study two lanes for each direction are assumed. The 'U.S. Department of Transportation' and the 'American Association of State Highway and Transportation Officials' regulate the lane width of highways [46]. The lane width in general can vary from 2.7 m to 3.6 m. The most modern highways have a lane width of 3.6 m [47]. The total span length is determined to be 10 m in this case study.

To outline the differences of the behavior between statically indeterminate

and determinate, a single span beam is also investigated. The length of the single span beam equates to the length of one of the two span beams. The shape and all boundary conditions are similar for both structural systems. For the results it is important to know that the stresses and the displacements of the single span beam are naturally higher than the stresses and the displacements of the two span beam. The results of the post-fire behavior have to be compared to the behavior of the particular structural system before the fire event. The two structural systems are pictured in the figures (22) and (23).

The location of a vehicle fire can vary highly. Also the size of the area of the heating source varies because of the spilled fuel after an accident. In this case study multiple fire scenarios are investigated. It is suggested that just the left span of the steel girder is affected by a car fire. The first scenario is a fire over the full length of the span of the bridge. This could be caused by a collision of multiple cars leading to a large fire. The second case represents a vehicle fire on the right lane of a highway, it starts at the middle of the left span and ends at the middle support of the bridge. The third location of the fire is the area from the left support to the middle of the left span of the bridge. A reason for the accident could be a crash of a car at the left lane. The length of the fire is almost 5 m, for the fire locations 2 and 3, and it is suggested that the fire has the same heat over the full length. This is an unrealistic case, because in reality the heat would vary along the length of the girder and there would be no strict boundaries at the end of the heating. But since no two fires are the same, going with a homogenic fire that has the same temperature throughout will still offer results that are transferable to other incidents. All surfaces of the steel girder that are accessible by the fire as well as the concrete deck are facing the same amount of heat. All fire locations and the two different structural models are shown in the figures (22) and (23). The

areas of the steel girder directly over the bearing area are not affected by the fire impact. It is assumed that these areas of the bridge are not reached by the flames. This assumption has a big impact on the bridge behavior during the fire event.

Figure 22. Two span continuous beam with different fire locations (position 1,2 and 3)

Figure 23. Single span girder with different fire locations (position 1,2 and 3)

In the stress analysis the steel girder is bearing the dead load of the model. The dead load includes the self-weight of the steel girder, the self-weight of the concrete and the weight of the equipment of the bridge. The load for the steel girder is assumed to be $p = 100 \frac{kN}{m^2}$ and is applied at the top flange of the steel girder. The structural models for the stress analysis with the applied loads are shown in figure (24).

The structural models of the one span and of the two span girder can also be modified by restraints in the longitudinal directions at the ends of the steel girder, the reasons for these modification is discussed in chapter 4.5.2. The steel girder used in this model is a rolled section of the shape W30x211. The girder is made of carbon steel A36. A girder made from 50 ksi steel would behave similar, although the load for the stress analysis would be different.

Figure 24. Loading of the one span girder (a) and the two span girder (b)

The concrete slab is made of normal weight concrete, the distance between the girders is assumed to be 3 m and the height of the slab is 30 cm. These dimensions and assumptions are based on different design examples for steel girder superstructure bridges provided by the 'Federal Highway Administration' [48]. The examples are used to estimate the dimensions of the concrete and the approximate size of the steel girder, the variation of the cross section along the longitudinal direction is neglected to simplify the numerical model. The comparison of the numerical model to the numerical model of Esam Aziz and Venkatesh Kodur shows similar ratios of the concrete dimensions to the steel girder. The dimensions used in this case study are higher than the dimensions of the cross section of the numerical model of Esam Aziz and Venkatesh Kodur [23], due to the bigger length of the girder used in this numerical model. The cross section of the steel girder and the concrete slab used in this case study are pictured in figure (25).

Figure 25. Steel girder W30x211 und concrete slab 3x0.3 m^2 used in this case study

4.3 Basic model parameter for the heat transfer analysis

The temperature impact can be implemented in the load module by using a tabular amplitude. The assignment of the fire impact on the structure can be done through the interaction module. The surface film condition and the surface radiation are used to simulate the interaction of the structure with the fire impact. The emissivity for the surface radiation is different for the materials concrete and steel. For the material carbon steel an emissivity factor of 0.75 and for concrete an emissivity factor of 0.85 is used in this case study. For the surface film condition, a film coefficient of 50 is used for both materials. This is given by the EC-1 for hydrocarbon fires [18] and is transferred to the 'RABT-ZTV fire curves'. The contact between the concrete and the steel member and between the steel stiffeners and the steel girder also have to be defined by an interaction to guarantee the heat convection between the different parts. The convection is defined by a thermal conductance using only clearance-dependency data.

To determine the heating of the whole cross section, reference points are used. Similar to the case study of Payá-Zartofeza three different reference points on the vertical symmetry are selected to represent the heating differences along the cross

section [9]. These reference points are used in chapter 5.2 to describe the heat transfer analysis. The positions of these reference points are shown in figure (26).

Figure 26. Representative points for the heat transfer analysis

4.4 Mesh of the model in Abaqus

The mesh and the discretization of the model are important parts of the analysis with a finite element software. The results of the analysis depend on the meshing of the different parts of the model. The optimum meshing is very fine at the areas of the biggest importance for the results and a coarse meshing at the less important areas to reduce the size of the model and calculation time. This idea is difficult to implement in the model, thus the different areas of the model have to be connected to each other sensibly and the elements must not be distorted. Parallel edges of the steel girder automatically have the same meshing. Thus a fine meshing for the whole model is used to guarantee accurate results.

A difficulty of the fracture analysis is the meshing of the model. The micro-crack requires a much finer mesh than the rest of the steel girder. The very small crack of the size of 1 mm has to be included in the steel girder and the meshing around the crack has to be even finer than these 1 mm to show the growth of the

crack in an appropriate way. The rest of the model is meshed with edge sizes of 4 cm. Thus the appropriate mesh size of the girder is approximately 100 times larger than the required mesh for the crack. The mesh for the crack cannot be applied to the whole model, this would exceed the computing capacity. But also the crack area cannot be meshed independently since the big difference of the mesh sizes would lead to highly distorted elements. Therefore some areas outside of the crack area have to be meshed very finely. The meshing of the model is shown in the figures (27).

Figure 27. Meshing of the steel girder, stiffener and concrete slab (a) and the meshing of the crack area as a part of the steel girder mesh (b)

For the stress analysis and the heat transfer analysis the mesh is exactly the same. To include the heat transfer analysis in the stress analysis it is important that every node has the same number in both analyses. The temperature results of the heat transfer analysis are assigned to the nodes of the stress analysis.

4.5 Boundary conditions

The boundary conditions influence the behavior of a bridge significantly. The structural system includes the most important boundary conditions, but the transformation into a numerical model and additional boundary conditions have to be discussed.

4.5.1 Elastomer support

The type of the bridge bearing has a big impact on the behavior of the bridge structure. I-girders of bridges are often supported by elastomeric bearing pads [49]. Elastomeric bearings offer stiffness in all directions. Displacements and rotations of the I-girder at the bearings depend on the stiffness of the elastomeric bearing pads in the particular direction. It is also possible to prevent movements in certain directions completely, except for the vertical direction. A big advantage of elastomeric bearings is the good replaceability of these bearings.

Elastomeric bearings consist of several rubber layers. These layers are bound by a reinforcement layer [14]. A typical construction of an elastomeric layer is shown in figure (28).

Figure 28. Structure of a Elastomeric Bearing [14]

This basic structure can be modified by different design alternatives. For example, the elastomeric bearings can be tapered by steel plates, also inclined mounting of bearings for special loadings is possible [15].

The most important stiffness exists in the longitudinal direction, the vertical direction and the rotation. The stiffness in longitudinal direction depends on the shear modulus (G_{eff}), the bearing area (A) and the total elastomer thickness (H_r). The equation to determine the stiffness in horizontal direction (K_H) is given in equation (12) [49].

$$K_H = \frac{G_{eff} * A}{H_r} \quad (12)$$

The calculation of the stiffness in vertical direction of the elastomeric bearing is similar. But in this case the effective Young-Modulus is used instead of the shear modulus. The equation (13) is given below and instead of the total elastomer thickness the elastomer bearing height ('H') is included in the equation. The calculation of the effective Young Modulus is given in equation (14), it depends on the numerical factor (k), the shape factor (S), which is defined as the loaded area divided by the force area, and the Young Modulus (E_0) [15].

$$K_H = \frac{E_c * A}{H} \quad (13)$$

$$E_c = E_0 * (1 + 2 * k * S^2) \quad (14)$$

The rotational stiffness (K_δ) is defined by the Young Modulus, the Elastomer moment of inertia (I) and again the total elastomer thickness ((15)) [49].

$$K_\delta = \frac{E_0 * I}{H_r} \quad (15)$$

The properties of elastomeric bearings are given in table (29). The properties are given for three different hardness values of the elastomeric material.

To simplify the numerical model the elastomer supports are not truly included: Instead of an extra part the properties of an elastomer bearing are transferred in springs. Theses springs are linked to a defined surface of the steel girder.

Hardness (Shore A)	Young's Modulus E_s	Shear Modulus G	k Factor
50	312 psi (2.2 MPa)	100 psi (0.68 MPa)	0.73
60	635 psi (4.4 MPa)	150 psi (1.04 MPa)	0.57
70	1040 psi (7.2 MPa)	245 psi (1.69 MPa)	0.53

Figure 29. Properties of elastomeric material [15]

The three displacements of the nodes of these surfaces are connected to the springs. Rotations cannot be restrained, the nodes are described via displacements and not via rotations.

4.5.2 Horizontal movement

To prevent stress caused by temperature changes, expansion joints are included in the bridge design. Expansion joints can be based on different technologies. For example the Watson Bowman Acme offers high performance silicone, elastomeric, armoless and joint seal expansion joint systems [50]. These expansion joints are developed for the seasonal temperature changes. The expansion joints are designed for the average temperature of 20°C (68°F) similar to the whole bridge design. The reference temperature can vary for different countries.

Expansion joints are installed with a defined joint opening. The minimum and the maximum joint openings can vary highly from model to model. The maximum horizontal movement under high temperatures is defined by the difference of the maximum joint opening and the joint opening before the fire occurred. The Watson Bowman Acme offers joints for bridges with maximum horizontal movements compared to the reference temperature of 120 mm (4.7 in) to 160 mm (6.3 in) [50]. The maximum possible movement of the expansion joint depends on the type of the joint and the model.

The general calculation of the expansion caused by plain temperature changes are given in equation (16). The heat conductivity of carbon steel for the normal

seasonal temperatures is included in the equation.

$$\Delta L = \alpha_T * \Delta T * L = 1.2 * 10^{-5} * \Delta T * L \quad (16)$$

The proportional expansion of a steel girder depends on the value of the temperature change and the length of the girder. Bridges are designed for temperature differences up to 30°C. The temperature change caused by a fire event can be forty times the seasonal temperature difference, thus the expansion is forty times the designed horizontal expansion. It is important to consider the fact that not the whole bridge is affected by the temperature change caused by a fire event. The temperature change included in the design of the bridge relates to the whole bridge structure. The length of the affected bridge is included in the equation (16) similar to the temperature change. The non-uniform heating over the length of the girder leads also to stresses at the transition zones. The horizontal expansion is not uninhibited anymore. The real horizontal expansion has to be analyzed in the model.

The comparison of the possible horizontal expansion in the expansion joints and the expansion caused by the fire lead to different boundary conditions during the fire event. After the maximum movement in the joint is reached, the structural model changes and a horizontal movement is not possible anymore. This affects the stresses and the displacements of the structure. Therefore the steel girder is tested with and without a possible expansion in longitudinal direction of the girder. I. Payá-Zaforteza and M.E.M. Garlock already outlined the importance of the consideration of the limits of expansion joints in their paper [9]. Accordingly, they included a rigid body in their model to limit the horizontal expansion. In this paper two different models are included to discuss the differences of a numerical model with and without horizontal movement.

4.6 Modeling of the materials

The principal properties for the materials are discussed in chapter 2.1. The steel girder for the numerical model is a W30x211 shape, made of A36 steel. The stiffeners are made of the same material as the steel girder. The density of the steel is $7850 \frac{kg}{m^3}$. Also important for the heat transfer analysis are the properties' conductivity and the specific heat; the values for the different temperatures were discussed in chapter 2.1.

For the stress analysis the stress-strain curve of the material controls the behavior of the structure. The general stress-strain curve and the variation of the stress-strain curve during heating is discussed in chapter 2.1. Thus the strain-hardening phase of the curve decreases under increasing temperatures, the stress-strain curve is modeled in Abaqus as ideal elastic/ ideal plastic. After the yield strength of the material is reached, the stress value is constant until the strain at failure is reached. The strain at failure is set to 20 % for every temperature. The stress-strain curves for the different temperatures are visualized in figure (30).

Figure 30. Stress-strain model of steel in Abaqus

Concrete is modeled with a density of $2500 \frac{kg}{m^3}$. The properties conductivity

and specific heat depend on the temperature and are explained in chapter 2.1.

4.7 Influence of the concrete deck on the structural model

As mentioned before, the concrete deck is only included in the transient heat transfer analysis. The upper side of the top flange of the steel girder is connected to the concrete slab and not directly loaded by the fire impact. Thus the upper side of the flange gets heated by the concrete slab. This implicates that the heating and the cooling process of the upper side of the top flange are delayed in time compared to the rest of the cross section. To model the heat transfer of the concrete slab correctly, the same properties are needed as for the modeling of the heat transfer analysis of the steel girder. After the heat transfer analysis the concrete slab is turned off in the analysis and the dead load of the concrete slab is applied on the top of the steel girder.

4.8 Crack analysis in Abaqus

The modeling of three-dimensional micro-cracks in Abaqus and the growing of these cracks have to take place in several modules. The crack has to be modeled as an additional part and included in the assembly. The material of the cracking member has to be amplified by damage for traction separation laws. This can be implemented, for example, through the maximum principal stress of the material. The crack can be created in the interaction module and the crack growing can be brought into action [30]. The properties of the material for the growth of the crack can also be implemented directly into the interaction properties of the crack interaction. The values can be determined as described in chapter 3.

The inclusion of the crack analysis in the stress analysis of the steel girder is complicated because of the meshing, as described in section 4.4. The crack included in the model has a height and a depth of 1 mm, the width of the crack is

equal to zero.

4.9 Failure assessment

Failures of the bridge structure during a fire event are either due to large displacements or the exceedance of the maximum possible strain. The non-linear effects of the material and the geometry of the numerical model lead to an automatic abort of the analysis. Until the failure occurs, the results can still be visualized.

Cracking as a result of exceeding the maximum strain is more difficult to model, thus it is easier to compare the strain results with the possible maximum strain to make a statement about possible new cracks and a failure of the structure. The crack growth of the implemented micro-crack is observed independently from the two failure criteria. As described in chapter 4.8, the crack growing is stress-controlled.

CHAPTER 5

Results of the case study

The plurality of the influences on the behavior of the steel girder lead to many different models with a lot of result data. To verify the accuracy of the results, a comparison of the maximum stress and the maximum displacement with analytic results has to take place. When the validity of the model is approved the further models can be carried out.

5.1 Validity of the Abaqus model

The stress analysis of Abaqus without any heat impact can be compared to the analytic calculation. The analytic calculation of the maximum stress and the maximum displacement of a one-span girder is shown in equation (17) and (18).

$$u_{max} = \frac{q * b_f * l^4}{76.8 * E * I} = \frac{100 * \frac{kN}{m^2} * 0.38354 * m * 10000 * m^4}{76.8 * 2.1 * 10^8 * \frac{kN}{m^2} * 0.004287 * m^4} = 5.547mm \quad (17)$$

$$\sigma_{stress} = \frac{M}{S_x} = \frac{q * l^2}{8} = \frac{100 * \frac{kN}{m^2} * 0.38354 * m * 100 * m^2}{8} = 4.40 * 10^4 * \frac{kN}{m^2} \quad (18)$$

The results of the Abaqus model for the stress analysis without any temperature influence is shown in figure (31).

The Abaqus model leads to a maximum deflection of 5.627 m instead of 5.547 m which is a difference of 1.4 %. Because the analytic result does not include any behavior in longitudinal direction of the steel girder, the normal Abaqus model without the additional restrains of the movement in longitudinal direction is used. The maximum stress of the Abaqus model is $4.323 * 10^4 \frac{kN}{m^2}$. The analytic calculated maximum stress is $4.40 * 10^4 \frac{kN}{m^2}$, which leads to a difference of 1.8 %. The small differences are given by the boundary conditions of the Abaqus model. The bearing surface of the Abaqus model decreases the length of the span slightly.

Figure 31. Stress (a) and displacement (b) of the one span beam without temperature influence

The flexibility of the elastomeric bearing in vertical direction leads to an additional vertical movement of the whole structure and the restrain of the horizontal movement affects the behavior of the member as well. Also other boundary conditions such as the loading and the meshing can influence the results. Thus the discrepancy of less than 2 % is within the limits of accuracy.

The analytic equations of a two span beam differ to the one span beam. The two spans influence each other, thus the maximum displacement and the maximum stress at midspan are lower than at the one span beam. The maximum stress of the two span beam takes place at the middle support. The value is exact the same as the maximum stress at the midspan of the one span beam. The values for the displacement and for the maximum stress are given in the equations (19) and (20).

$$u_{max} = \frac{q * b_f * l^4}{76.8 * E * I} = \frac{100 * \frac{kN}{m^2} * 0.38354 * m * 10000 * m^4}{130 * 2.1 * 10^8 * \frac{kN}{m^2} * 0.004287 * m^4} = 3.277mm \quad (19)$$

$$\sigma_{stress} = \frac{M}{S_x} = \frac{\frac{q * l^2}{8}}{S_x} = \frac{\frac{100 * \frac{kN}{m^2} * 0.38354 * m * 100 * m^2}{8}}{0.0109 * m^3} = 4.40 * 10^4 * \frac{kN}{m^2} \quad (20)$$

The results of the Abaqus model for the stress analysis of the two span beam without temperature influences is given in figure (32). The differences of the analytic results and the results of the Abaqus model are explained above.

Figure 32. Stress (a) and displacement (b) of the two span beam without temperature influence

The maximum stress at the top of the steel girder represents the maximum stress of the beam due the load impact better than the stress at the lower flange. The higher values of the area at the lower flange is due to local effects, induced by the elastomeric bearing and the stiffener. The differences of the analytical calculation and the Abaqus model are bigger at the two span beam than at the one span beam. The boundary conditions and modeling parameters have a bigger influence on the two span beam. The difference of the maximum displacement is 6.6 %. The maximum stress of the Abaqus model is $4.651 \frac{kN}{m^2}$, this equates to a difference of 5.7%. But these differences are not due to mistakes of the model, but to the included boundary conditions in the model discussed above.

The heat transfer analysis of the Abaqus model can not be verified by analytic calculations. The verification of the heat transfer analysis by experimental data is not possible because of the lack of information in this field. The temperature profile of the cross section of the steel girder can be compared to previous case studies although the investigated fire event varies. Because of convection and radiation the heating of the different parts of the profile is delayed to the actual fire temperature. The isolation of the upper part of the top flange leads to a bigger delay of the heating. Also the thickness of the different areas impact the

temperature curves. Thus the profile of the steel girder is the same for the two structural systems in this case study, the results for the heat transfer analysis are the same for the different fire impacts. The temperature profiles for the fire event 1 is shown in figure (33).

Figure 33. Heating curves at the three reference points of the cross section under fire model 1 for carbon steel

As mentioned before, the cooling phase of a bridge fire event was not included in a case study yet, thus only the heating phase of the model can be compared to prior results. Although Payá-Zaforteza investigated a hydrocarbon fire the results show many similarities. The results of the heat transfer analysis, evaluated by Payá-Zaforteza et al. [9], is very suitable for a comparison, because of the same reference points of the cross section.

The web gets heated first because of the small thickness of the steel. The upper flange needs more time to get heated because of the almost isolated upper side. The heating behavior at 700°C is special in both case studies. The slow heating at this temperature is induced by the very high values of the specific heat at these special temperatures.

The heat transfer analysis of these two different case studies seem to have the same boundary conditions and the same property models for high temperatures, although it is not possible to compare the exact values.

5.2 Heat transfer analysis

The four fire curves lead to a different heating of the different parts of the steel girder. The heating of the steel cross section has to be determined over a very long time, because of the heating delay of the several parts of the steel girder. The time of the heat transfer analyses varies from 3.5 hours to 4.5 hours, although the fire impacts reach the ambient temperature of 20°C in all cases after less than 2 hours.

The temperature curve of the cross section induced by the fire curve 1 is already partly discussed in chapter 5.1. The web reaches the maximum temperature of 855°C after 22 min. The maximum temperature of the upper and lower flange ($722^{\circ}\text{C}/785^{\circ}\text{C}$) is reached after 37 min respectively 28 min. The ambient temperature of 20°C is almost reached at the whole cross section after 210 min, however the web reaches this temperature much earlier.

The other fire curves are based on the fire curve 1, only the duration of the maximum temperature and the value of the maximum temperature vary for the fire curves 2,3 and 4. The temperature evolution generated by the fire curve 2 is pictured in figure (34).

Figure 34. temperature evolution of the cross section induced by fire event 2

The maximum values at the three different points of the cross section do not

appear at the same time. The maximum temperature of the cross section increases due the longer period of the fire impact. The web reaches 886°C after 24 min. Also the upper and the lower flange reach higher temperatures due to the longer period of the maximum temperature of 900°C of the fire impact. As long as the temperature of the fire is still higher than the temperature of the particular point, the temperature increases. Thus the temperature of the web starts to decrease first. In fact the temperature of the web is lower than the temperature of the other two reference points after approximately 50 min. The ambient temperature of 20°C is almost reached at all reference points after 270 min.

The fire impact 3 has a higher maximum temperature than the fire impacts 1 and 2. Thus the temperatures of the three referent points increase. The maximum temperature of the cross section reaches 930°C. The heating curve of the cross section induced by the fire impact 3 is shown in figure (35).

Figure 35. Temperature evolution of the cross section induced by fire event 3

The fourth fire curve leads to the highest temperatures of the cross section. The web gets almost heated up to the maximum temperature of 960°C. The lower flange reaches over 900°C and the upper flange 800°C. The temperature evolution of the cross section is shown in picture (36).

Figure 36. Temperature evolution of the cross section induced by fire event 4

5.3 Stress analysis

The temperature evolution caused by the four different fire events are applied to the numerical model over the time. Every fire event is applied to the numerical model at three different fire locations, described in chapter 4.2. The stress analysis includes two different steps. Step number one is called the loading step. This step is only used to apply the load and the boundary conditions over the length of the step. The temperature is still the ambient temperature of 20°C. The second step is based on the first one, thus the total load is applied to the model from beginning. The second step takes place over the full time of the heating and cooling phases of the steel girder and the temperature evolution is applied to the stress analysis. The numerical model and the boundary conditions of the stress analysis are described in chapter 4.

The most important result of the stress analysis is the vertical displacement after the fire event. To understand the displacement diagrams in this chapter, it is important to consider that the maximum displacements are shown. The location of the maximum displacement can vary highly during the fire event. Many aspects influence the displacement curve of the model, thus it is important to understand how the change of the location influences the displacement curve. To show the

differences of the maximum displacement and the displacement of a certain point, the two span girder is used. For this example the two span girder is loaded at the fire position 1 and the fire event 1 is used. The maximum displacement of the model is shown in figure (37).

Figure 37. Maximum vertical displacement of the one span beam under the fire event 1 at position 1 without horizontal restraints

The vertical displacement of four reference points is shown in figure (38). The curve of the vertical displacement consists of these four different curves. This applies to every maximum displacement curve shown in this chapter.

Figure 38. Vertical displacements of four reference points along the girder at 4.5 m, 5.2 m, 6 and 15 m

One reference point is located at the second span, which is not directly affected by the fire. During the cooling phase the second span shows the maximum

deflection of the whole girder. This is due the expansion progress within the structure.

5.3.1 Fire event 1

The displacements of the one span beam vary strongly during the time of 3.5 hours. The behavior of the maximum displacements of the steel girder is highly nonlinear. The maximum deflection reaches over 85 mm in the case that the full length of the girder is loaded with the fire impact. The maximum deflection is approximately 50 mm, in the cases that only the right or the left side of the steel girder is loaded. The displacement evolution due to the fire impact 1 is shown in figure (39).

Figure 39. Maximum displacements of the one span beam under the fire event 1 without horizontal restraints

The deflections of the three different cases decrease during the cooling phase. The deflections reach interim lower values than the initial value. This is due to the variations of the stiffness ratio within the steel girders. The elastomeric bearings have bigger vertical displacements than the girder in the middle of the field at this time period. The structural models of all different cases stabilize after approximately 3 hours. The final displacements of the models which are loaded only on one side of the model are 10.2 mm respectively 10.4 mm. The final

deflection of the one span beam loaded over the full length is 11.55 cm. Thus the final deflection of the full loaded beam is just a little bit more than the deflection of the other two cases, although the maximum deflection at the fully developed fire is significantly higher. The steel girder is able to do a lot of movement without a very high permanent post-fire deflection. Amongst others, this is due to the statically determinate structural model.

The same model is analyzed with additional horizontal constraints at both ends of the steel girder. Because of the symmetry of the one span beam, the results for the stress analysis under the fire impact on the right and the left side are very similar. The figure (39) shows only a few differences during the cooling phase, the other values are almost the same for both models. Therefore these two cases are combined for the one span beam in the following.

The restrain of the horizontal movement at the ends of the beam influence the deformed shape highly. While half the structure is loaded with the fire impact 1, the maximum deflection of the beam increases slightly. The maximum value of the deflection of the numerical model without horizontally restraints is 54.2 mm instead of 50 mm, this accords to an increase of 7.7 %. The final deflection after the fire impact is 39.9 mm. This equates to an increase of 291 %. Also the curve of the maximum displacement, for the fire locations 2 and 3, behaves highly different to the displacement curve of the numerical model without horizontal restraints for these fire locations. The variation of the displacements over the time of the fire impact is very low. The restraints at both ends of the girder lead to additional stresses and plastic behavior of the girder. The higher percentage of plastic strains lead to less flexibility of the steel girder during the cooling phase. The maximum displacement continues to happen for the most parts, even after the girder has completely cooled down. The very low values of the numerical model without

horizontal restraints during the cooling phase are extenuated at the numerical model with horizontal restraints. The values are overall more constant on a higher level.

The maximum displacement of the fire location 1 is completely different to the other two fire locations with horizontal restrains, although the displacements curve for the fire location 1 is similar to the other two fire locations without horizontal restrains. The displacements of the girder increase up to over 24 cm during the heating phase. The ratio of the elastic strains to the total strains arise cooling phase. The girder buckles slightly at the location of the maximum displacement after three hours. The maximum plastic strain also occurs at this location. The maximum displacements of the numerical model are shown in figure (40).

Figure 40. Maximum displacements of the one span beam under the fire event 1 with horizontal restraints

The behavior of the two span girder is similar to the one span girder. The maximum values of the two span girder are generally lower than the displacements of the one span girder. The structural model is not symmetric anymore, thus the results for the fire locations 2 and 3 are different. The maximum deflection, which occurs before the temperature decreases, of the fire location 3 is higher than the maximum displacement of the fire location 2. This is due the lower distances between the locations of the maximum displacements at the beam at different

points of time. The maximum displacement of the fire location 3 decreases highly after the peak of the displacement appeared. This is also due the location of the maximum deflection. The location of maximum displacement of the fire location 3 is near the left support which does not restrain the horizontal direction. Thus the strains and the displacements decrease fast. The displacements decrease slower - in case of the fire location 2. The stiff parts of the girder at both sides of the heating area restrain the movement slightly. The part of the displacements curve after approximately 70 min up to 140 min was already discussed above. The maximum displacements for all 3 fire locations are located at the right span of the two span girder at this point in time. The final displacements for the three different fire locations are between 5 cm and 8 cm.

Figure 41. Maximum displacements of the two span beam under the fire event 1 without horizontal restraints

The behavior of the two span girder with horizontal restrains is similar to the behavior of the one span girder. The displacement curves for the locations 2 and 3 are very similar. The highest values of the displacement for the fire locations 2 and 3 are approximately 49 cm. The value does not change much during the cooling phase. The final vertical displacement is 3.7 cm for the fire location 2 and 3 cm for the fire location 3. The steel girder loaded at the fire location 1 behaves differently. The maximum deflection is 21.7 cm before the cooling phase starts.

The final deflection is almost the same with 21.8 cm. The maximum displacements for the three different fire locations are shown in figure (42).

Figure 42. Maximum displacements of the two span beam under the fire event 1 with horizontal restraints

5.3.2 Fire event 2

The fire model 2 is characterized by a fire that takes comparatively longer time before it begins to cool down. The maximum temperature does not increase highly, thus the maximum vertical deflection is almost the same. But the longer time of heating leads to additional plastic strains and to another small peak at approximately 40 minutes. The change of the deformed shape during the cooling phase is the same as under fire model 1. The final deflection of all fire positions increases slightly because of the longer time period of the fully developed fire. Especially the maximum displacement of the steel girder loaded at fire location 2 develops more plastic behavior. The final displacement is approximately 150 % of the equivalent value for the fire model 1. The maximum displacements of the one span beam under fire event 2 is shown in figure (43).

The diagram for the numerical model with horizontal restrains under fire event 2 is very similar to the results under the fire impact 1 with the same boundary conditions.

Figure 43. Maximum displacements of the one span beam under the fire event 2 without horizontal restraints

The final vertical deflection of the one span girder loaded at the fire position 1 is approximately 310 cm. The final vertical deflection for the load position 2 is still stable at under 5 cm. The two deflection curves are shown in figure (44).

Figure 44. Maximum displacements of the one span beam under the fire event 2 with horizontal restraints

The two span girder behaves similarly to the one span girder. The maximum vertical displacements are also similar to the results of the fire model 1. The curves can be distinguished by another peak at approximately 30 min. The final vertical deflections increase slightly for all three fire locations. The maximum displacements for the two span girder under the fire event 2 without horizontal restraints are shown in figure (45). The increase of the vertical displacements

between 70 and 150 minutes results from the interaction of the two different spans of the steel girder. This effect is pictured in figure (37). The behavior of the left span is similar to the static model of the one span beam. The thermal expansion reduces from the middle of the heating surface to the ends of the steel beam, this leads to a change of the displacement shape. After the whole girder cooled down completely the displacement shape looks similar to the displacement shape after the heating time.

Figure 45. Maximum displacements of the one span beam under the fire event 2 without horizontal restraints

The vertical deflections of the fire positions 2 and 3 with horizontal restraints are almost the same as the results for the fire model 1 under the same boundary conditions. The maximum deflections of the steel girder loaded at the fire position 1 increase in relation to the fire model 1. The final deflection of the girder is more than 27 cm. The final deflection is higher than the maximum deflection before the cooling phase begins. This is due to the very high plastic strains. The diagram of the vertical deflections for the different fire locations are shown in figure (46).

Figure 46. Maximum displacements of the two span beam under the fire event 2 without horizontal restraints

5.3.3 Fire event 3

Fire event 3 has a maximum temperature of 960°C as discussed in chapter 4.1. Although the different parts of the girder do not reach this temperature they get warmer as if the steel girder is loaded under the fire model 2. The structural model of the one span beam shows higher values for all three fire locations. The fire positions 2 and 3 under fire event 1 lead to a similar results as under the fire events 2 and 3. The vertical deflection of the steel girder loaded at the fire location 1 has different characteristics. The high temperatures lead to additional plastic strains and a plastic behavior of the bridge. This leads to an additional increase of the deflections at approximately 30 minutes. The values increase up to a level of 110 cm. During the cooling phase the maximum vertical deflections decrease again but the plastic strains remain in the structure. Thus the final vertical deflection of the steel girder loaded at the fire location 1 is six times higher than before. It is easy to see how the plastic strains remain in the structure while the partly elastic behavior is also visible. The shape of the displacement curve of the steel girder for the fire location 1 is similar to the curves under the fire models 1 and 2 but on a much higher level. The two curves are shown in figure (47).

Figure 47. Maximum displacements of the one span beam under the fire event 3 without horizontal restraints

The vertical deflections behave similar to the fire events 2 and 3 if the horizontal movement is restrained. The plastic strains for the fire locations increase because of the horizontal restraints. The final deflection of the steel beam at the fire location 1 is more than 35 cm. The curve for the fire locations 2 and 3 are almost the same as under the fire impacts 1 and 2. The final deflection is almost 5 cm. The maximum vertical deflection for the one span beam with horizontal restrains is shown in figure (48).

Figure 48. Maximum displacements of the one span beam under the fire event 3 with horizontal restraints

The behavior of the two span beam under the fire event 3 is similar to the behavior of the one span beam. The displacement shape of the numerical model

under the fire event 3 at the fire location 2 is very different to the displacement shape of the same numerical model under fire event 2. The displacement shape is similar to the displacement shape of the fire location 1. The maximum vertical displacement after the heating time is more than 10 cm for the fire locations 1 and 2. This is equal to an increase of approximately 75 cm for the fire location 2. It seems that the numerical model reaches critical values where nonlinear effects lead to a very high increase of the vertical displacements. The fire load at the fire location 3 leads to fewer displacements. The displacement curve is similar to the fire events 1 and 2 with a slightly increase of the values of the maximum vertical displacements. The described behavior of the three different numerical models are shown in figure (49).

Figure 49. Maximum displacements of the two span beam under the fire event 3 without horizontal restraints

The horizontal restraints have a big impact on the different numerical models. The final vertical displacements due to the fire event 3 at the fire locations 2 and 3 are approximately 3.5 cm (fire location 3) and 5 cm (fire location 2). The fire location 1 leads to a final displacement of more than 30 cm. The diagram of these three maximum vertical displacement curves is shown in figure (50).

Figure 50. Maximum displacements of the two span beam under the fire event 3 with horizontal restraints

5.3.4 Fire event 4

Fire event 4 leads to different displacements for the fire locations 2 and 3 than all three fire events discussed before. The very high temperatures lead to big plastic strains. The maximum displacements increase to over 40 cm for the three different numerical models. The effect of the elastic behavior is very small because of the high plastic strains of the one span steel girder. The final vertical deflection of the numerical models loaded at the fire locations 2 and 3 is bigger (40 cm) than the final deflection of the numerical model with the fire location 1 (37 cm). The behavior of the one span girder without horizontal restraints under fire event 4 is shown in figure (51).

The horizontal restraints influence the vertical displacement of the different numerical models very differently. The numerical model of the fire location 1 with horizontal restraints shows a behavior similar to the same model without horizontal restraints during the heating phase. The maximum displacements increase up to 42 cm, slightly more than without horizontal restraints. But the horizontal restraints prevent the release of the thermal expansion, this leads to additional stresses, plastic strains and vertical displacements. The final maximum displacement is about 50 cm. The displacement curve for the fire locations 2 and 3 is similar to

Figure 51. Maximum displacements of the one span beam under the fire event 4 without horizontal restraints

the fire events 1,2 and 3, although the vertical displacements is very high for the numerical model including the fire locations 2 and 3 without horizontal restraints. The final vertical displacement is about 5 cm high. The reason for this is the location of the maximum displacement after the fire. The location of the maximum displacement moves from the middle of the steel girder close to the supports, if just half of the girder is heated (fire locations 2 and 3). This is easier if the horizontal movement of the steel girder is not restrained. It explains the very high values for the numerical model of the fire locations 2 and 3 without horizontal restraints. The final vertical displacement is still in the middle of the girder, if the full length is heated (fire location 1). The displacement curves are shown in figure (52).

The numerical models of the two span beam including the fire locations 1 and 2 fail after approximately 23 minutes. The software stops the analysis due the fast increase of the vertical deflections. The vertical displacements at the failure are 37 cm (fire location 1) and 30 cm (fire location 2). The fire event 4 at the fire location 3 leads to smaller displacements. The displacements are not on the critical level, where the vertical deflections increase very fast.

The displacement curve for the fire position 3 is very similar to the displacement curves of the fire events 1,2 and 3. The displacement curves without hori-

Figure 52. Maximum displacements of the one span beam under the fire event 4 with horizontal restraints

zontal restrains are pictured in figure (53).

Figure 53. Maximum displacements of the two span beam under the fire event 4 without horizontal restraints

The maximum displacements for the fire locations 2 and 3 are almost the same although they are so different for the same numerical models without horizontal restraints. The final deflections for the numerical models for the fire locations 2 and 3 are about 5 cm. The shape of the maximum vertical deflection for the fire location 1 is similar to the fire events 1,2 and 3. The final vertical displacement is approximately 55 cm. The analysis completes although the displacements are higher than the vertical deflections of the numerical model without horizontal restraints. The maximum vertical displacements of the numerical models under

fire 4 at the fire locations 1,2 and 3 with horizontal restrains are shown in figure (54).

Figure 54. Maximum displacements of the two span beam under the fire event 4 with horizontal restrains

5.4 Crack growth during the stress analysis

The crack analysis has not been carried out for every single numerical model. The principal behavior of a micro-crack during a fire event has to be investigated. Three different locations for the micro-crack are possible because of the meshing. All crack locations are close to stiffener, thus these areas are very vulnerable to fatigue cracking. One crack is located at the border of the heating surface at the left support. The second one is at the border of the heating surface at the middle support and the third crack location is a little bit outside of the heating area, right next to the middle support. To evaluate the behavior of the crack during the heating process, the first step has to be the crack analysis without any kind of temperature change.

The crack does not grow at all at the ambient temperature, the stresses and strains are too small, no matter at what place the crack is located. A growing of the crack at this point would imply errors of the numerical model. These micro-cracks are probably caused by very high loads and fatigue mechanisms. The dead

loads alone can not lead to a further growing of the crack.

The heating process leads to a decrease of the stiffness of the affected parts of the girder and to very high stresses and plastic strains at the border of the heating surfaces, i.e. at possible crack locations. As described above just overall behavior will be discussed and not the influences of certain boundary conditions. The fire events lead to a growing of the crack if the micro-crack is located at the border of the heating surface. An example of crack growth is shown in figure (55).

Figure 55. Growth of the micro-crack while the steel girder is exposed to fire

The figure (55) shows the crack after the fire exposure. The original crack with a length of 1 mm is highlighted in red. The crack grows in both directions during the fire event. The final crack length in this case is approximately four times bigger than the original crack. The crack starts to grow relatively late after 50 minutes compared to the displacement-time-curves discussed above.

The crack location outside of the heating area do not lead to a growing of

the crack. The high stresses and plastic strains are restricted to the heating area. Cracks located outside of the heating area are not vulnerable to crack growing.

CHAPTER 6

Summary

The results of the maximum vertical displacements of the steel girder by different boundary conditions are multifaceted. The different displacement curves are shown and described in chapter 5. The most important finding is that vertical displacements happen due to two different mechanisms. The material steel loses strength and stiffness under high temperatures. A low stiffness leads to big displacements. These displacements can go back on a lower level, depending on the ratio of the plastic and elastic behavior. The second mechanism is the thermal expansion of the material. The high length of the heated surface and the very high temperature difference lead to large expansions, as described before. The expansion leads to bigger displacements during the heating phase and to lower displacements during the cooling phase. The exact impact of the thermal expansion depends on the boundary conditions, such as horizontal restraints or the statical system of the hypothetical bridge.

This case study includes many of these different aspects. The main focus in this case study is on the influence of the following:

- The intensity of the fire event
- The location of the fire
- Availability of horizontal movement
- Statical model of the steel girder
- Micro-cracks inside of the steel girder

The outlining of the influences of the several aspects lead to general findings of the post fire behavior of steel girders.

6.1 The influence of the different fire events

The fire model has the most obvious influence. The maximum temperature of the fire event defines the temperatures of the heated surface. The duration of the fully developed fire also affects the temperature of the cross section, due to the convection and radiation of the surface.

As expected, the fire event 4 leads to the largest displacements of the numerical model and the fire event 1 to the smallest. The fire event 1 leads to a maximum temperature of the web of 855°C . The long time of heating of the steel surface under fire event 2 leads to a maximum temperature of 886°C . The fire event 3 leads to a maximum temperature of the web of 932°C and the fire event 4 leads to the highest temperature of 956°C . The difference of the maximum temperatures do not seem to be very big, but the stiffness of the material steel decreases significantly at these temperatures. To outline the influence of the fire event on the behavior of the bridge the figure (56) is presented.

Figure 56. Maximum displacement of the two span girder under the fire events 1,2,3 and 4 at the fire location 2 without horizontal restraints

Shown are the maximum displacements of four different numerical models under different fire events but with the same boundary conditions. In order to point out the differences, only the first 40 minutes are being displayed. The diagram indicates that the behaviors of the numerical models are similar during the first

fifteen minutes. This is due to the identical heating phases until the fire models reach 900°C . After this point the numerical models behave differently. Fire event 1 and 2 lead to very similar results although the fire event 2 leads to higher temperatures than the fire event 1. The maximum displacements under fire event 3 are much higher than the displacements under the fire events 1 and 2. The maximum displacement is approximately 10 cm after 40 minutes. At this point, the two span steel girder does not fail yet. The maximum deflections under fire event 4 are even higher and the structure fails after 23 minutes.

The fire intensity influences the behavior of the bridge highly. The critical temperatures start at approximately 900°C . The displacements increase highly nonlinear in this sector. The vertical displacements of the numerical model under fire event 4 increase to values three times higher (before the structure fails) than the vertical displacements under fire event 3, although the maximum temperature of the steel surface under fire impact 4 is only 2.5 % higher than the maximum temperature of the surface under fire event 3. This is due to the high decrease of the stiffness of steel under these temperatures. The expansion does not lead to this high displacements, the differences of the temperatures is to low.

6.2 The location of the fire

Three different fire locations have been investigated in this case study. The differences of the vertical displacements for the three fire locations are discussed in chapter 5 illustrated by several diagrams. Fire location 1 is over the full length of the span (at the two span girder just the left span), thus two times the length of the fire locations 2 and 3. The big length leads to a high thermal expansion and to big displacements. The shape of the displacement curve is similar to the displacement curves for the fire locations 2 and 3 for the numerical model without horizontal restraints under the fire events 1 and 2. The big differences of the

maximum displacements of the fire location 1 to the fire locations 2 and 3 with horizontal restraints is due the thermal expansion and the nonlinear effects of very high displacements. The girder can not expand due to the horizontal restraints, thus the vertical deflection has to balance the thermal expansion.

Another reason for the high vertical deflections for the numerical model loaded at the fire location 1 are the similar locations of the maximum deflections during the fire event. The fire locations 2 and 3 lead to a new location of the maximum deflection after the fire. This means less displacements but a big ratio of plastic strains. Thus the final deflection is almost on the same level as the maximum deflection before the cooling phase starts.

For the two span girder the analysis also showed differences between location 2 and 3. The results are the same for both fire locations at the one span girder because of the symmetry. The difference between the fire locations 2 and 3 only appears at the numerical models without horizontal restraints. The location of the maximum displacement is at 4.64 m before the fire event occurs. This leads to bigger displacements of the numerical model loaded at the fire location 3 than the numerical model loaded at the fire location 2. The free movement at the left supports leads to a quick reduction of vertical displacements in the case of the fire location 3. Both ends of the heated surface of the fire location 2 are very stiff and slow the reduction of the vertical displacements down. The fire location 2 leads to more plastic strains, which leads to slightly bigger final displacements under the fire events 1 and 2 and to much higher displacement under the fire impacts 3 and 4 (without horizontal restraints). The results for the vertical deflections with horizontal restraints are similar for the fire locations 2 and 3, the effect of the free movement at the left support does not apply anymore.

6.3 Influence of horizontal restraints

As mentioned before, the horizontal restraints affect the behavior of the steel girder highly. The horizontal restraints reduce the maximum deflection before the fire occurs to almost 50 %. Due to the thermal expansion the vertical deflections increase much more than without horizontal restraints. The vertical displacement curves for the numerical models with horizontal restraints have very similar shapes for all fire events. The numerical model without horizontal restraints fails under fire event 4. The displacements reach the critical values and the nonlinear effects lead to an additional big increase of the vertical displacements. The same numerical model with horizontal restraints leads to a stable displacement curve similar to the fire events 1, 2 and 3.

The differences of the maximum values for the fire locations 2 and 3 affected by the different fire events are very small. This is due the small differences of the maximum temperatures and the small additional thermal expansion, the numerical models act very stiff. After the vertical deflection reaches the maximum, the variation of the maximum vertical displacements is small because of the constant plastic strains. The elastic strains get reduced during the cooling phase, but the impact is not as big as at the numerical model without horizontal restraints because of the big ratio of plastic strains.

The differences of the numerical models with horizontal restraints of the different fire events at the fire location 1 are more concise. The maximum vertical displacements after the heating time increase by 29 cm from fire event 1 to fire event 4. The additional increase of the maximum vertical displacements have a distinct impact after approximately 80 minutes. This is due the progress of the reduction of the thermal expansion. The nonlinear effects increase at higher displacements. The vertical displacements under fire event 4 are increased by 10 cm

at the end of the cooling phase.

6.4 Comparison of the different static models

The results for the two different static models included in this case study have many similarities. The structural system of the one span beam leads to higher values for the maximum vertical displacements of the stress analyses, this is due to the static model itself. Also the locations of the maximum displacements for the two different static models, before the fire event occurs. All boundary conditions are the same for both numerical models so the two different static models can be compared.

The only difference between the two statically models is the differentiation of the fire positions 2 and 3. Because of the symmetry the results for the fire positions 2 and 3 of the one span beam are almost identical. The maximum vertical displacements are different for the fire positions 2 and 3 of the two span beam, but only if the girder is not restrained in horizontal direction.

6.5 Behavior of the micro-crack

Micro-cracks with a length of 1 mm grow to up to 5 mm in this case study. The lesser strength of carbon steel at high temperatures leads to lower resistance to crack growing. Vulnerable to crack growing are the borders of the heating phase, because of the high stresses and plastic strains in this area. The exact values of the cracks after the fire in this case study are not very reliable, because of the many different boundary conditions that influence the growing. Nevertheless the goal of the crack analysis in this case study was to show the general behavior of micro-cracks during fire impacts at different locations. Areas close to the several supports are vulnerable to fatigue micro-cracks and to micro-cracks caused by high stresses due to welding of the stiffener, for example. These areas are also often at

the border of the heating surface of a fire event and thus very vulnerable to crack growing during a fire event.

CHAPTER 7

Conclusion

This case study shows the post fire behavior of a two span girder under small vehicle fires. The two span steel girder represents a standard highway bridge, to minimize the numerical model and to show the impact of several boundary conditions on the post fire behavior of the structure. Unlike to prior case studies in this field the collapse of the bridge was not the key point of the analysis, but the research question was the behavior of a bridge structure during all different phases of the fire and after the whole structure cooled down.

Fire impact on bridge structures affect the bridge structure in two different ways. The most obvious impact of a fire on a bridge structure is the decrease of the stiffness of the materials. The decrease of the stiffness of the material steel is non-linear and the stiffness gets very low at high temperatures. The low stiffness leads to high displacements and plastic behavior within the bridge structure. The other very important impact of the fire on the bridge structure is the thermal expansion. The high temperatures and the big length of the heating surface lead to significant values for the thermal expansion. The interaction of these two effects and the influence of the several boundary conditions lead to the results of the different numerical models.

A very important finding of the case study is the critical temperature of the several parts of the steel girder. The different fire events used in this case study lead to temperatures close to the critical temperature of the bridge structure. The final displacements of a hypothetical bridge are not very big, if the car fire induces the temperatures of a bridge's steel girder to rise up to but not above 900°C and half of the span of the bridge is affected by the fire. Temperatures higher than

the critical temperature can lead to very high plastic strains. This behavior is mostly due the very low stiffness at these temperatures, the thermal expansion can not lead to these big differences, because of the small discrepancies between the maximum temperatures.

The available movement in horizontal direction affects the behavior of a bridge highly. Horizontal restraints lead to higher displacements during the heating time. But horizontal restraints also lead to more stiffness within the structural model, the bridge behaves almost like a frame bridge. The additional stiffness within the bridge structure leads to similar results for the different fire events, at least if half of the span is loaded by the fire. The final displacement of approximately 5 cm after the fire leads to replacements of parts of the steel girder. But the system is not at risk to collapse, the structural model is stable under the several fire events.

Also important while analyzing a bridge structure under fire is the consideration of the condition of the bridge. Especially when the post-fire behavior of the bridge is investigated. This is another aspect that was not included in any case study yet. As shown in the analysis, micro-cracks can grow extensively during a fire. Only micro-cracks of the size of 1 mm were included in this case study, bigger cracks can grow even more. Also the interaction of multiple micro-cracks during a fire event could influence the growing of a micro-crack. The border of the heating surface should be investigated very accurately after a fire to find and repair cracks.

General repair strategies for steel and concrete structures are discussed in chapter 2.2 and can be applied to damaged bridge structures after a fire event.

CHAPTER 8

Research outlook

This case study offers some information about the post-fire behavior of steel girder bridges. To complete the analysis more case studies are needed. But the most important lack of information affects the temperature data of car fires. Almost all fire experiments so far are applied to huge gasoline tankers. These fires are difficult to simulate with a software. The temperature development on a bridge caused by a vehicle fire at a certain distance is affected by multiple boundary conditions. Fire experiments have to be carried out in a way similar to fire experiments in tunnels, such as the 'RABT-ZTV curve' used in this case study. Exact fire data would increase the accuracy of a case study.

The condition of a bridge has to be included in future researches. Rust is also a common problem of existing highway bridges. Rust reduces the cross section of a girder which decreases the capacity of the steel girder. The behavior of a reduced cross section caused by rust during a fire event should also be subject to another case study.

Further research based on this thesis should be carried out about the post-fire capacity of the composite bridge structure. The bond of the steel girder and the concrete degrades due to the fire impact. Also the changed properties of the materials concrete and steel, due to the fire impact, lead to a lower capacity of the bridge structure.

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APPENDIX

Appendix

Figure A.1. Vertical displacements of the two span girder without horizontal restraints before the fire event occurs

Figure A.2. Final displacements of the two span beam under the fire event 1 at the fire location 1 without horizontal restraints

Figure A.3. Final displacements of the two span beam under the fire event 1 at the fire location 2 without horizontal restraints

Figure A.4. Final displacements of the two span beam under the fire event 1 at the fire location 3 without horizontal restraints

Figure A.5. Vertical displacements of the two span girder with horizontal restraints before the fire event occurs

Figure A.6. Final displacements of the two span beam under the fire event 1 at the fire location 1 with horizontal restraints

Figure A.7. Final displacements of the two span beam under the fire event 1 at the fire location 2 with horizontal restraints

Figure A.8. Final displacements of the two span beam under the fire event 1 at the fire location 3 with horizontal restraints

Figure A.9. Final displacements of the two span beam under the fire event 2 at the fire location 1 without horizontal restraints

Figure A.10. Final displacements of the two span beam under the fire event 2 at the fire location 2 without horizontal restraints

Figure A.11. Final displacements of the two span beam under the fire event 2 at the fire location 3 without horizontal restraints

Figure A.12. Final displacements of the two span beam under the fire event 2 at the fire location 1 with horizontal restraints

Figure A.13. Final displacements of the two span beam under the fire event 2 at the fire location 2 with horizontal restraints

Figure A.14. Final displacements of the two span beam under the fire event 2 at the fire location 3 with horizontal restraints

Figure A.15. Final displacements of the two span beam under the fire event 3 at the fire location 1 without horizontal restraints

Figure A.16. Final displacements of the two span beam under the fire event 3 at the fire location 2 without horizontal restraints

Figure A.17. Final displacements of the two span beam under the fire event 3 at the fire location 3 without horizontal restraints

Figure A.18. Final displacements of the two span beam under the fire event 3 at the fire location 1 with horizontal restraints

Figure A.19. Final displacements of the two span beam under the fire event 3 at the fire location 2 with horizontal restraints

Figure A.20. Final displacements of the two span beam under the fire event 3 at the fire location 3 with horizontal restraints

Figure A.21. Final displacements of the two span beam under the fire event 4 at the fire location 1 without horizontal restraints

Figure A.22. Final displacements of the two span beam under the fire event 4 at the fire location 2 without horizontal restraints

Figure A.23. Final displacements of the two span beam under the fire event 4 at the fire location 3 without horizontal restraints

Figure A.24. Final displacements of the two span beam under the fire event 4 at the fire location 1 with horizontal restraints

Figure A.25. Final displacements of the two span beam under the fire event 4 at the fire location 2 with horizontal restraints

Figure A.26. Final displacements of the two span beam under the fire event 4 at the fire location 3 with horizontal restraints

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