FIRE PERFORMANCE OF FIN PLATE SHEAR CONNECTIONS MODELLED WITH COMPONENT BASED METHOD AND 3D FINITE ELEMENTS

by

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ABSTRACT

FIRE PERFORMANCE OF FIN PLATE SHEAR CONNECTIONS MODELLED WITH COMPONENT BASED METHOD AND 3D FINITE ELEMENTS

Steel buildings bring together structural fire problems that are very dangerous especially for structures in large cities. Fire performance of steel structures is ignored in design and construction stage of the buildings. Recently, an innovative mathematical model called Component-Based Method (CBM) has been developed in order to analyze the fundamental behavior of fin-plate shear connections under a realistic fire situation. This approach has achieved popularity because of its advantages compared to the other analytical mathematical methods. It may be used for any joint type, various connections and loading situations. This method also demonstrates the load-deflection and moment-rotation curve of components that is properly defined in the research. This study aims to evaluate the fire performance of a beam with a fin-plate connection type and to obtain realistic data for the structures with CBM under realistic fire conditions. It also aims to investigate the behavior of the fin plate connections by comparing with the idealized pinned and fixed connections at high temparatures by using the CBM. Firstly, a fin-plate connection is modelled by using CBM. Then, the beam axial force, connection moment, rotation as well as midspan deflection history of CBM model are compared the connection model using 3D finite elements model that is created by using same material properties and loading situations. The results show a semi-rigid connection performs differently compared to idealized pinned or fixed beam end conditions. The results suggest that the CBM model shows good agreement with the computationally expensive 3D finite element model and it can be used to simulate semi-rigid connections under fire conditions.

ÖZET

'COMPONENT-BASED' VE ÜÇ BOYUTLU SONLU ELEMANLAR YÖNTEMİYLE MODELLENEN PLAKALI KESME BAĞLANTILARININ YANGIN PERFORMANSI

Çelik yapılar özellikle büyük şehirlerde, çok tehlikeli olan yapısal yangın problemlerini beraberinde getirmektedir. Çelik yapıların yangın performansı genellikle tasarım ve yapım aşamasında ihmal edilir. Son zamanlarda Component Based Method (CBM) adlı yenilikçi bir matematiksel metot plakalı kesme bağlantılarının yüksek sıcaklık altındaki temel davranışlarını analiz etmek için geliştirilmiştir. Bu yaklaşım diğer analitik metotlara kıyasla sahip olduğu avantajlarından dolayı gittikçe daha fazla popülerlik kazanmıştır. Bu metot herhangi bilesim tipi için, çeşitli bağlantı konfigürasyonlarında ve yükleme durumlarında kullanılabilir. Ayrıca bu metot, araştırma sırasında doğru tanımlanmış her bir bileşenin kuvvet-yer değiştirme grafiğini gösterir. Bu çalışma, çelik kesme bağlantılarının yüksek sıcaklıktaki performanslarını değerlendirmeyi ve CBM analizi ile gerçek yangın şarları altındaki yapısal bir sistem için gerçekçi sonuçlar elde etmeyi amaçlamaktadır. Ayrıca CBM ile, idealleştirilmiş pinned ve fixed bağlantıları ile, plakalı kesme bağlantılari karşılaştırilarak, plakalı kesme bağlantılarının yüksek sıcaklıktaki davranışları analiz edilmiştir. İlk olarak, gerçekçi yangın koşulları altında ABAQUS sonlu elemanlar kullanılarak plakalı kesme bağlantılı CBM model oluşturuldu. Daha sonra bu model, CBM model ile aynı malzeme özellikleri ve yükleme koşulları altında oluşturulan 3 boyutlu sonlu elemanlar modeli ile karşılaştırıldı. Sonuçlar, kesme bağlantılarının idealleştirilmiş pinned veya fixed bağlantılarından farklı davrandığını göstermektedir. CBM model, diğer hesaplamalı sonlu elemanlar metodu ile uyum göstermektedir ve kesme bağlantılarının yangın şartları altında simülasyonuna olanak sağlamaktadır.

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LIST OF ACRONYMS/ABBREVIATIONS

AISC	American Institute of Steel Constructions
EUROCODE	European Committee for Standardization
SCI	Steel Construction Institute

1. INTRODUCTION

1.1. Background

High-rise steel structures can bring together structural fire hazards in large cities. If a building is not properly designed and constructed to withstand potentially catastrophic events due to risk factors posed by fires, such disasters can occur. The hardest part of this incident is to put out the fire in time and to evacuate the people from the fire building. In order to design of sustainable steel structures, the buildings must be robust and resistant, considering the possibility of extreme fire scenarios. These structural fire scenarios are more damaging to steel buildings because of the complex interactive behavior of steel members.

Latest studies regarding structural fire open new choices and challenges in multi-disciplinary study fields such as earthquake, risk assessment and reliability analyses. In some earthquake-prone regions, it is very important to design structures to be durable and strong for subsequent fires after an earthquake (Cruz *et al.*, 2004). The possibility of a fire event, a fire duration and a fire's severity are quite difficult task to measure. Because of this reason, it is required to assess the imposed risk to structural integrity during fire (Scheich, 2010).

From the beginning of fire science till now, structural fire process has changed on a large scale especially in testing methods. In 1908 the American Society for Testing Materials (ASTM) published a standard test method based on the need to develop a common approach to evaluating the fire safety of building construction materials. In 1918 a time-temperature curve was established as part of the standard based on the maximum temperatures experienced in real fires at the time (Parkinson, 2006). Actually, the curve was not based on the response of building components to a real fire but rather what the authors have described as a worst-case time-temperature relation to be expected during a fire (Parkinson, 2006).

The structural fire engineering is important for using basic knowledge of different research areas including structural mechanics such as heat and mass transfer, thermodynamics, fire growth and fire dynamics. When structural materials subjected to elevated temperatures, both their thermal and mechanical properties change. For example, the expansion of steel will cause significant geometric changes in structural systems therefore; it can result in large deformations and failure depending on the fire severity and the amount of fire protection (Purkiss, 1996).

Design of any building based on fire safety rule has become an important way for designers and has become more popular in recent years. Historically fire safety design of buildings has been seen as a constraint to innovative design, but this needn't be the case. Many advanced building designs now utilize fire safety engineering rather than having to rely on only functional based codes. This approach can enable architects to achieve innovative cost-effective designs while meeting fire safety design needs. Successful fire safety design requires an understanding of a wide range of issues and components, and the interactions between them such as fire sources, smoke movement, heat transfer to the building structure, detection, human behavior and toxicity.

Recent fire studies have shown that steel shear connections have an important role in building's fire damage. One of the vital problems of analyzing tall steel structures is simulating the force and moment equilibrium between the steel member and other structural elements accurately (Lim, Buchanan, Moss and Franssen). Another problem of steel structures under the fire hazard is the performance of steel connections and their rotation capacities and also internal forces and moment equilibrium of the steel beam sections (Dong and Prasaad). It is questionable if the steel shear connections can sustain large rotation and tensile force under a fire exposure even if implementing fire safety design rules (Beitel and Iwankiw). There are many studies about the force and moment equilibrium of steel connections and these studies assume that the steel connections are isolated. It is a fact that the force and moment equilibrium of steel connections and fire resistant of the whole system depend on the relationship between steel connections and other structural elements.

Beam-to-beam joints in steel buildings transfer the floor and beam loads to the columns, and play an important role between the basic structural elements for the whole structural durability. The forces transmitted through the joints can be axial and shear forces and bending moments. Moreover, the magnitudes of the actions transferred by the connections to the supporting members depend on the connection type and the surrounding conditions. In the case of steel frame structures with moment-resistant connections, such as bolted end plates, the bending moments are predominant compared to axial and shear forces. In contrast, the use of shear connections, such as fin plates, would make the shear force dominant compared to axial force and bending moment. Also, these actions are greatly influenced by unusual circumstances such as an earthquake or a severe fire (Sarraj, 2009). It is a fact that the large mid-span deformations are induced in beam elements during the stage of a fire when unprotected steel beams lose strength. However, in the case of simple shear joints, the end joints are forced to provide two unusual kinds of reaction, against tensile catenary action, and against the induced rotations or end moments. These end moments, provided by the joints, reduce the mid-span moments in the beams, improve the load carrying capabilities and fire resistance of a nominally simply supported beam as long as the ductility of the connections are preserved (Sarraj, 2009).

1.2. Objectives and Scope

The aim of this study is to investigate the effect of a beam with a fin-plate steel connection on structural resistance under realistic fire conditions and draw conclusions for a robust structural design. In this thesis study, a common beam-to-beam fin-plate shear connection type is evaluated under the increasing temperature and, the axial capacity and moment curves of this type of beam connection is calculated. The first stage of this process is to find the stiffness and strength values of used beam by mathematically using some equations, and to implement this obtained values into the corresponding model by a finite element software program. The second stage of this process is to compare the results that obtained from corresponding model with the 3D finite element model. After accurate and consistent results are obtained, the corresponding model is used mainly to understand the fin plate connection behavior under heating and cooling conditions, and compare with the idealized pinned-pinned and fixed-fixed connections at these temperatures. With this thesis study, it will be aimed to analyze a beam with a fin-plate steel shear connection under realistic fire condition and to contribute to the fire regulations by obtaining rational results. The method used in this study is the first researched based method for a fin-plate steel connection in Turkey. By this method, steel connections may stay robust in the case of a fire hazard.

The literature review on the fire performance of connections suggest that there have been only limited numerical studies conducted about this topic and most of them have aimed to develop the moment-rotation characteristics of connections. It is expected that deficient areas of structural fire literature will be filled and new research areas will be developed in the future. This thesis mainly focuses on the structural steel beams with fin-plate connections and it should be noted that these connections have some deformations and partial failure in fire incidents (Beitel and Iwankiw, 2001). The most important issue is to advance one step further in the improvement of structural connections in all type of structural frames. Therefore, this thesis will attribute to literature with developing resistant shear connections in design.

The proposed thesis is important because it will fill the knowledge gap and it aims to understand the fire performance of a beam with a fin-plate connection type in a system, and to investigate the interaction of axial load- deformation, moment-rotation graphs of these connections by using a mathematical method. It is important that to validate this proposed model with a 3D finite element model is needed to obtain more accurate and acceptable results.

Expected results with this study will create awareness for the structural fire safety in beams with fin-plate shear connections and help structures to improve against significant hazards. Also it should be noted that, it is very important to enhance the use of mathematical expressions for three-dimensional structural connection modeling for thermo-mechanical problems in research universities. Moreover, obtained results from this study will compare the connections for fire safety from high to low risk of failure. However, the most important consequence that founded with this research is to understand how the fin-plate shear connections are affected in the case of a fire in the buildings.

- Evaluating a beam-to-beam with a fin-plate shear connection type that are used in a structural building.
- Plotting Axial Force- Deflection and Moment- Rotation behaviors of this type a connection by using a mathematical method via a finite element program. Thus, it is the first calculation method for the beams with a fin-plate shear connection in Turkey.
- Comparing this obtained graphs and models with a 3D finite element model to validation.
- Simplifying the results and guidelines for fire performance of structures in a suitable form for reference.

1.3. Literature Review

1.3.1. Structural Fire

Engineers dealing with the structural fire problems have a challenge to analyze and design of structures for fire loading. The common design approaches at ambient temperature include non-varying components for the fire calculations. Therefore, the design procedures allow to simplfy for the assumptions and therby, these simplifications cause to smaller deflections in constrast to elevated temperatures. Due to material and geometric non-linearity of the structures at high temperatures, there are unpreventable complexity in the structure as well as huge deformations. This scenario is increased by the effects of restraint to thermal expansion of heated elements by cooler structure. Moreover, the complications include non-uniform stresses that are created by the heating- cooling cycle. In a real fire condition, the temperature shows differences between the structural members' different parts at a point during the growth and decay phases (Taib, 2012).

As mentioned in detail above, fire behavior is a complex process influenced by a large number of parameters. A typical time-temperature relationship of a standard and real fire is illustrated in Figure 1.1. The standard fire curve according to National and European standard is compared with a typical real compartment fire. The growth, flashover, fully developed and decay phases are illustrated in this figure. The possible processes of fire development are described in below (Cruz, 2004).

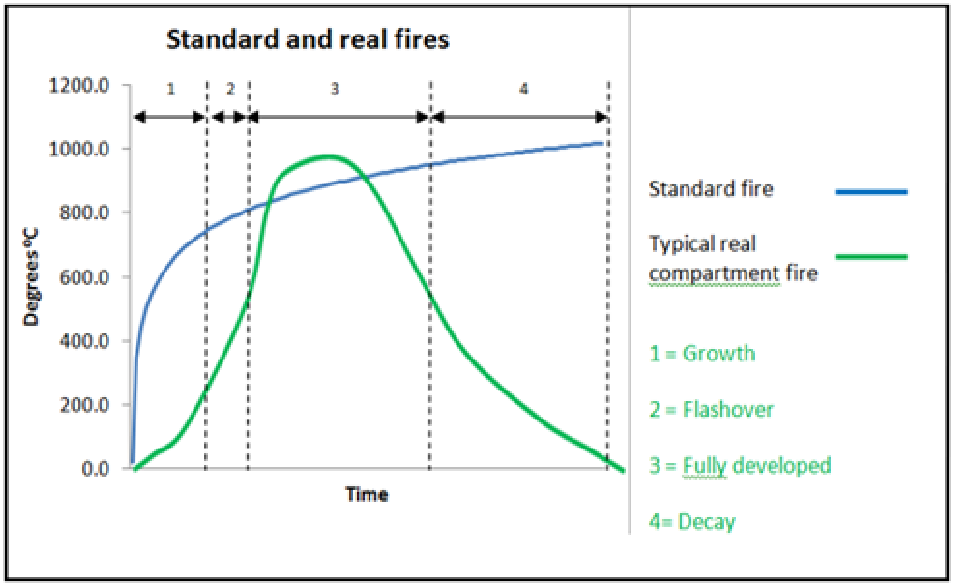

Figure 1.1. Phases of a Standard and Real Fire Scenario (Cruz, 2004).

Fire phases can be described as;

(i) *Growth phase (pre-flashover):* During the initial phase the fire will remain localized. The products of combustion will accumulate beneath the ceiling forming a hot layer. Depending on the availability of oxygen for combustion, the growth phase may be characterized by flaming. As seen in Figure 1.2, the fire starts in a point and spreads all points because of existing oxygen. This phase is most serious for life safety as tenability conditions can often be compromised through the production of carbon monoxide and other toxic gases. The fire will continue to grow given sufficient heat release from the item first ignited, sufficient oxygen and no intervention either from active protection measures or from the Fire Service personnel.

(ii) *Flashover:* This is the transition between a localized and fully developed fire. Flashover can be assumed to occur when sustained flaming from combustible material reaches the ceiling and the temperature of the hot gas layer is between 550°C and 600°C. Following flashover the rate of heat release will increase rapidly (accompanied by a reduction in the oxygen concentration) until it reaches the maximum value for the compartment.

(iii) *Fully developed phase (steady state):* This is the stage at which all the available fuel is burning. The maximum rate of heat release will be dictated either by the availability of oxygen (ventilation controlled) or the quantity and nature of the fuel (fuel bed controlled). This is the most critical stage of the fire in terms of structural damage and failure of compartmentation.

(iv) *Decay phase (cooling):* After a period of sustained burning (typically once 70% of the fuel has been consumed), the fire will decay with temperatures reducing over time.

Figure 1.2. The Fire Spreading Because Of Existing Oxygen (Schleich, 2010).

A structure under the fire and at the high deformation; moments and forces are transferred directly to the connection and to the other assemblies. Therefore, the connections can have an essential effect on the survival time of the structures.

Dissimilar from the standard fire curve, a real fire also is shown in the above table for comparison. It is important and needed that to evaluate not only the effect on the structural resistance during the heating phase, but also the high cooling strains in the joints induced by distortional deformation of the heated elements during the decay phase.

Steel framing construction was early designed on the assumption that the concrete slab acts independently of steel in resisting loads. No consideration was given to the composite effect of the steel and concrete acting together. The structural capacity of a beam or column section is limited by one of three limit states: Full section yielding; yield capacity flexural and lateral torsional buckling. However, if the temperatures are high enough to reduce the yield stress, the resulting thermal gradient will change the plastic P- M interaction curve shape changing with time (Maria, Garlock and Spencer, Quiel).

Elastic materials shows the elastic behavior in a loading case, and Hooke's law relates the stress of these material with the strain in the elastic region. Through experiments, it can be understood that most materials show a spring behavior in that the force is proportional with the displacement. However, dissimilar with a spring behavior, an element has a cross-sectional area that effects the displacement. Therefore, it is simplier to relate the stress to the strain (Ferdinand, Russell, John, 2002). According to stress- strain formula $\alpha = E\varepsilon$; E is the material property that represents the stiffness of the material (called Young's Modulus). Young's modulus is determined through experiments and are commonly listed in engineering handbooks. ε is the strain that used to measure the deformation or extension of a material that is subjected to a force. The strain of a material is generally defined as the change in length divided by the initial length (Ferdinand , Russell, John, 2002).

Because of its advantages such as lightweight, high strength and good ductility steel is used in buildings and many structures. However, the material thermal properties for thermal analysis including the thermal conductivity, specific heat, and density; and the material mechanical properties for the structural analysis including the yield strength, Young's modulus, stress-strain relationship and the expansion coefficient are very crucial data for the structural fire engineering (ASTM, 2007). In Figure 1.3, material mechanical properties are defined basically by the stress-strain relationship for a standard type steel under tension stress at ambient temperature. The basic linear elastic stress- strain relationship assumes that there is no plastic deformation in the material. However, if a material is highly strained, if will permanently deform and when a material deforms, the stress- strain relationship is no longer linear.

The stress and strain relationship of a material is known as that a stress-strain curve. It is different for each material and is calculated by several testing the amount of deformation (strain) at particular periods of a diversity of loadings (stress). Stress-strain curve displays many of the properties of a material including data to define the Modulus of Elasticity, E. Figure 1.3 shows a common plot is known as the 'Stress-Strain Curve' for the stress as a function of the strain and each material has a unique curve. The initial curve is a straight line reflecting the linear relationship between the stress and strain and called as "Elastic" phase. Hooke's law is only implemented in this phase (ASTM, 2007). If the material continues to be stressed after past its elastic limit the material will be permanently deformed and plastic deformations will start. It means that the material has yielded and will be deformed easily with little additional load. After yielding, the material cross-section will become smaller which is called "necking". Because the cross-sectional area is reducing, the effective stress derives. However, many times the stress calculations will use the original cross sectional area (called the nominal area). This situation will cause the stress-strain curve appear to decrease (ASTM, 2007).

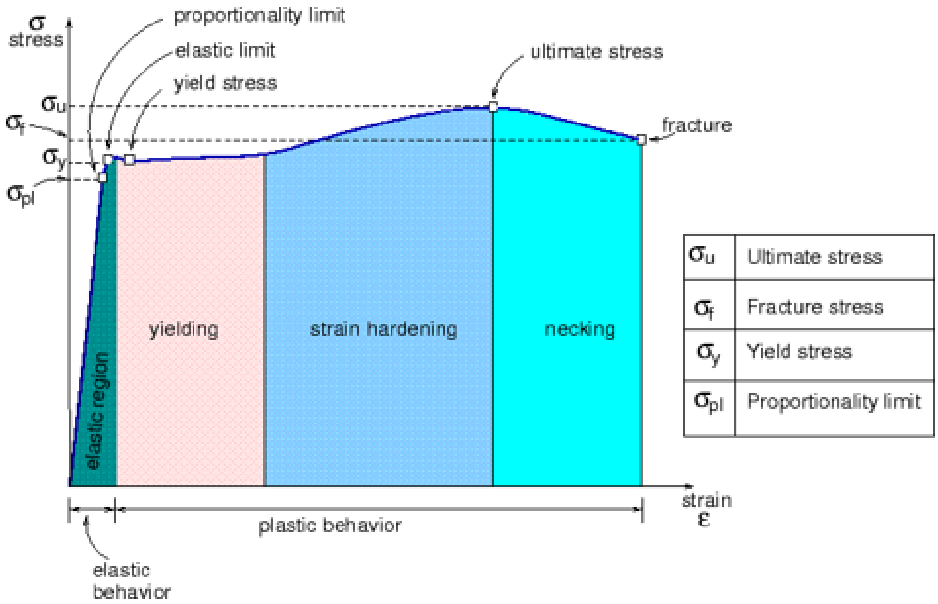

Figure 1.3. Stress-Strain Relationships for Steel at Ambient Temperatures (ASTM, 2007).

Unprotected steel members have some disadvantages in structural buildings; one of them is that steel doesn't perform properly compared to other construction materials if exposed to fire. Because of its high thermal conductivity, its strength and stiffness is lost progressively when exposed to a fire. In this case, depending on the applied loads and situation of supports and fire load, structural elements can have excessive deformation and failure. Elevated steel temperature depends on the severity of the fire, the area of steel exposed to the fire and the amount of applied fire protection (Sarraj, 2009). Although the melting point of steel is about 1500^{o}C, only 23% of the ambient-temperature strength remains at 700^{o}C, whereas, at 800'C this strength reduces to 11%, and at 900^{o}C to just 6% (SSEDTA, 2001).

Eurocode 3 Part 1.2 defines the stress-strain curve for a carbon steel at elevated temperature as seen in Figure 1.4. Reduction factors are described for yield strength, proportional limit and the Young's modulus in order to define the stress-strain relationships at elevated temperatures. For temperatures below 400^{o}C, overall buckling does not occur according to Eurocode 3 Part 1-2.

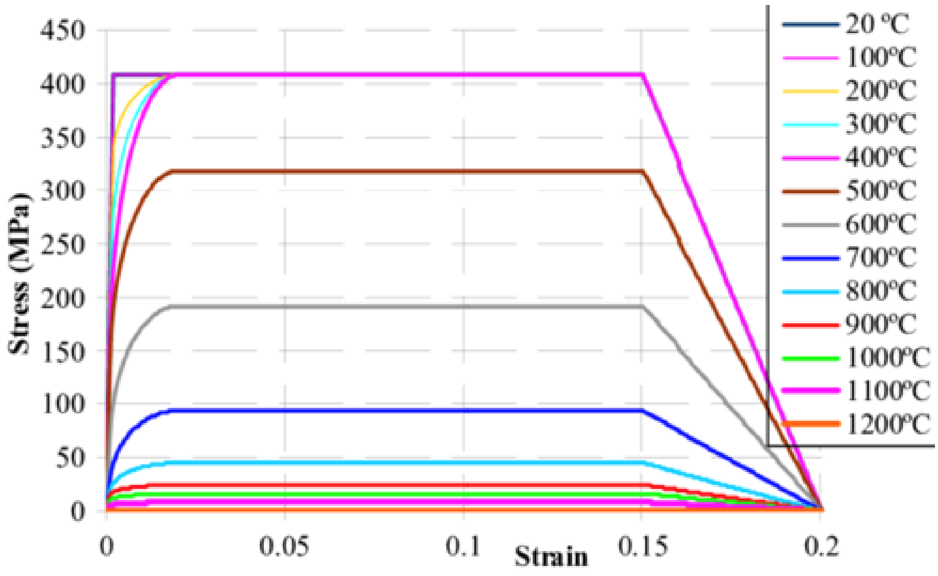

Figure 1.4. Stress-Strain Relationships for Steel at Elevated Temperatures (Parkinson, Kodur, 2006).

The numerical results on the response of fin-plate steel connections under fire conditions are relatively recent and limited. The development of tensile and compressive in-plane forces will influence the yield line moments grown in the slab with reductions in bending resistance occurring in the tensile zone. Bending resistance occurs in the tensile zone and the bending resistance of the yield lines improves in the compression zone. The emerging collapse behavior of a tall building generally has been studied after the collapse of World Trade Center Towers in 2001 (Hoffman, 2004; Gross, 2005). A challenging analysis of moment resistance and dual system steel frame due to a column loss are conducted by Kim *et al.* (2011), and they concluded that the beam yield strength is the most crucial design parameter. A detailed three-dimensional composite floor structure with shear connections are modeled by Sadek *et al.* (2008) and Alashker et al (2010). One-floor composite structure is promptly subjected to a column loss and it is found out that the metal deck carries the tensile forces in the composite floor. Further, Yu et al have discovered the effect of shear connection capacity is limited (2009). The nonlinear dynamic behavior is observed due to column loss at different locations and increased gravity loading. The dynamic behavior of a 20-story steel building is studied with several different column locations by Fu (2009). Izzuddin *et al.* (2009)

and Vlassis *et al.* (2009) organized a theoretical groundwork for steel frames due to a sudden column loss. A steel building might also collapse due to a severe event such as a fire hazard. The underlying mechanism of tall buildings when subjected to fire has been investigated by Lange *et al.* (2012). It was found that the fundamental reason of collapse is the outward and inward deflection of the columns above and below the heated floor. Sun *et al.* (2012) studied the vulnerability of steel braced frames and the progressing of plastic hinges at elevated temperatures. Although it is known that steel braced frames are more resistant to fire hazard, it is ambiguous that in this type of structure steel shear connections will expose the fire or not (Astaneh- Asl *et al.* 2002; Selamet and Garlock 2010).

Steel shear connections have an important role for the structure to be stable, especially to lateral support for the columns (Chung and Ip 2001; El Rimawi *et al.* 1997; Gann 2008). In this situation, inadequate connection stresses or inadequate ductility against the forces that happen in fire hazard can bring on the collapse in the structure. The deformation of the beams or columns under the fire can be obtained only by improving rotation capacity of the steel shear connections and by designing more ductile structures.

1.3.2. Steel Connections under Fire

Steel connection is an important part of any steel building. They provide strong links between the other principal structural components, and therefore contribute to overall building stability. Early steel connections were fabricated with rivets, but these have seldom been used since 1950's. Today steel connections are fabricated mainly with bolts or welding. The behavior of the connection region of a structural frame is complex because of the wide range of parameters involved. Based on that, many researchers conducted in the past, and research still underway has attempted to improve understanding of steel connection behavior under fire and design methods.

When steel is exposed to fire, it loses both its strength and stiffness. Also, steel expands when it is heated and contracts on cooling phase. Furthermore, the effect of restrains to thermal movement can cause high strains in both the steel members and the associated connections. Although steel has some superior features such as high strength, durability, easy manufacturing, and high elasticity modules; high heat-sensitivity of them is the biggest problem for steel elements.Steel is not a burning material but if temperature increases, its strength and elasticity modules decrease. Especially if temperature arrives over 200°C, its strength values clearly decrease. With insulation of steel elements in a composite structure with steel carrier system, the weakness of steel to high temperatures can be solved in a particular rate (Astaneh- Asl *et al.* 2002).

The usual approach to structural fire performance deals with considering the fire resistance of structures with the parallel design approach, which is the achieved by designing various components on isolation. In the latest design, performance for fire safety is given with the correspondence of the structural members as they grow, contract and perhaps fail under combined thermal and gravity loadings (El Rimawi *et al.* 1999). The British Research Establishment (BRE) in Cardington, U.K., performed several fire tests on a full-scale 8-story steel framed composite building (British Steel 1999; Bailey *et al.* 1999; Wald *et al.* 2006). These advanced researches proved that better prediction of structural performance could be gained by considering structural interaction, end restraint, continuity, and membrane action. In similar studies showed that the connection behavior affects the simple member behavior, which, in turn, affects the behavior of the whole structural system (Beitel and Iwankiw 2002; Selamet and Garlock 2012). Among the research areas on structural fire safety, the research on fire resistance of steel connections is rare. However, connections during a structural fire hazard brace the columns for stability in building frames (El-Rimawi *et al.* 1997; Gann 2008). The problem of prediction the forces and moments on a structural connection during a fire originates from the irregular geometry, the complex interactions between the connection components such as bolts, plates and angles, the global buckling in the unrestrained beams (Vila Real *et al.* 2008; Becque and Rasmussen 2009) and the local buckling instabilities near the connection region (Galambos 1988; Schafer and Pekoz 1998; Heidarpour and Bradford 2008; Maljaars *et al.* 2009). Kirby (1995) also observed that the bolts behave differently when compared the hot-rolled steel members at elevated temperatures.

Studies related to modeling the connection behavior at elevated temperatures were developed by Liu for the first time (1996). Liu reviewed beam members and beam to column connections with different loadings and structural continuity. Yang *et al.* (2000) modeled double angled connections and tested them under a combined shear and tension loading. Astaneh *et al.* (2002) carried out studies on the behavior of single plate shear connections at ambient temperature. Al-Jabri *et al.* (2006; 2008) analyzed the behavior of flush end plate steel connections at elevated temperatures using the finite element software Abaqus (DS-Simulia 2008). Bursi and Jaspart have done detailed research on finite element modeling of bolted end plate steel connections and proposed that three-dimensional models are superior in estimating the connection capacity (Bursi, and Jaspart, 1998). Van der Vegte and Makino (2004) compared the implicit and explicit analysis techniques when solving the bolted connection models and concluded that the explicit analysis provided an easy solution to highly nonlinear problems but the results should be investigated more carefully. Another difficulty in assessing the fire performance of connections is the effect of the connection on the internal forces, moments and the rotation at the connections (Moss *et al.* 2004; Wald *et al.* 2006; Santiago *et al.* 2010).

While important and noticeable studies have been conducted on the overall structural response of steel buildings under the fire, the behavior of connections subjected to fire is not well-studied and analyzed in detail. Connections are very curial elements for maintaining the integrity of a structure such as steel elements. When a structural steel is subjected to elevated temperatures, both its strength and stiffness will decrease. At temperatures above 100^{o}C, the stiffness will start to decrease. The strength will start to reduce at temperatures above 400^{o}C. Further, the expansion of steel will cause crucial geometric changes in structural systems which could cause large deformations and failure depending on the fire severity, the amount of fire protection and the area of steel exposed to fire. Thus, the behavior of steel structures could be significantly affected at elevated temperatures during a fire.

2. STEEL CONNECTIONS ON THE STRUCTURAL SYSTEM

Steel buildings generally collapse after a fire event. Recent structural collapses caused by fire have focused attention on research about fire safety in building design. When flooor systems in steel buildings are exposed to fire, changes in the geometry of the structure are apparent due to the large axial loads, moments and shear from gravity loading. In structural fire engineering, steel connections play an important role in maintaining the integrity of a structure, studying how these connections are affected by a fire event is crucial. Fire experiments are necessary to investigate the way connections behave in a real fire event, however, using a fire furnace is very expensive. A more cost effective way for investigating how steel connections perform in a fire is the finite element method. One of problems with using the finite element method is the convergence difficulties occur between the surfaces and edges or corners. (Selamet, 2011).

Researching how connections perform in fire events is important in steel frame performance-based design because of the lack of knowledge in predicting how steel structures behave under fire conditions. The finite element model gives appropriate results in estimating the ultimate load capacity of steel because of the unpredictable behaviour of steel when exposed to high temperatures. Forces that occur from fire, along with the instability of fully or partially restrained beams and the deformations that result from elevated temperatures make the finite element model an important numerical analysis mechanism on steel connections (Selamet, 2011).

There are two types of steel connections that are mostly used in a structure; moment connections designed as with full-moment resistance and pin connections designed as with non-moment resistant. This theoretical assumption has been verified by the experimental studies on various steel connection types and takes part in Eurocode 3 (Eurocode 3, 2005).

2.1. Moment Connections

Moment connections are generally more rigid and able to withstand a higher moment load by comparison with shear connections. On the other hand, due to more material are used to assemble the connection, it will not be cost-effective to use a moment connection than the shear connection. Only one or two moment connections is used in a steel structure to minimize the cost. The major dissimilarity is how the connection behaves between the members. In a moment connection, the rotation will be start after a while and due to this movement of moment connection it will cause the connected member to bend. However, in the case of a shear connection, this kind of behavior in the system will not be seen because of the shear connections' freedom to rotate. A shear connection will allow rotation and prevent the moment force to transfer.

Moment connections are divided into 3 categories. Extended end-plate connections, T-sub connections, and flush end-plate connections as seen in Figure 2.1 (Sarraj, 2009).

(i) Extended end-plate connections: They are classified into two types as with an end plate extended on the tension side only and both the tension and compression sides.
(ii) T-sub connections: This type connection is a fabrication of two T sections bolted to the columns and the beam flange.
(iii) Flush end-plate connections: It is an end-plate welded to the beam end along both the flanges and web in the fabricator's shop, and bolted to the column flange in the field.

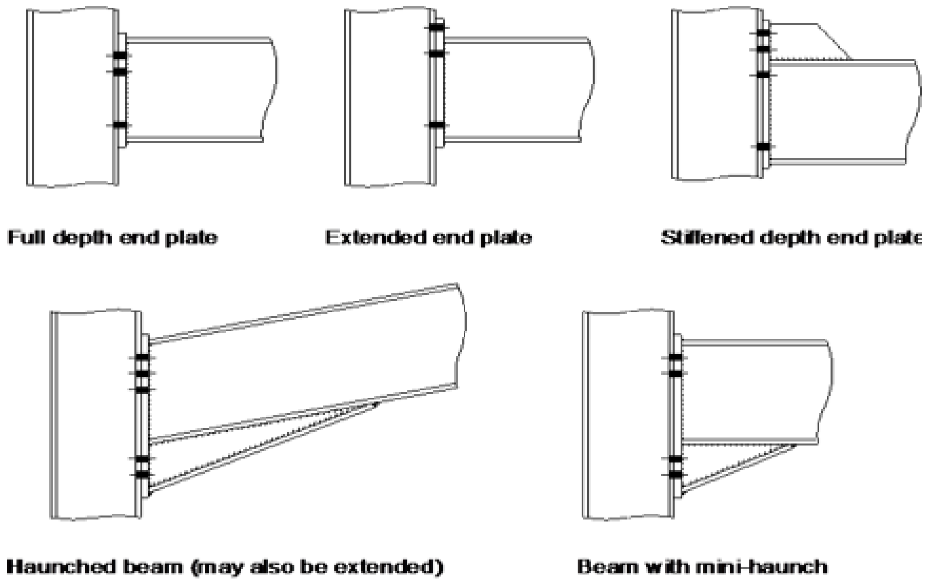

Figure 2.1. Moment Connections (Sarraj, 2009).

2.2. Simple Connections

Simple connections are generally known as pinned connections are divided into 6 categories as header plate steel connection, top and seat angle connection, double web angle steel connection, top and seat angle with double web angle connection, single web angle steel connection and fin-plate steel connection (Sarraj M, 2009). In Figure 2.2, different type of pinned connections include is shown.

(i) Header plate steel connection: This type of connection consists of an end plate whose length is less than the depth of the beam, welded to the beam and bolted to the column.

(ii) Top and seat angle connection: This connection type is used to provide lateral support to the compression flange of the beam and the seat-angle for transferring only the vertical reaction of the beam to the column.

(iii) Double web angle steel connection: Double web angle steel connection consists of two angles bolted to the column and the beam web.

(iv) Top and seat angle with double web angle connections: This type of connection is a combination of a top and seat angle connections and a double web angle connection. Double web angles are used to improve the connection restraint characteristics of top and seat angle connections, and for shear transfer.

(v) Single web angle steel connection: Single web angle steel connection consists of one angle bolted to the both the column and the beam web.

(vi) Fin-plate steel connection: A fin-plate connection consists of a single plate welded to the column flange-web or beam web in the workshop and bolted to the beam web in site. This connection type is generally most used connection type in a structure by reason of being both economic and easy to fabricate compared to other connection types. Because of this advantages of the fin-plate steel connections have, this type of connections behavior have been concantreted on in this thesis study.

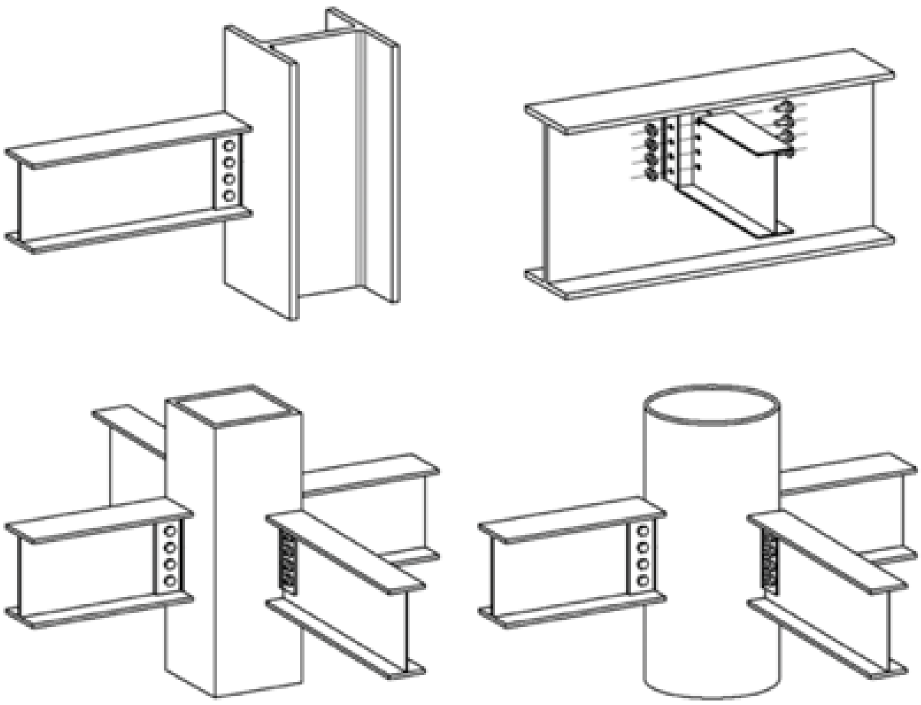

Figure 2.2. Different Joint Configurations (Sarraj, 2009).

Simple connections are assumed to transfer only translations and have negligible resistance to rotation. Therefore, significant moments at the ultimate limit state cannot be transferred with the simple connections to neighboring structural member opposite with the moment connections. According to that, the design and modelling of multi-storey braced frames is generally created by considering this effect of a simple connection. The beams are commonly designed as simply supported and the columns are designed for axial loads and the limited moments. The design of a simple connection is based on Eurocode 3 Part 1-8 and its accompanying National Annex.

2.2.1. Fin-Plate Steel Shear Connections

Fin plate shear connections are most common connection types between the other types of connections. They are generally called different ways such as web side plate or shear tab plate. Commonly; fin plate shear connections consist of a single plate, which is welded to the supporting member as shown in Figure 2.3. The supporting member is generally a flange, or web of a column, or web of a beam. Fin plate connections have some advantages such as being economical to fabricate and simple to erect. These type connections are most popular connections that used in a structure, as they can be the quickest connections to erect and overcome the problem of shared bolts in two-sided connections.

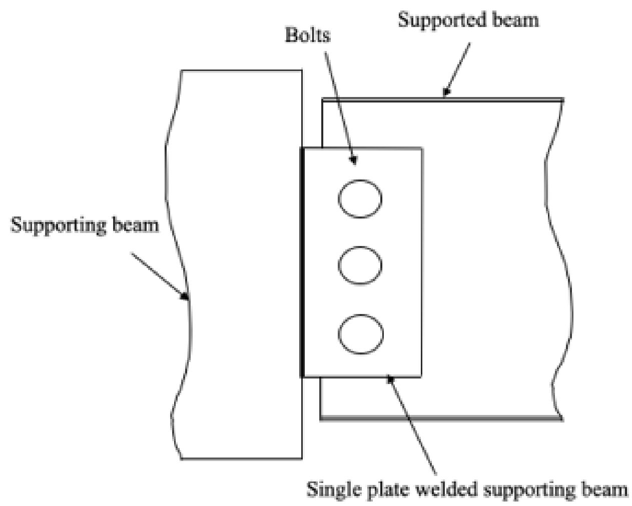

Figure 2.3. A Typical Beam-to-Beam with a Fin Plate Connection.

In the design and modeling phases of a fin plate connection, there is a crucial point is that to identify the appropriate line of action for the shear force. There are two possibilities: either the shear acts at the face of the column or it acts along the center of the bolt group connecting the fin plate to the beam web (Richard, and Elsalti, 1991). For this reason, both critical sections should be controlled for a minimum moment value and then checked for the resulting moment combined with the vertical shear. The rotational capacity of fin plate connections comes from bolt deformation in shear. Fin plate connections generally tend to twist and fail by lateral torsional buckling (Richard, and Elsalti, 1991).

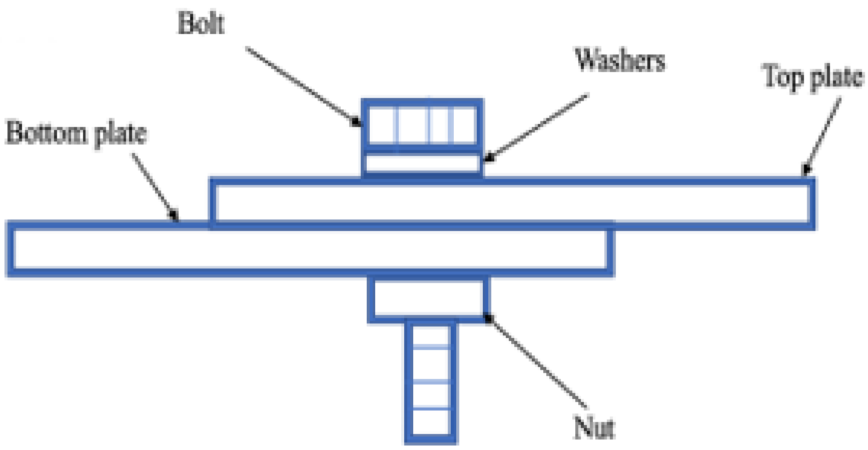

Figure 2.4. Illustration of a Lap Joint with a Single Bolt.

Fin-plate steel shear connections are used in the steel structural buildings to conduct the reactions of a supported structural beams or columns to its supporting members. To be more specific, a shear connection is the part that transfers the loads between connection components via bolts. In according with that, Figure 2.4 demonstrates a lap joint with a single bolt case in detail. These type of connections can transfer significant end moments to the beam and its supporting member. Fin plate steel shear connections also will be exposed to bending effects under the increasing forces or temperatures as all types of connections. It is very difficult to observe and to find the possible failure modes of a single lap joints under the loadings. However, it is classified as 4 failure modes for a single shear lap joint as seen in Figure 2.5 (Ballio, and Mazzolani, 1983).

(i) Net section failure: This type of failure mainly occurs when the ratio of hole diameter to plate width is high.

(ii) Bearing failure: This mode of failure is not as critical as others, because the connection does not generally lose any load-carrying capacity.

(iii) Shear-out failure: This failure type is caused by shear stress acting through the plate-shearing path and occurs mainly in joints where the distance between the hole edge.

(iv) Bolt shear failure: This type failure is the main failure mode in axially loaded joints with plates stronger than the bolts.

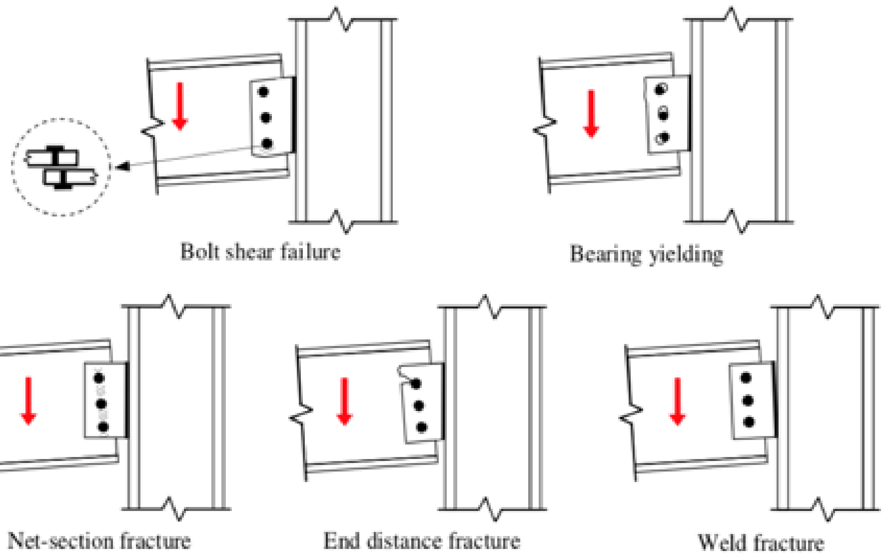

Figure 2.5. Failure Types of a Fin Plate Connection.

3. COMPONENT BASED METHOD

3.1. Introduction

Recently, an innovative mathematical model called 'Component-Based Method' has been developed in order to analyze the fundamental behavior of fin-plate shear connections in a structural system under the fire conditions. The key aspect of the component method is that it characterises the force-displacement properties of each active component at any temperature as a non- linear "spring". This scientific approach has progressively achieved large popularity because of its advantages compared to the other analytical mathematical methods. It may be used for any joint type, various connection configurations and loading situations. Component based method is used to obtain moment-rotation characteristics of connections and takes part in Eurocode 3. It also demonstrates the load-deflection curve of each component that is properly defined in the research.

In Component-Based method, first of all, the active joint components of the system are identificated and then, the axial force-displacement response of these components are characterized. Finally, by using springs and rigid links, the model of the system is assembled numerically (Sarraj, 2009). The illustrated method is shown in Figure 3.1.

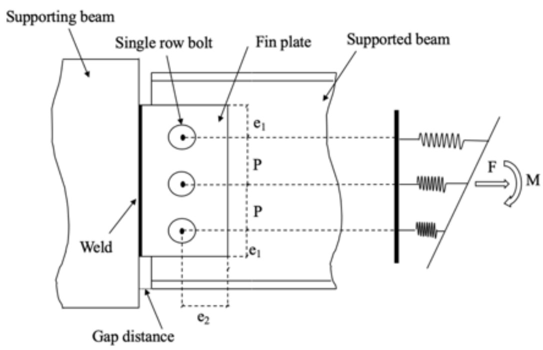

Figure 3.1. The Illustration of Fin Plate Connection Component-Based Model.

There are numerous studies regarding component-based method on steel connections, however the modelling of a beam with a fin-plate shear connection type by using component-based method has not received much attention. In the case of high temperatures, connections have a complex force situation such as moment, axial forces and shear forces. Depending on that, fin plate shear connections can have a big effect on the whole system. Very little information has been generated on the behavior and the resistance of a beam with a fin-plate shear connections under realistic fire conditions which includes both heating and cooling phases.

The aim of this section is to propose a component-based model built in ABAQUS software in order to investigate the behavior of a beam with a fin-plate connection type under realistic fire conditions. Abaqus is popular with academic and research institutions because it provides visual intrepretation at each step and mechanisms to visually shows deformations.

3.2. Problem Description

In a realistic fire scenario, the steel temperature reaches to its maximum value of 1000^{o}C and then decreases to the ambient condition (20^{o}C). During the analysis, the temperature of the connection is kept at 20^{o}C because it is assumed that the connection is fire protected. Three row one column bolted beam connection model is used in order to analyze the effect of connections at elevated temperatures. The model consists of bolts, plate, supporting beam and supported beam. The details about the numerical model is given in Figure 3.2.

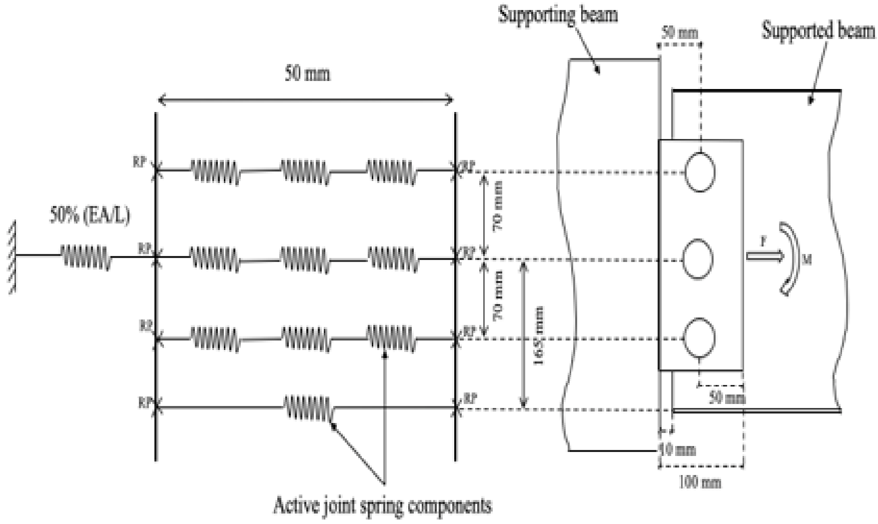

Figure 3.2. Corresponding Component-Based Model.

In the corresponding model, IPE330 (S275) Structural Steel Beam is used. The dimensions of the beam is 330 x 160 x 7,5 mm and also the flange thickness of the beam is 11,5 mm. The thermal and mechanical material properties of the beam used in the proposed model under ambient temperature is given in Table 3.1. Steel shows different material characteristics with increasing temperatures. Figure 3.3 shows the stress-strain relationships of S275 Steel at elevated temperatures which are used as an material input in the numerical model (Eurocode 3, 2005).

Table 3.1. Material Properties of S275 Steel Beam under Ambient Temperature.

Description	**Variable**	**Value**
Modulus of Elasticity [MPa]	E	210
Poisson's Ratio	ν	0.3
Shear Modulus [MPa]	G	79.3
Yield Strength [MPa]	σ_y	275
Ultimate Strength [MPa]	σ_u	343
Coefficient of Thermal Expansion	σ_α	1.2 x 10^{-5}

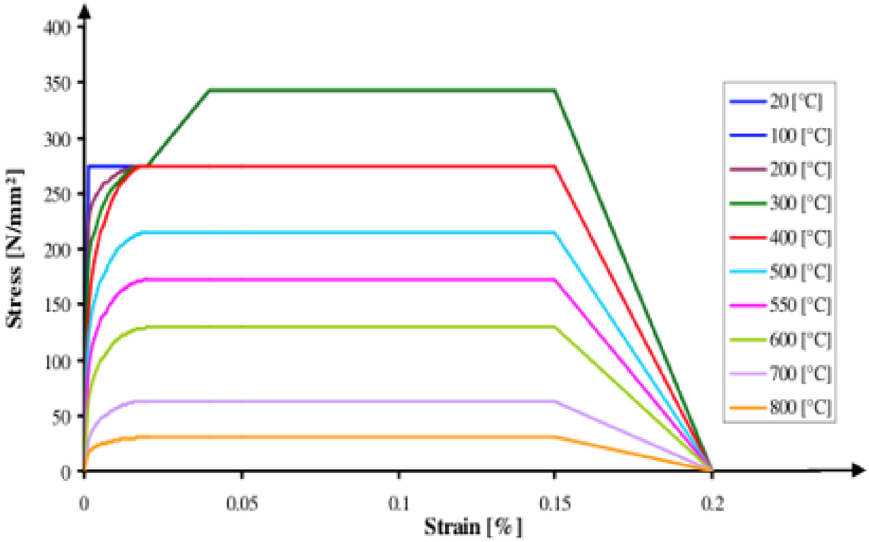

Figure 3.3. EC3 Stress-Strain Curves for S275 Steel at Elevated Temperatures.

The Component Based Method is an adaptable approach for calculating the axial stiffness of the joints both during the heating and cooling phases. The model is applied for the realistic fire conditions and the beam is used with shear connection and axial restraint for that purpose. In order to represent the axial stiffness of neighboring structural elements, elastic springs are attached at the heated beam ends. Axial restraint spring stiffness can be taken as 50% of the beam axial stiffness (see Equation 3.1) at room temperature of 20^{o}C. According to that, the beam axial stiffness is calculated as;

$$k = \frac{EA}{L} \tag{3.1}$$

where E is elastic modulus, A is the cross-sectional area, and L is the length of the used steel beam. Axial restraint spring stiffness can be found as 65730 kN/m. In addition to the semi-rigid connection type (fin-plate shear connection), idealized pinned-pinned and fixed-fixed beam end boundary conditions are analysed for comparison as demonstrated in Figure 3.4.

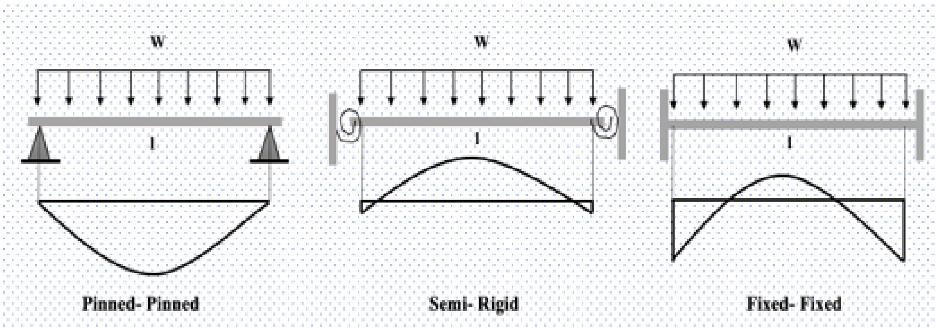

Figure 3.4. Idealised Connection Types That Used in This Study.

In numerical procedure, IPE330 beam is first designed for ambient conditions. The beam has a utility (demand/capacity) ratio of 85% of design plastic moment capacity (0.85 ϕM_p), which is 169 kN.m. The M_p at midspan is calculated using the formula in Equation 3.2, which shows the maximum value of a parabolic moment diagram (see Figure 3.4). At 85% capacity, the beam will have 13.5 kN/m uniformly distributed load. However, for fire conditions, the total distributed load is reduced to 65% with load reduction factor η_{fire} as designated by Eurocode 1 Part 1-1 (Eurocode 1, 2002) as shown in Equation 3.3. The new distributed load for fire conditions is therefore $w_f ire$=8.8 kN/m.

$$\mathrm{M}_{\max} = \frac{wl^2}{8} \tag{3.2}$$

$$w_{fire} = \eta_{fire}.w_{ambient} \tag{3.3}$$

where w is the distributed load per unit length (kN/m) and l is the span length (m), η_{fire} is the load reduction factor.

3.3. Component Model for Fin Plate Shear Connection

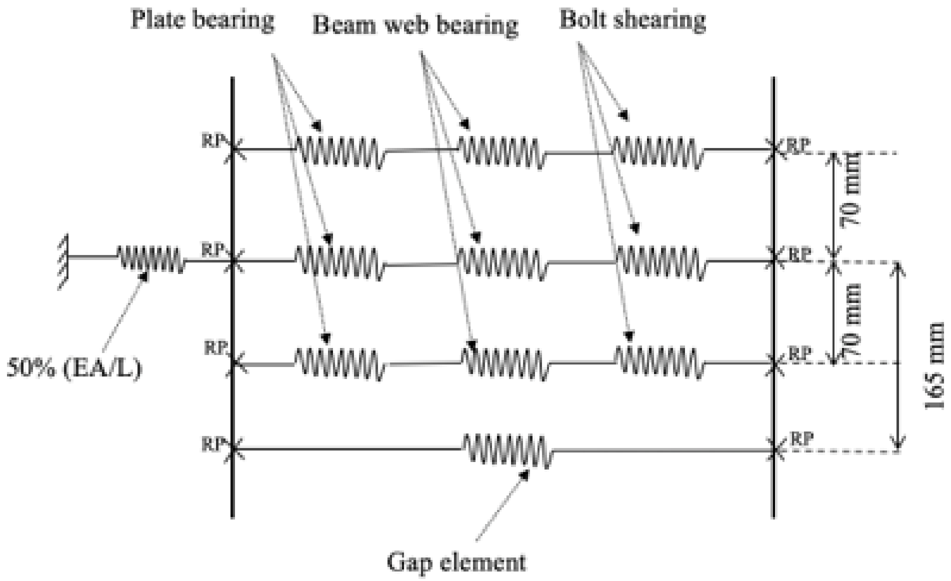

Figure 3.5. Spring Assembly and Active Joint Components for Fin-Plate Connections.

Component model for a fin-plate shear connection is illustrated in Figure 3.5 with three bolt lines. The active joint components for this type of configuration, according to Eurocode 3, are; fin plate in bearing, beam web in bearing, bolts in single shear, and the gap element. As shown in the figure, these joint components are made up of extensional springs in the model. It is proposed that this spring assembly is used to describe the full joint performance.

In numerical analysis, there is a requirement to obtain a load-deflection behavior of each spring component. In the following sections, the calculations are done for that purpose for plate bearing, web bearing and bolt shear components, and also the force-deflection behavior of gap element is given in detail. In the analysis, it is assumed that the fin plate and the supporting structural member have a perfect bond, i.e. no weld material is modelled. The effect of friction occurred by pretensioned bolts is ignored due to possible elongation of bolts and thereby reduction in friction at higher temperatures.

3.4. Plate Bearing and Web Bearing Spring Components

The load-deflection behavior of plate bearing and web bearing spring components have been calculated with the same equations. The only different parameter between two components is the thickness value. While the thickness value of plate is taken as 10 mm, the beam web is only 7.5 mm thick. Richard non-linear equation has been used to simulate the load deflection behavior of these plate/web bearing springs as shown in Equation 3.4 (Richard, 1991).

$$\frac{\mathrm{F}}{\mathrm{F_{b,Rd}}} = \frac{\Psi\Delta\beta\mathrm{K}_i/\mathrm{F_{b,Rd}}}{\left[1+(\Delta\beta\mathrm{K}_i/F_{b,Rd})^{0.5}\right]^2} - \frac{\Phi\Delta\beta\mathrm{K}_i}{\mathrm{F_{b,Rd}}} \tag{3.4}$$

where F is plate load $[N]$; F_b, R_d is nominal plate strength $[N]$; Δ is hole elongation [mm]; β is steel correction factor (for typical steels taken as β =1.0); and Ki is initial stiffness [N/mm] which can be calculated with Equation 3.5. Table 3.2 gives the equations used to find the plate and web bearing stiffnesses. The temperature effect was considered by applying the strength reduction factors of Eurocode 3 part 1-2 to the material ultimate stress. These stress values and other curve fitting parameters can be shown in Table 3.3.

Table 3.2. Calculation of Plate and Web Bearing Stiffnesses (Richard, 1991).

Stiffness [N/mm]	**Symbol**	**Formula**
Bearing stiffness	K_{br}	$\Omega t_p f_y (d_b/25.4)^{0.8}$
Shearing stiffness	K_v	$6.67 G t_p (e_2/d_b - 0.5)$
Bending stiffness	K_b	$32 E t_p (e_2/d_b - 0.5)^3$

$$\mathrm{K_i} = \frac{1}{\frac{1}{\mathrm{K_{br}}} + \frac{1}{\mathrm{K_v}} + \frac{1}{\mathrm{K_b}}} \tag{3.5}$$

Table 3.3. Plate and Web Bearing Curve Fit Parameters and Stress Values at Various Temperatures.

Temp.[^{o}C]	f_u, θ	f_u, θ	Ω	Ψ	Φ
20	445.00	1.0xfu	145	2.1	0.012
100	343.75	$1.25xf_u$	180	2.0	0.008
200	343.75	$1.25xf_u$	180	2.0	0.008
300	343.75	$1.25xf_u$	180	2.0	0.008
400	275.00	$1.0xf_u$	170	2.0	0.008
500	214.5	$0.78xf_u$	130	2.0	0.008
600	129.25	$0.47xf_u$	80	2.0	0.008
700	63.25	$0.23xf_u$	45	2.0	0.008
800	30.25	$0.11xf_u$	20	1.8	0.008

3.5. Bolt Shearing Spring Component

There is limited information in the literature about the bolt's case in a single shear under both ambient and elevated temperatures. However, Eurocode 3 Part 1.8 represents some equations for bolt shear at ambient and elevated temperatures. In order to represent the bolt shear load-deflection data at elevated temperature, Ramberg-Osgood expressions are used as shown in Equation 3.6 (Ramberg, Osgood, 1943).

$$\Delta = \frac{\mathrm{F}}{\mathrm{K_{v,b}}} + \Omega\left[\frac{\mathrm{F}}{\mathrm{F_{b,Rd}}}\right]^{\mathrm{n}} \tag{3.6}$$

where Δ is relative bolt deflection [mm]; F is the corresponding level of shear force [N]; $K_{v,b}$ is temperature dependent bolt shearing stiffness [N/mm] can be calculated as given Equation 3.7; $F_{v,Rd}$ is temperature dependent bolt shearing strength [N] can be calculated as given Equation 3.9 (Ramberg, Osgood, 1943).

$$\mathrm{K_{v,b}} = \frac{\mathrm{kGA}}{\mathrm{d_b}} \tag{3.7}$$

where k is the shear correction factor (taken as 0.15 from Eurocode 3 Part 1.8); G is the temperature dependent shear modulus [N/mm^2] can be calculated as given Equation 3.8. E_θ is the temperature dependent elastic modulus can be taken from Table 3.5 (Eurocode 3 Part 1.8).

$$\mathrm{G} = \frac{\mathrm{E}_\theta}{2\,(1+\nu)} \tag{3.8}$$

$$\mathrm{F_{v,Rd}} = \mathrm{R_{f,v,b}} \times \mathrm{f_{u,b}} \times \mathrm{A} \tag{3.9}$$

where $R_{f,v,b}$ is the strength reduction factor for bolt in shear; n is the parameter defining the curve sharpness, (taken as $n = 6$ from Eurocode 3 Part 1.8); Ω is the temperature dependent parameter for curve fitting. In the Table 3.5, bolt shearing curve fit parameters corresponding to analysed temperature are shown.

Table 3.4. Temperature Dependent Elastic Modulus (Eurocode 3, 2005).

Temp. [^{o}C]	E_θ **[kN/mm^2]**
20	205
100	198.6
200	191
300	184.1
400	167
500	154.5
600	140.7
700	124.1
800	124.1

Table 3.5. Bolt Shearing Curve Fit Parameters corresponding to Analysed Temperature (Eurocode 3, 2005).

Temp. [^{o}C]	**20**	**100**	**200**	**300**	**400**	**500**	**550**	**600**	**700**	**800**	**900**
$\mathbf{R}_{f,v,b}$	0.580	0.575	0.538	0.500	0.426	0.323	0.234	0.139	0.061	0.041	0.019
F_v, R_d	145.7	144.4	128.1	125.6	107.0	81.1	58.8	34.9	15.32	10.3	4.77
$k_{v,b}$	184.26	184.26	165.8	147.4	128.980	110.56	83.84	57.12	23.95	16.58	12.43
Ω	2.5	2.8	2	2.2	2	2	2	1.3	0.6	0.7	0.02

Figure 3.6 shows the final load-deflection behaviors of plate bearing, web bearing and bolt shearing spring components. It can be understood from this figure that the responses of plate bearing and web bearing shows non-symmetric behavior in tension and compression loading. On the other hand, the bolt shearing has a symmetric load-deflection curve. The data taken from Figure 3.6 is used to simulate the spring components in the CBM numerical analysis.

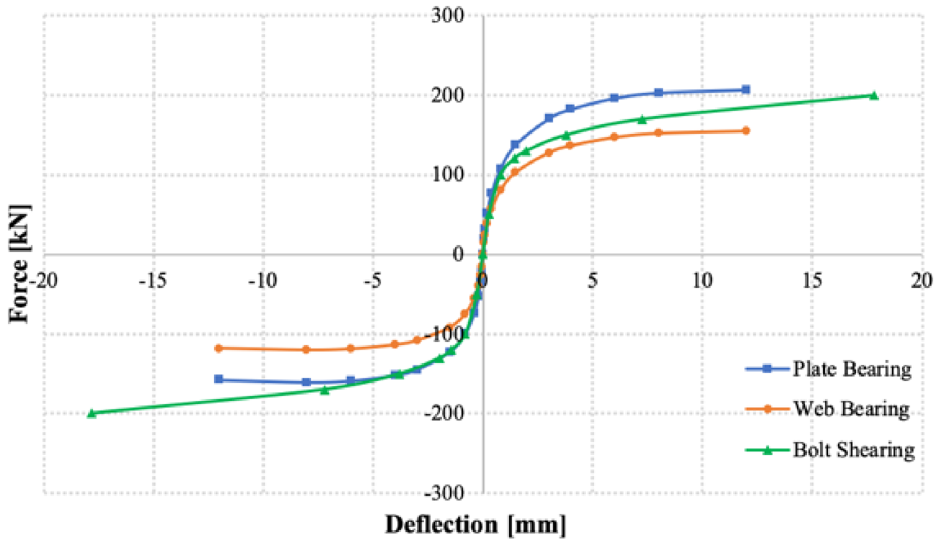

Figure 3.6. Plate bearing, beam web bearing and bolt shearing force-deflection comparisons by using mathematical expressions.

3.6. Gap Element Spring Component

In the proposed model, there is a 10 mm gap between the supported beam flange and the supporting beam web as shown in detailed Figure 3.7. Gap elements has an important role on this type of connections because they limit rotational capability of the connectionand thereby affect on the beam's axial force and moment. Figure 3.8 demonstrates the undeformed state and deformed state of the model in which this gap is closed. In model, gap element spring component is created in order to obtain accurate and realistic results.

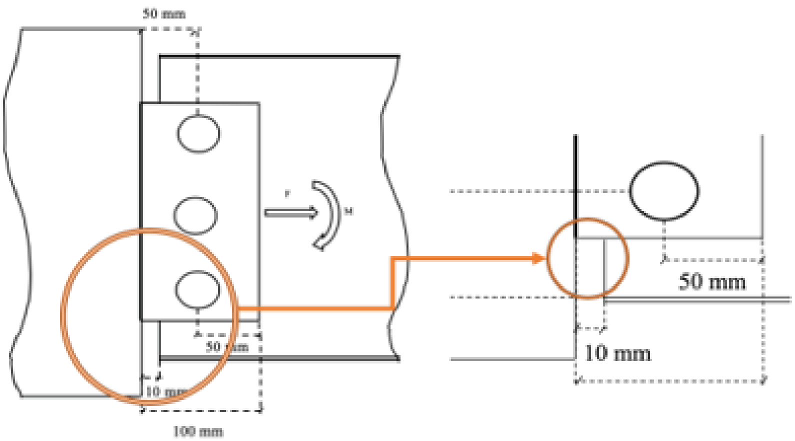

Figure 3.7. Gap Distance (10 mm) between the Beam Flange and the Web.

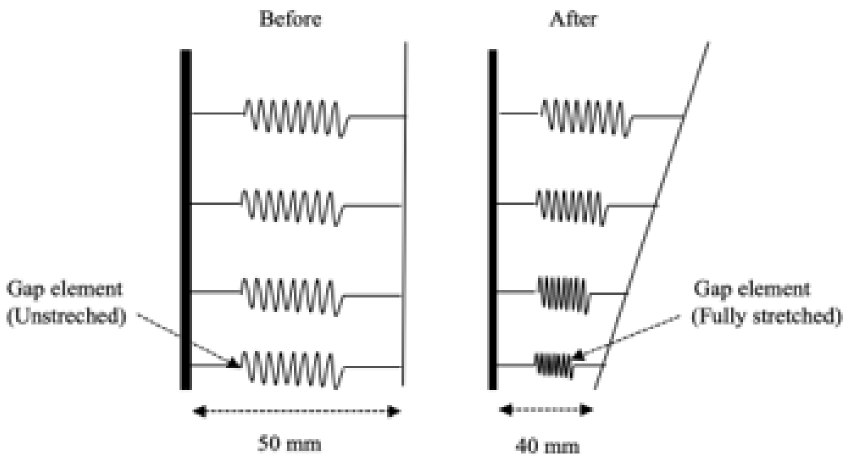

Figure 3.8. Description of Gap Element in Undeformed and Deformed States of the Corresponding Model.

The load-deflection behavior for the gap spring component is shown in Figure 3.9. According to the graph, it can be seen that the spring provides no resistance until the gap is entirely closed. After the point where the gap is closed, the force response reaches to infinity for this spring component. For numerical purposes, a larger values (close to 10MN) is taken to represent infinity.

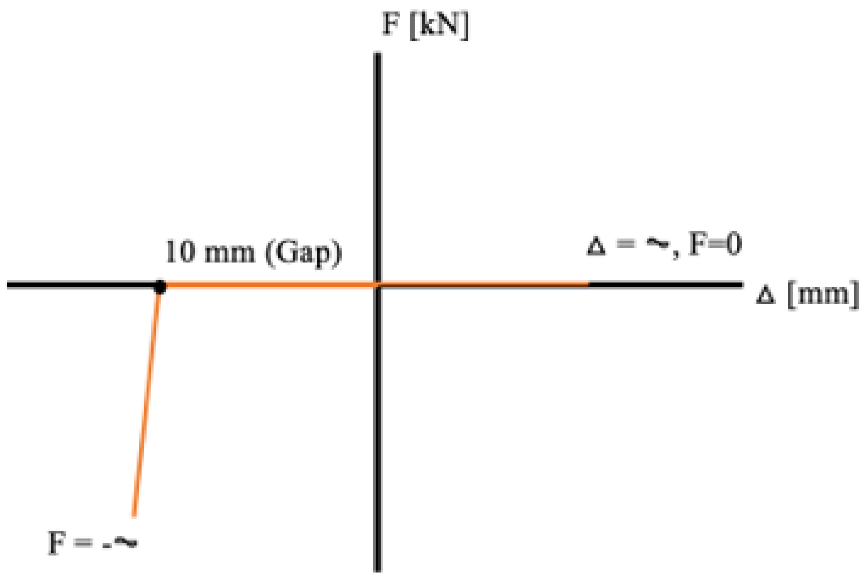

Figure 3.9. Force- Deflection Behavior of Gap Element.

3.7. Reference Points and Rigid Body Constraints

In this study, only half of the beam is modelled by considering the symmetry conditions. This is applied to model in order to reduce the run time. In the CBM connection model, all applied constraints and reference points are shown in Figure 3.10.

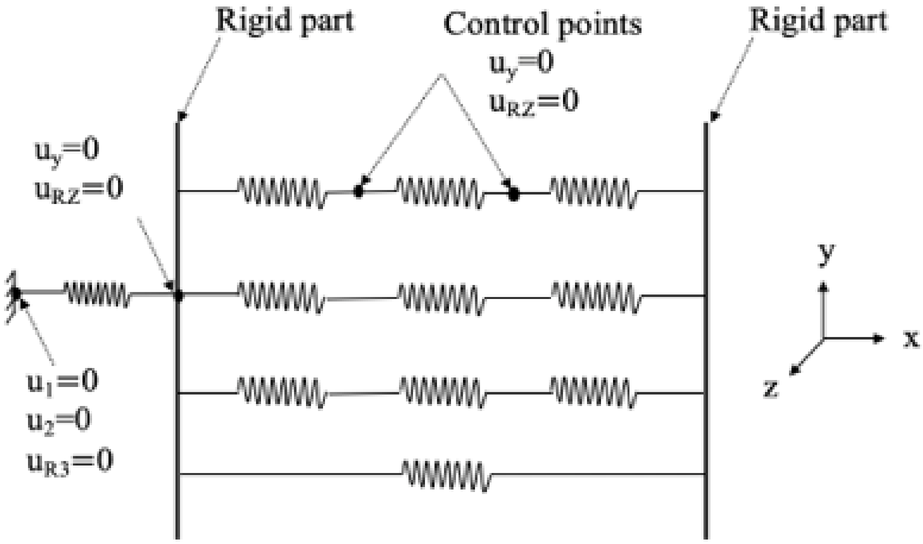

Figure 3.10. Boundary Conditions for the Component Based Model Connection.

3.8. Results and Discussion

The component-based method is applied with the details mentioned in previous chapter and the data taken from previous chapter is used as spring component characteristics in numerical analysis in ABAQUS software. The beam axial force, moment, deflection and rotation results are obtained from the analysis under a realistic fire scenario. In this section, it is aimed to understand and discuss the fire performance and behavior of the restrained beam with a fin-plate (semi rigid) connection.

3.8.1. Beam Temperature

The beam temperature distribution used in the analysis (see Figure 3.11) is taken from a fire experiment of a beam with fin-plate shear connection in a fire furnace (Selamet, Yolacan, 2017). This research was the first research-based structural fire resistance test on a steel-concrete composite floor in Turkey. The tested composite floor designed for a high-rise steel building was subjected to ISO-834 standard fire curve from the bottom surface for 105 minutes followed by 90 minutes cooling. The beam temperature-time data that obtained from this fire test is used in order to get an accurate and realistic results for the beam with a fin-plate connection under elevated temperatures.

Firstly, the temperature values increase up to 800^{o}C and cool down to 20^{o}C during the rest of the analysis as seen in Figure 3.11. According to this figure, the beam is heated from below, because of this reason the bottom flange temperature of the beam is higher than the top flange temperature of the beam.

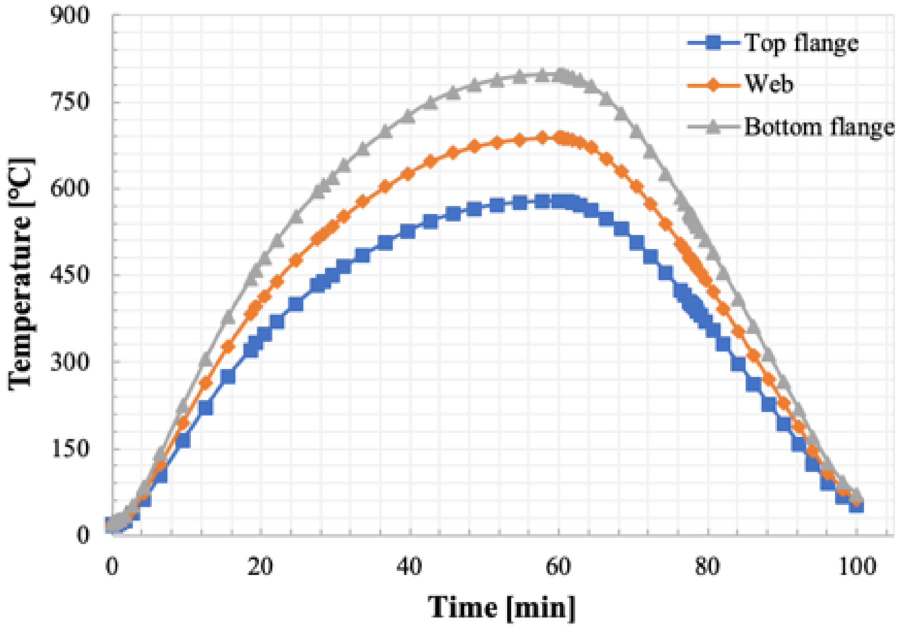

Figure 3.11. Beam Temperature-Time Plot (Selamet Yolacan, 2017).

3.8.2. Moment-Temperature Results

The moment results for three types of connections are given in Figure 3.12. As seen in the plot, the steel beam with pinned type connection gives almost zero moment at connection due to the free rotation. The peak values of moment for the beams with fixed and fin-plate connections can be evaluated as similar. However, for the fixed connection, the higher moment values are seen at the beginning of the process, while the moment outpout is seen after the gap element is closed for the beam with fin-plate connection type. In other words, the 10 mm gap distance in the fin-plate connection allows rotation and the translation until it entirely closes. The contact which is established between the beam flange and the supporting member is seen in Figure 3.12. The fin plate (semi-rigid) connection behaves as a pinned (shear) connection at first; but at around 150^{o}C of beam temperature (when the gap is closed), the connection picks up hogging moment (clockwise moment) and it quickly behaves as a fixed (moment) connection. It means that, in a fin plate connection, the axial force and the moment values increase dramatically after gap element is closed and the fin plate connection behaves like a moment connection.

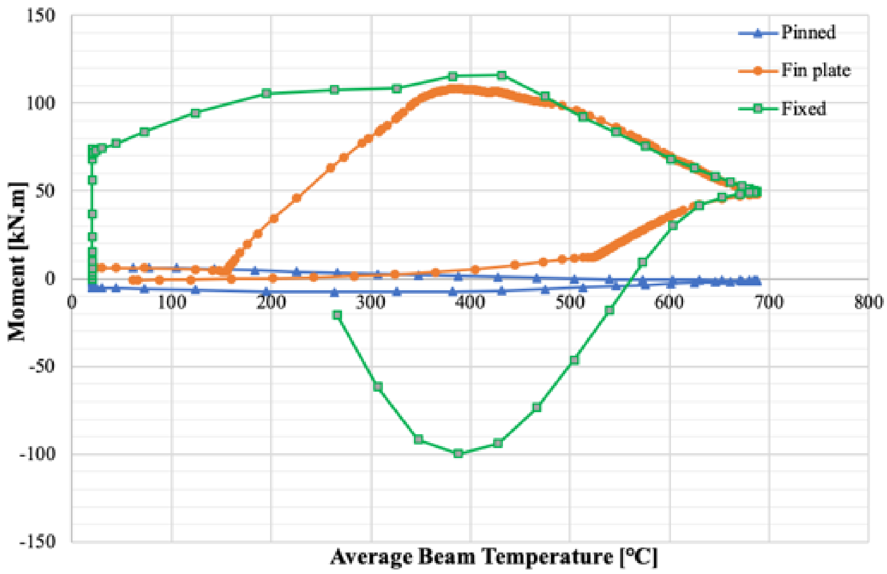

Figure 3.12. Moment vs. Average Beam Temperature.

When moment diagram is taken into consideration in Figure 3.13, it can be seen that the moment result of the beam with the fin-plate connection shows similarity with the pinned connection type in the cooling phase. Beam with a fin-plate connection demonstrates almost zero moment as with the pinned connection after 400^{o}C. This similar behavior occurs because fin-plate connection does not have a contact point on the top flange hence there is no resistance to rotation during cooling. In short, the component based method enables to model a semi-rigid connection which during heating acts as a idealized pinned connection (until the contact is established), then as a fixed (moment) connection at contact, and lastly as a pinned connection again during the cooling phase.

3.8.3. Axial Force-Temperature Results

Beam axial force and the average temperature curve for three types of connection are shown in Figure 3.13. In addition to fin-plate connection, results for the idealized fixed-fixed and pinned-pinned connections are also included in the graph for comparison. The process can be discussed in two parts which are heating and the cooling phases.

According to the results for the heating phase, a peak value of axial force is observed at different beam temperatures for the connection types. According to the axial force graph given in Figure 3.13, while this peak value of 254 kN is seen at 194^{o}C for the pinned connection, the peak axial force for the fixed connection is 703 kN at 326^{o}C temperature. For the fin plate connection the peak axial force is 528 kN at 316^{o}C temperature. However, after the axial force reaches to its peak value in the heating phase, it starts to decrease due to the reduction in stiffness and strength at higher temperatures. This scenario is valid for all types of connections. Moreover, due to the increasing mid-span deflection with increasing temperature, the axial force and moment values demonstrate a gradual decrease after the peak value of each.

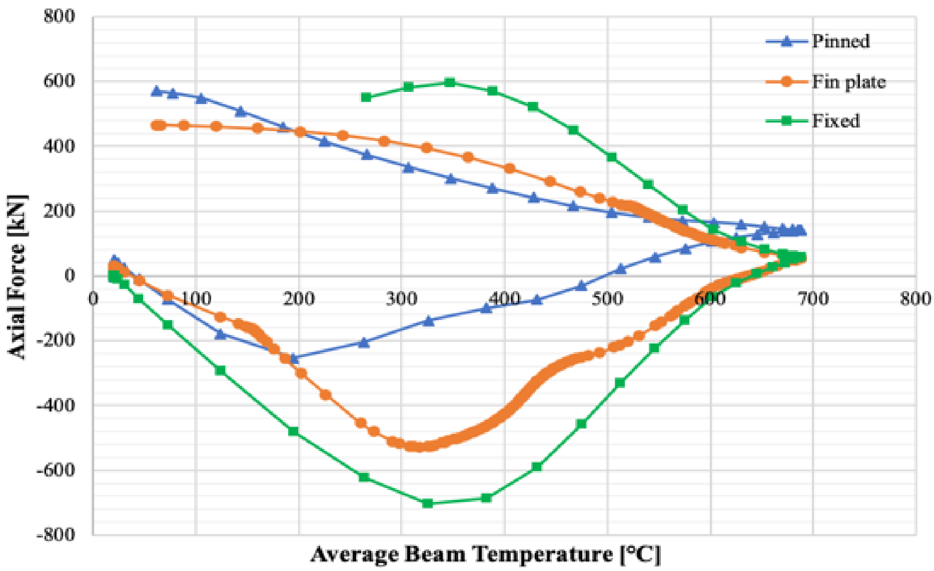

Figure 3.13. Axial Force vs. Average Beam Temperature.

In addition, the beam with idealized pinned connection shows a force trends with lower values when compared to other two connection types. Being rotation in z-direction and the translation in x-direction is allowed for the pinned connection type can be the reason of lower axial force values. Furthermore, for the beam with semi-rigid connection, the effect of the contact established between beam flange and the supporting member on the axial force is clearly seen in Figure 3.14. Once the contact is established in semi-rigid connection, the beam end rotation stops and the rate of increase of axial force (compression) picks up. As predicted, the semi-rigid connection behaves in between the fixed and pinned connections.

According to the results for the cooling phase from Figure 3.14, the force trends can be evaluated as similar for all three connection types. It is mainly observed for this section that, the beam with fin plate connection reaches the lowest axial force value of 467 kN (in tension) at the end of the cooling phase. The beams with idealized pinned and fixed connections give higher results of axial force when compared to fin plate connection. It is worthwhile mentioning that the beam with idealized fixed type connection fails before the cooling period ends. The failure is in the beam; the plastic moment capacity is reached near the connection region.

3.8.4. Connection Rotation-Temperature Results

As predicted in the corresponding study, during gravity loading both fin plate and idealized pinned connection rotates 1.5^o (clockwise), wheras the fixed connection does not have rotation during the entire simulation as seen in Figure 3.14. During the heating phase, due to reduction in rigidity, the beam end rotation picks up for all type connections. At 150^oC, the fin plate connection does not rotate (the curve stays flat) until 250^oC. This is due to the established contact of the beam flange to the supporting member. As the rigidity further decreases, the fin plate connection starts to rotate again. During the cooling phases, both fin plate and idealized pinned connection reverses rotation. However, due to excessive mid-span deflection the beam end rotation always stays clockwise.

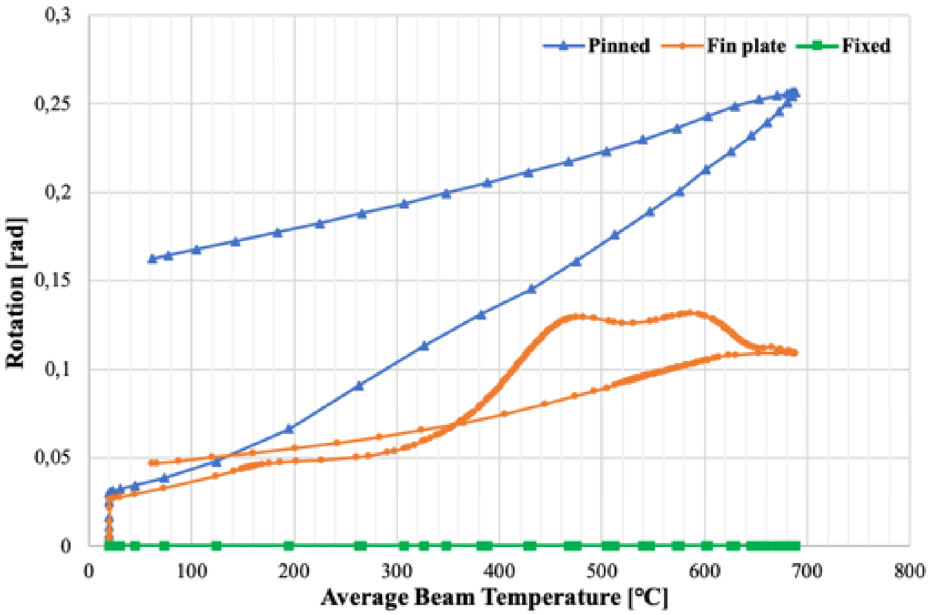

Figure 3.14. Connection Rotation vs. Average Beam Temperature.

3.8.5. Deflection-Temperature Results

Mid-span deflection results under realistic fire condition is demonstrated in Figure 3.15. It is found that, the mid-span deflection with different connection types shows different results at the end of the heating and the cooling phases. As expected in the present study, the gravity loading causes smaller mid-span deflection in the beam with fixed-fixed connection type. The fin plate connection behaves similar to pinned connection until the contact is established (see Figure 3.15). The rate of deflection decreases until 300^{o}C. As rigidty decreases further, the deflection picks up and the fin plate connection reaches to the same deflection of the pinned connection at around 450^{o}C as shown in the Figure 3.15. However, in the cooling phase, the fin plate connection behaves similar to the pinned-pinned connection. It is also clear that the fixed connection fails before the cooling period ends. The failure in the beam is caused due to the plastic moment capacity limit near the connection.

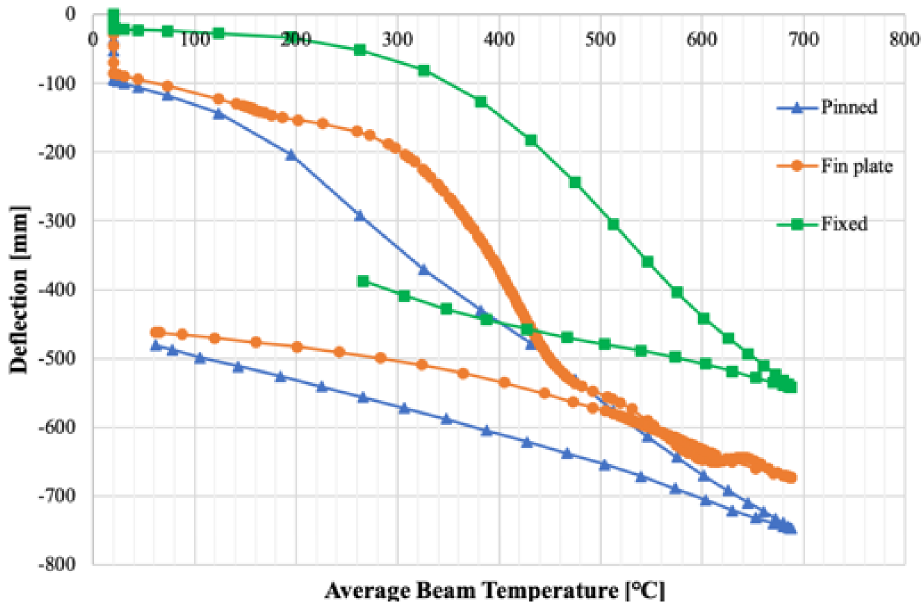

Figure 3.15. Midspan Deflection vs. Average Beam Temperature.

3.8.6. Moment-Rotation Results

Analyzing a semi-rigid connection requires a clear understanding of the moment-rotation relationship. A beam with a fin plate connection clearly shows the contact point where it starts to transfer moment as demonstrated in Figure 3.16. During the heating phase, the moment (clockwise) increases as the rotation increases. During the cooling phase, fin plate connection reduces its moment down to zero, with some residual rotation due to excessive midspan deflection. Additionally, the effect of the contact after gap is closed can be seen in the Figure 3.16. The arrows show the path of the process. The maximum rotation is calculated to be 0,13 radians (7.5^o degrees) at the end of the heating phase. After that point, the cooling phase starts and the moment drops to almost zero.

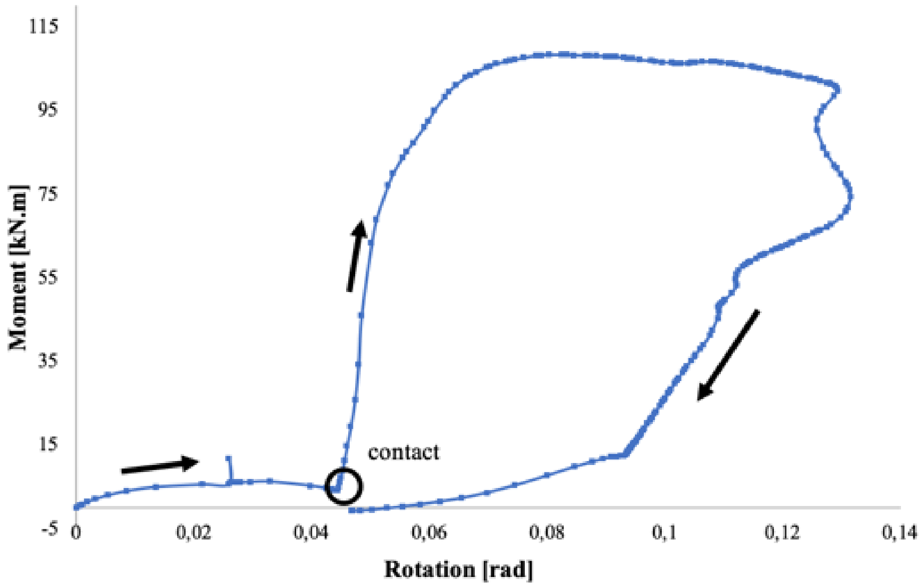

Figure 3.16. Moment with Rotation of Beam with Fin Plate Connection.

4. 3D FINITE ELEMENT MODELING

The main objective of this chapter is to evaluate the fire performance of a beam with a semi rigid connection under realistic fire condition with 3D Finite Elements Model (FEM), than to compare the results of 3D FEM approach with the CBM. For that reason, a highly detailed 3D FEM is created by using the ABAQUS software under the same loading and temperature situation by giving same material properties. This is a complex model accounting for material nonlinearity, large deformation and contact behaviour. Contact is very important for modeling the behaviour of the joint. In this study, the contact elements are used at the bolt-hole interface and also at the surface between the web of the beam and the fin plate. The connection model is analysed through the elastic and plastic ranges up to failure. Plate bearing, beam web bearing and bolt shearing are observed as failure modes. Moreover when the connection model is extended to include an attached beam (assumed as a steel plate), it is found that it finally experiences large tensile force when exposed to fire. At the end of this chapter, it is found that the CBM has a high level of accuracy for the beams with fin plate connections at ambient and elevated temperatures.

An illustrative three-dimensional FE model of a beam with a fin plate connection is developed in order to perform and understand the behavior of such a connection at ambient and elevated temperatures as seen in Figure 4.1. The starting point for this model is a simple plate with a bolt against a hole. This model is then improved to form a single plate, bolt and IPE330 beam as applied in the CBM.

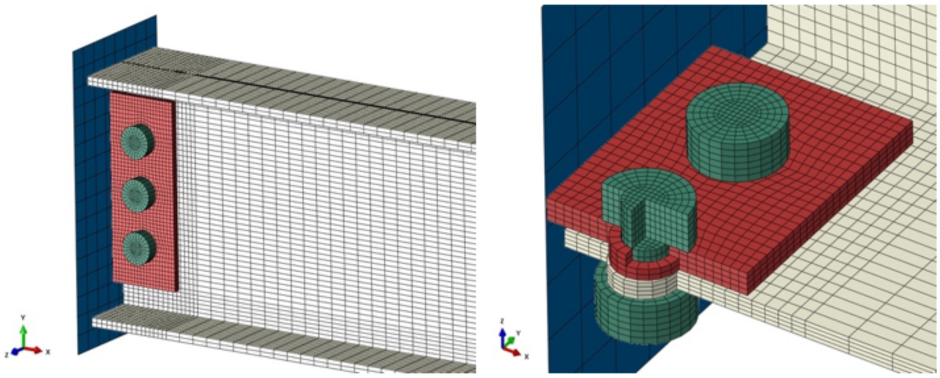

Figure 4.1. Developed FE Model.

Three essential parts of the steel fin plate connection, which are IPE330 beam, fin plate and bolts, are modeled using the C3D8 eight-node linear brick elements. In addition to that, to symbolize the beam-to-beam fin plate connection, an addition rigid plate is attached to fin plate as seen in Figure 4.1. Each element in the system consists of total 8 nodes, including one at each corner. The mechanical and the thermal material properties of S275 structural steel material are selected as the same with the properties used in the CBM section. In the previous section, the elastic and plastic properties of materials under elevated temperatures and required calculations were discussed in detail.

Modeling a 3D Finite Elements Model is required to simulation a structural elements accurately. However, in a beam to beam fin plate connection, the bolts have an important role in maintaining the integrity of a structure. It is known that, they provide links between the principal structural members. Therefore, to create a realistic bolt model is required to give pretension to the each bolts as seen in Figure 4.2. In order to create a real-like model, two types bolts were observed in system as the preloaded bolts and no preloaded bolts. Owing the fact that the model with no preloaded bolts showed a stabilization problem, the component based results are compared with the model with preloaded bolts in the system.

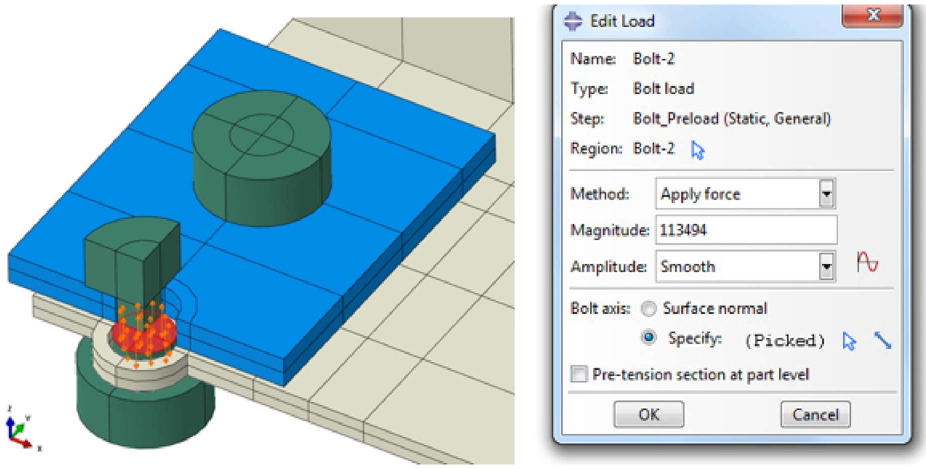

Figure 4.2. Preloaded Bolts used in Corresponding Model.

4.1. Defining of Parts

In this study, three main parts are defined in the part manager as the bolt, IPE330 beam and the plate. Bolts are available in standard size and a range of lengths. Bolts can act in two ways, namely in tension and shear. The individual parts of a bolt comprise a hexagonal head and a circular section shank, which may be fully or partly threaded. They come in shank diameters from 6 mm to 36 mm. Larger diameters may be available for special order. Two grades of bolts are commonly used as namely 4.6 bolts and 8.8 bolts. The first digit relates to the ultimate strength of the material, while the second is the ratio of yield stress to ultimate strength. Thus, grade 8.8 bolts have an ultimate strength of 800 N/mm2 and a ratio of yield stress to ultimate strength of 80%. Depend on this information about the bolts, the FE model in this study is created for a single M20 grade 8.8 high strength bolts and the bolt holes are modeled 1 mm larger than the bolt shank radius. Three row bolts are used in this study as the same with corresponding component model and the surfaces are divided into pieces to obtain more accurate results and easy mesh patterns as shown in Figure 4.3. In this way, cubic mesh patterns could be used for whole parts.

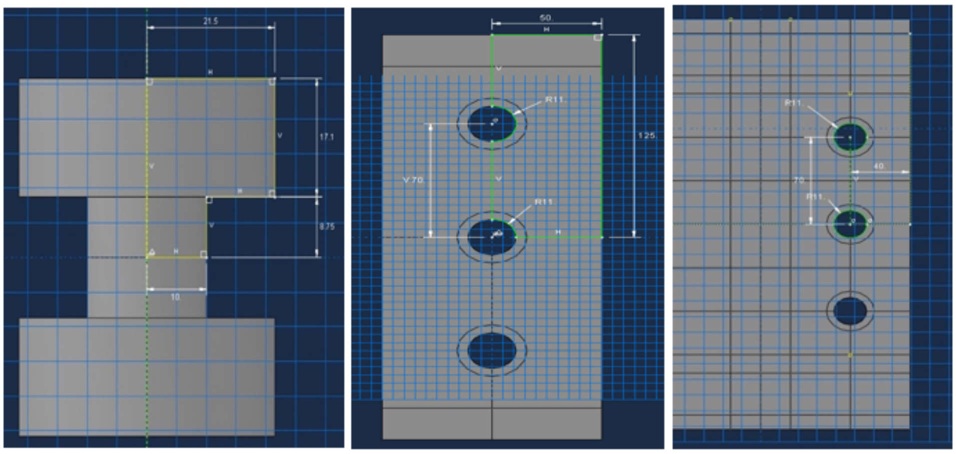

Figure 4.3. Bolt Sizes, Plate Thickness and IPE Profile Size of the 3D FE Model.

4.2. Contact Elements

Smooth amplitude is very important for the contact interactions. For accuracy and efficiency quasi-static analyses require the application of loading that is as smooth as possible. Applying the load in the smoothest possible manner requires that the acceleration changes only a small amount from one increment to the next. If the acceleration is smooth, it follows that the changes in velocity and displacement are also smooth. In this study, smooth amplitude is used in all the steps in order to observe the contact interactions correctly and to obtain more accurate results. Otherwise, the solution process would be longer or no solution would be taken.

Abaqus has a simple, built-in smooth step amplitude curve that automatically creates a smooth loading amplitude. When you define a smooth step amplitude curve, Abaqus automatically connects each of your data pairs with curves whose first and second derivatives are smooth and whose values are zero at each of your data points. Since both of these derivatives are smooth, you can apply a displacement loading with a smooth step amplitude curve using only the initial and final data points, and the intervening motion will be smooth. An example of a smooth step amplitude curve is shown in Figure 4.4.

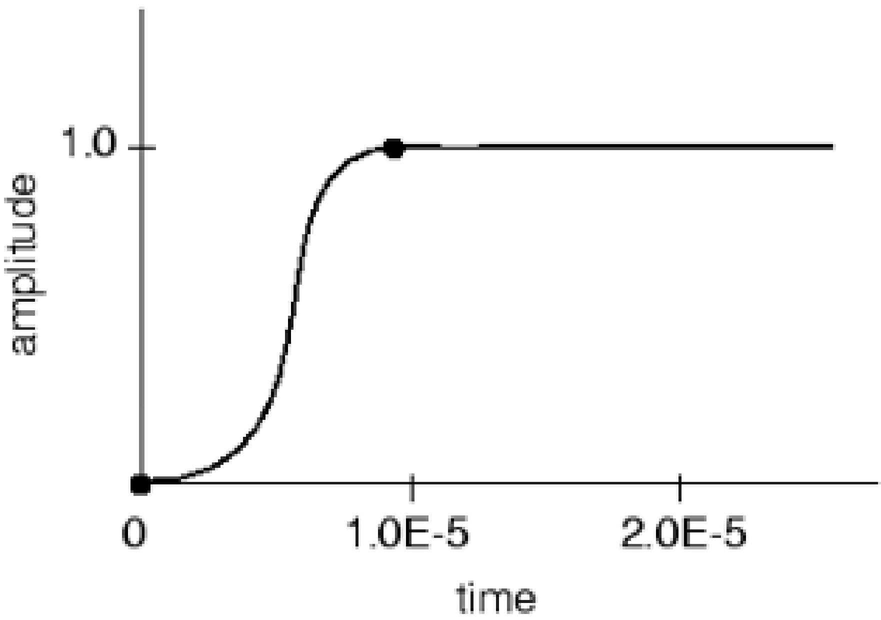

Figure 4.4. Amplitude Definition using a Smooth Step Amplitude Curve.

There are two types surface nodes in the contact manager as the master node and the slave node. In the contact manager, contacts are given as automatically and created by using the "find contact pairs" command. The adjustment value is implemented to the bolt pressure surfaces in order to begin the contact with the 0.01 mm contact tolerance. Thus, the beginning of the contact would be started with the first step of the analyses. This behavior supplies to define as numerically that "when the contact begin, the bolts are in contact" and the program doesn't need to an extra computing process. Because each master element tries to find the its slave node and it takes time. Master surfaces are specified in red colour and slave surfaces are specified in pink colour in the model. In order to represent the interactions between the different connection components, 13 interactions and ties are created. Surface-to-surface contact element items are principally used to symbolize the interactions between the connection components. Moreover, two types of contact properties were defined in the model as tangential and normal contact. In the case of tangential contact, the friction coefficient is taken as 0.25. Also, in the case of normal contact, hard-contact feature is selected and isn't allowed to separate between contact surfaces.

4.3. Boundary Conditions

Five types boundary conditons are created in the 3D FE model to represent the beam with fin plate connection condition. In the first situation, the system is only free in the x-direction in the part of the fin plate connected to kinematic coupling. The axial restraint connection element is defined in between the kinematic cupling (see Figure 4.5) and free end to represent the semi-rigid connection behavior properly and in this part the displacement/rotation is restricted to 6 degrees of freedom and the movement of the system is not allowed. In the mid span kinematik coupling condition, the system is restricted in all degrees of freedom in order to represent the symmetric boundary condition. It is assumed that the system will not have a major deformation in the z-direction, it is restricted to movement in this direction by defining a point along x direction from the top and bottom of the profile. It is also assumed that the bolts will not have movement in the x- direction and y-direction only for preload step. After finishing the preload step, this end condition will be inactive.

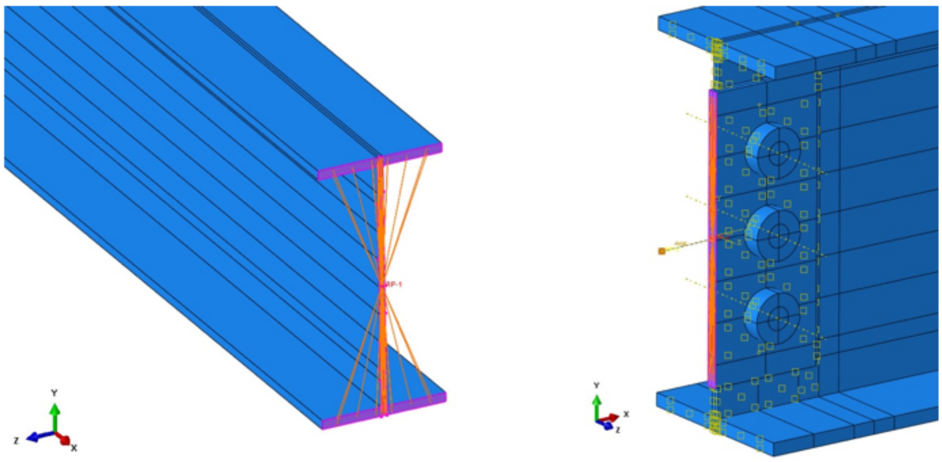

Figure 4.5. Kinematic Couplings in Midspan and Fin Plate Connections.

After the modeling and loading steps, job manager command is selected in order to run the analyses. Component- based model totally 20 minutes takes as a time and 3D Finite Element Model also takes 110 minutes as a time by using a 8 cpu computer. After finishing the running, the model will be submitted and completed.

4.4. Results and Discussion

The fire performance of the beam with a semi rigid (fin-plate) connection is modelled with 3D Finite Elements. The axial force and the moment results depending on the realistic fire condition are obtained and compared with the data from the Component Based Method. The undeformed and deformed shapes of the proposed beam subjected to gravity loading and the realistic fire condition is given in Figure 4.6 and Figure 4.7. The deformed shape given in the figure demonstrates the early stages of the process where the temperature is about 150°C. As can be seen from the figure, when the beam deformed at that point, the gap is closed and the contact between beam flange and the supporting member is started. In short, a fin-plate connection behaves as a idealized pinned connection during the heating phase until gap closes.

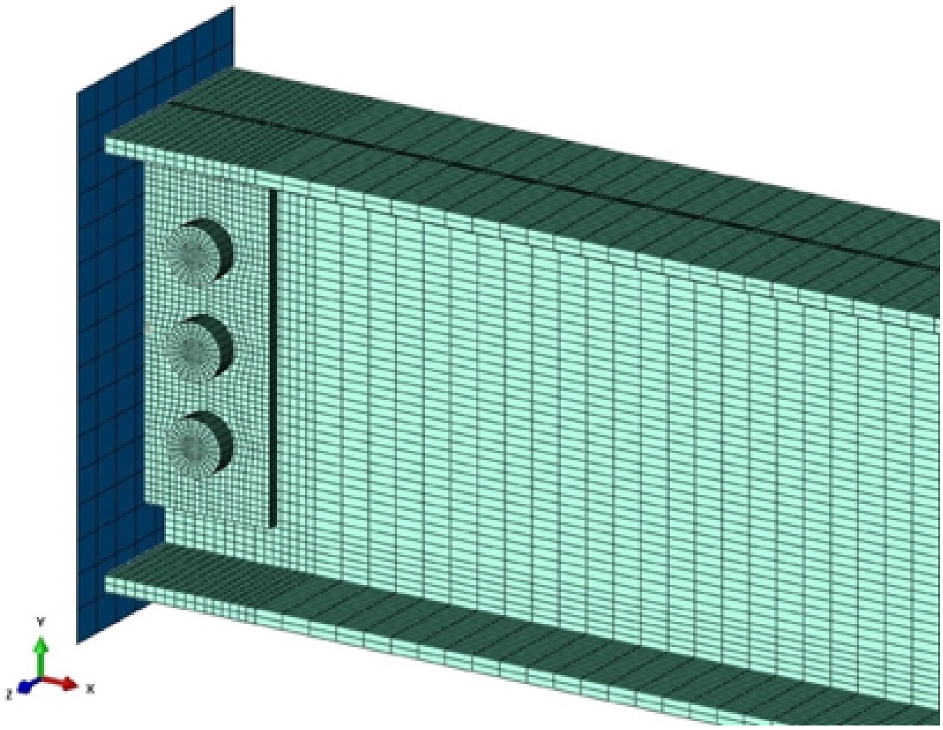

Figure 4.6. 3D FE Fin Plate Connection Model.

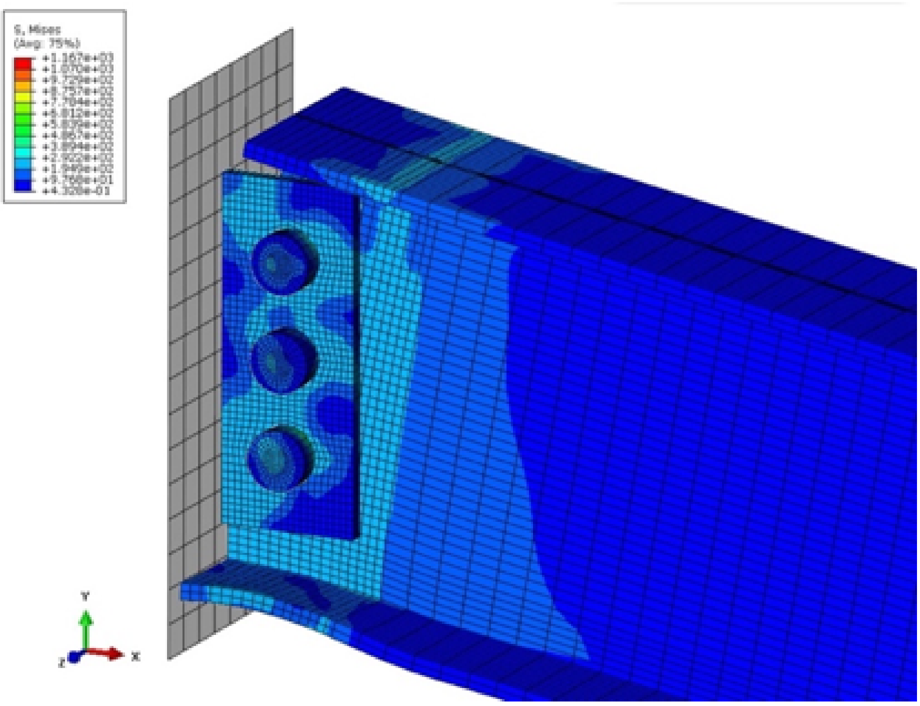

Figure 4.7. The Deformed Shape of 3D FE Fin Plate Connection Model (at contact).

The behavior of the beam before and after the gravity loading and elevated temperatures conditions can be seen as visual in Figure 4.8. According to this figure, (a) demonstrates the beam to beam connection model at first situation without any loading and fire conditions and (b) shows the connection model under the only gravity loading at ambient temperatures. In the stage of heating (c), the material thermal and mechanical properties change and major internal forces and moment occur in the connection. Beam with fin plate connection model in the cooling step can be seen in Figure 4.8. (c).

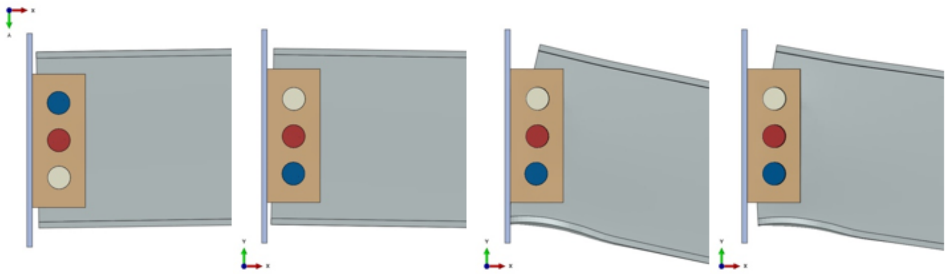

Figure 4.8. The Stages of Fin Plate Connection Model under the Loading and Fire Condition.

The axial force results are compared as given in Figure 4.9. An acceptable agreement can be seen between the CBM and 3D Finite Element Models in terms of the axial force. The models generally demonstrate similar trends, especially for the heating region where the sudden increase occurs due to the contact later in the 3D Finite Element Model. As discussed in the previous section, in the heating phase, the semi-rigid connection acts like a pinned connection until the contact is established and then, it starts to act as a moment (fixed) connection. According to Figure 4.9, there is a good correlation for the maximum axial force values between two approaches. As a comparison of presented models, the temperature at the point where the contact is established shows some deviation. This situation can be explained by the presence of three dimensional bolts in the real system. By taking the run time reduction provided by CBM into consideration, this difference can be discussed as reasonable. For the cooling phase as shown in Figure 4.9, the graphs have similar force trends with considerable deviation. A reduced axial force (in tension) during the cooling phase can be attributed to the clear distance between the bolt holes and the bolts in the 3D Finite Element Model.

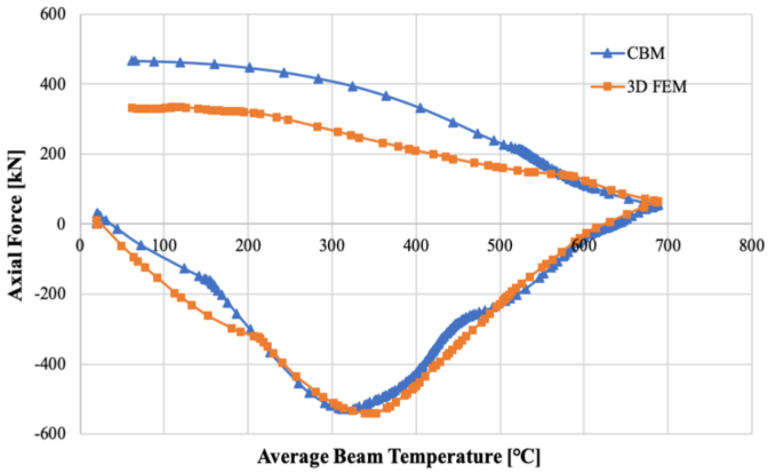

Figure 4.9. The Comparison of CBM and 3D Models as Axial Force and Temperature.

The midspan deflection results are compared in Figure 4.10. According to this figure, Component-Based and 3D models show a good agreement and almost have same a trend with regards to midspan deflections. At temperatures 20°C, there is an increasing deflection value due to gravity loading. After that, with increasing temperatures, the deflection starts to increase nonlinearly due to changing material properties. It is an important point that, in the cooling stages deflection will decrease and at the end of this stage, some plastic deformations will be perminant due to decreasing material strength and stiffnesses.

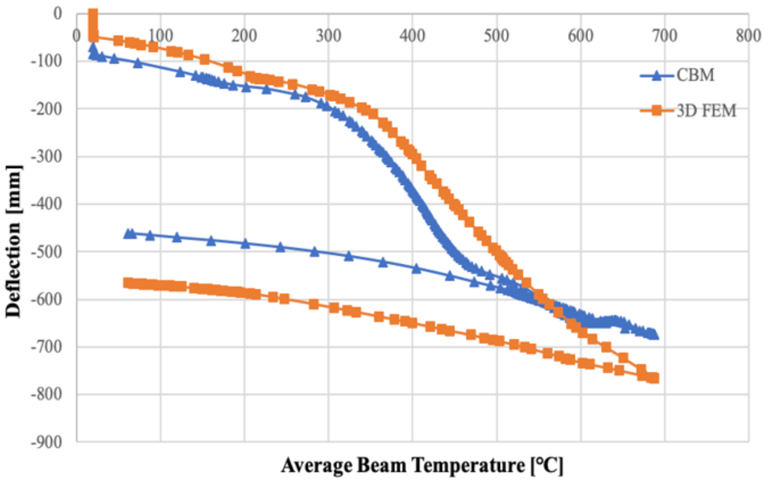

Figure 4.10. The Comparison of CBM and 3D Models as Midspan Deflection and Temperature.

The moment results at connection are compared as given in Figure 4.11. An acceptable agreement also can be seen between the CBM and 3D Finite Element Models. The model trends are similar, especially for the heating region where the sudden increase occurs due to the contact later in the 3D finite element model. As discussed in the previous section, in the heating phase, the semi-rigid connection acts like a pinned connection until the contact is established and then, it starts to act as a moment (fixed) connection. In the 3D finite element model, a small negative (counter clockwise) moment is observed, whereas in CBM, the connection moment always stays in the clockwise direction.

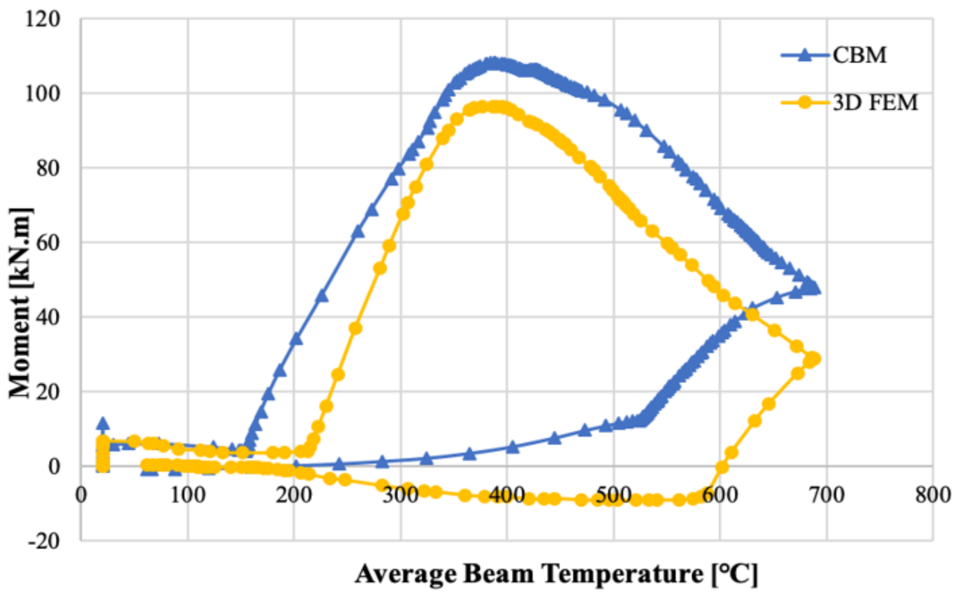

Figure 4.11. The Comparison of CBM and 3D Models as Moment at Connection and Temperature.

5. CONCLUSION

In this thesis study, the fire performance of a beam with a fin-plate connection was modelled with the Component Based Method by using ABAQUS Finite Elements software. The behavior of fin plate connections were compared with the idealized pinned and fixed connections. Moreover, a benchmark study is conducted using 3D finite element model with the same material properties, boundary and loading conditions. Since the results show a reasonable agreement suggest that CBM is an advantageous approach and can be used to understand the behavior of the beam with fin plate connections when subjected to gravity loading and also under realistic fire scenarios. For fire conditions, the behavior of semi rigid connections significantly differ from idealized pinned or fixed boundaries. It has been shown that, the component based method enables to model a semi rigid connection which during heating acts as a idealized pinned connection, then as a fixed connection, and lastly as a pinned connection again during cooling. In short, the fin plate connection behaves in between fixed and pinned connection. In addition, CBM proves that it can be used in large structural systems with ease. Where a 3D finite element model for an entire building is computationally very expensive, utilizing CBM at beam ends is a faster and accurate way. This study is expected to create awareness for the structural fire safety in beams with fin-plate shear connections. Because it is important to enhance the use of mathematical expressions for three-dimensional structural connection modeling for thermo-mechanical problems in structural engineering practice. One of the most important results is to understand how the beams in building systems with proper restrained boundary conditions and with fin-plate shear connections perform during fire. The other significant gaining of this study is the run time reduction with CBM while obtaining accurate results. By adopting the component based method instead of the 3D finite elements, a 90% computer run time reduction is achieved (10 minutes vs. 3 hours).

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