UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL APPLIED SCIENCES

ANALYSIS OF STRUCTURES UNDER FIRE LOADING

M.SC THESIS
IN
CIVIL ENGINEERING

BY
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JANUARY 2013

Analysis of Structures under Fire Loading

M.Sc. Thesis
In
Civil engineering
University of Gaziantep

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January 2013

UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES
CIVIL ENGINEERING DEPARTMENT

Name of the thesis: Analysis of structures under fire loading

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Exam date: 28/ 01 / 2013

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Talor Qader MULA AHMED

ABSTRACT

ANALYSIS OF STRUCTURES UNDER FIRE LOADING

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January 2013, 75 pages

This study aims to understand the way the members of a structure act while part of the structure is subjected to fire. How many members are high deflected, their situation, and their role on the collapse of the whole building is also in the scope of this study. A finite element analysis program (STAAD PRO v8i) is used to analyze the 3D structure. The temperature is applied to one compartment, and the type of temperatures that to be applied to the members of that compartment are uniform temperature and gradient temperature. Whether the member is exposed to the same rate of the temperature along its length or the rate of the temperature increases gradually along the length of the member, is also considered. The deterioration of the mechanical properties as the temperature increases has a great importance. From the results it is noted that the highest deflection happens in the fired compartment, and only the members of the neighboring compartments of the fired compartment have a considerable deflection to be mentioned, especially in the floors higher than the fired compartment.

Key words: Multi structure; temperature analysis; thermal loading

ÖZET

YAPILARIN YANGIN YÜKÜ ALTINDA ANALİZİ

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Yüksek Lisans Tezi, İnşaat Müh. Bölümü

Tez Yöneticisi: Prof. Dr. Mustafa ÖZAKÇA

Tez Yardımcı Yöneticisi:Yrd. Doç. Dr. Nildem TAYŞİ

Ocak 2013, 75 sayfa

Bu çalışmanın amacı, yapının herhangi bir bölgesinde yangın çıkması durumunda, yapının diğer elemanlarındaki davranışı gözlemlemektir. Ayrıca hangi elemanlar fazla deformasyona uğramakta ve yapının çökmesi üzerinde bu elemanların etkisi ne kadardır, sorularının cevapları bu çalışma kapsamında araştırmaktır. Sonlu elemanlar analiz program olan (STAAD PRO V8i) üç boyutlu yapısal analizi için kullanılmıştır. Yangından dolayı sıcaklık değişimleri yapının ilgili bölümüne uygulanmaktadır. Bu bölümdeki elemanlara ısıl yükler, düzgün yayılı ve değişken olarak tanımlanmıştır. Sıcaklık eleman boyunca giderek artmakta veya sıcaklığın düzgün artığı Kabul edilmiştir. Sıcaklık arttıkça malzemenin mekanik özellikleri önemli oranda bozulmaktadır. Sonuçlar incelendiğinde, en yüksek deformasyonun yanan bölüm içinde olduğu ve özellikle yangın olan kısmın bir üst katı olmak üzere komşu bölümlerde dikkate değer deformasyonların olduğu gözlemlenmektedir.

Anahtar Kelimeler: Çok katli yapılar, sıcaklık analizi, ısıl yüklemeler

To my family, especially my children Zhyar and Zhelia.

ACKNOWLEDGEMENTS

I want to express my deepest gratitude to my supervisor Prof. Dr. Mustafa ÖZAKÇA and Asst. Prof. Dr. Nildem TAYŞI for their guidance, advice, criticism, encouragements and insight throughout the research.

I would like also to thank my friend and colleague Serdar Ali for his help and valuable comments about the program STAAD PRO.

I have special thanks to my parents for their continuous support and encouragements, that without their important advices and notes I wouldn’t find my right pathway.

Thanks to my wife for her understanding and support of my work, and her patience throughout the writing of this thesis.

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Abbreviations:

AISC	The American Institute for Steel Construction
ASTM	The American Society for Testing and Materials
CFD	Computational Fluid Dynamics
2D	Two Dimensional
3D	Three Dimensional
DL	Dead Load
f_1	The average change in temperature (uniform) in STAAD PRO program
f_2	The temperature differential from the top to the bottom of the member (gradient) in STAAD PRO program
f_4	The temperature differential from side to side of the member (gradient) in STAAD PRO program
FE	Finite Element
ISO	International Standards Organization
LL	Live Load

Latin Symbols:

A	Area
°C	Celsius temperature
D	Depth of the section
E	Young’s modulus
F	Force
T_y	Gradient temperature
T	Temperature

Greek Symbols:

σ	stress
ϵ	strain
α	Thermal expansion coefficient
ΔT	Change in temperature
δ	displacement
ϵ_T	Thermal expansion strain
ϵ_t	Total strain
ϵ_m	Mechanical strain
Ø	Curvature

CHAPTER 1

INTRODUCTION

1.1 General

Since ancient times until the recent days the fire remains a challenge for the civilization, the authorities still trying to control the fire and it is effects both on the human beings and on the constructions continuously. The building structures are not exceptions, also with the new modern precautions of fire protections and anti-fire systems. The fire still remains a great danger on the stability and the safety of the structures. So many building structures in the world are facing the fact that they are the next victims of the burning fires.

The buildings may be built from various materials, some of them may be built from concrete, others may be built from steel and others may be built from timber. Also some of these buildings may be high rise buildings or they are moderate height buildings.

The building structures in their interior geometry distributions are different. For example some of them have large compartments and some of them have small compartments. The building may has large open spaces like atriums inside it. Also the number of stories of the building may varies according to the area needed for occupation; and the length of the span of the beams may change according to the length between the columns.

All these factors mentioned have influence on the way the fire act and moving. Also they have influence on the time the fire takes during burning. Because there are many factors interact during happening of the fire that makes anticipating the results of these fires to be hard.

Many studies have been done for modeling and analyzing the fire behavior on the structures. These studies didn’t use the same methods and approaches for gaining their results. They have tried to use numerical and computational methods for modeling and simulating conditions look like the same conditions that happening in the fire compartment. Some of them have used hand calculations method; some of them have used zone modeling, and some of the have used field model method (computational fluid dynamics (CFD)).

In the recent days, the structures affected by fire temperatures are analyzed by computer programs depending on Finite Element (FE) Method software, the only exception is in the simple members in which the hand calculations can be used to solve them. Because the structures are composed of a lot of indeterminate connected members that makes the analysis of this complicated net of the members is very hard by the hand calculations. For pass over these obstacles the researchers used the finite element based programs for analyzing the members affected by mechanical and temperature loads.

Many studies have used the FE computer programs, because the results are very close to the reality, a lot of these studies have modeled 2D structures and some of them have modeled 3D structures. Because the environment of the 3D structures is more complicated than the environment of 2D structures, it needs more attention and accuracy to deal and work with the 3D model structures. But in other hand 3D structures are more close to the reality, in which the observer can better understand the layouts of the buildings.

The studies that concerned with the fire occurring in buildings are of great importance to our daily lives. All of these studies has just taken care of part of the subject, a lot of these works studied were about the fire environment inside the compartments, how the fires spread, the time took the fire to spread, types of fires, their intensity, the ventilation, how many ways can be used for analyzing and simulating the fires.

Some other studies dealt with the fire exposures on a specific member in two conditions, first whether this member is an alone member which is not continuous and not connected to any other members in which it can be examined in a lab. Second the member is part of a larger structure that all the members of this structure are connected and interact with this member. So that member act differently when it is alone member from where it is inside the body of the structure.

Instead of focusing on the fire’s properties itself, the temperature’s effect on the members of a steel structure are to be focused. Also the change induced in the members’ behaviors according to their position in the whole structure and according to the value of the temperature they are exposed to be focused on. Whether the members have been affected directly by the fire inside the compartment or they have indirectly affected apart from the fire compartment.

Knowing and predicting the members’ behaviors in the burnt buildings is very important to know in which member the fire makes the maximum damage and where the largest deflections could happen and whether the building or part of it could reach the collapse or not.

If the members of one compartment failed and collapsed, is that mean it will cause the whole building fail, or the building can sustain the failed members by the interlocking action of the members and by the members’ continuity and by the indeterminacy of the structure. Can the structure compensate the loss of the failed members by redistributing moments according to the new stiffness of the remained members? And if the damage of the members in the compartment of fire was large, is that mean the damage in the other members is large too? These questions are the aims of this thesis.

1.2 Structures

‘An engineering structure is any connected system of members built to support or transfer forces and to safely withstand the loads applied to it’.

See Figure 1.1, all the structures when designed the engineer must account for their safety and serviceability, without forgetting the economic and the environmental parameters [1].

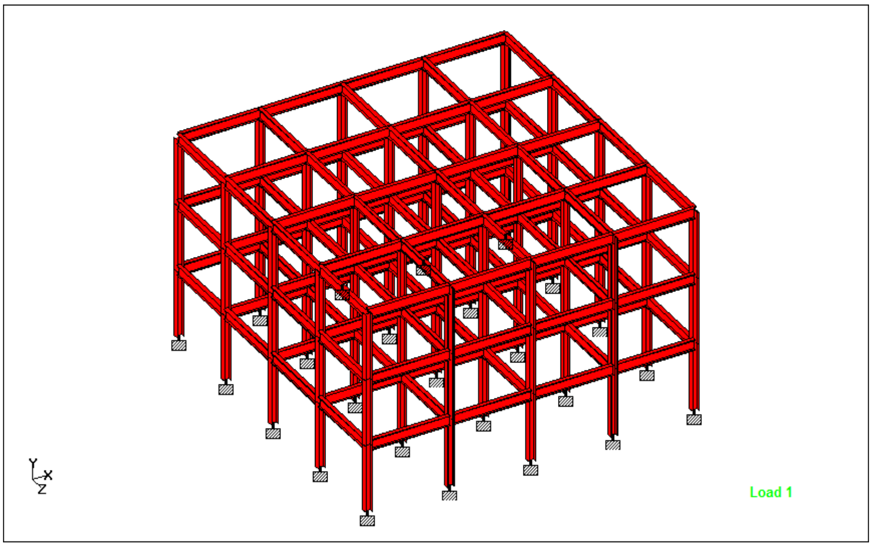

Figure 1.1 A 3D multistory building structure

1.2.1 Materials used in the structures

When the material used for a beam is metal such as steel or aluminum, the cross section is the most workable when it is shaped as a wide flange. Because of it is strength and flexibility steel is used in the high rise buildings such as skyscrapers [2].

Concrete beams in general have rectangular cross sections, because it is easier to form in this way. The concrete is weak in tension so steel bars need to be added to the beam. In columns made of concrete, circular and square cross sections with reinforcing bars can be used.

Beams can be made from timber; laminated beams are constructed from sections of wood that can be shaped in various ways, usually are fastened together using high-strength glues or bolts.

1.2.2 Loads on the structures

Loads are forces applied to a component of a structure or to the structure as a unit. For example, high-rise structures must resist the lateral loadings caused by winds that shear walls are used to keep the structure stable, another example is the buildings that subjected to earthquakes must be designed having special ductile frames and joints.

Some loads categories are as below:

Dead Loads: (DL) are the loads that relatively constant by the time, these loads consist of the weights of the structural members and the weights the objects that attached to the structure.

Live Loads: (LL) are the loads that change both in their magnitude and their location. They may induce from the weights of objects temporarily placed on a surface or moving vehicles. The minimum live loads in codes are determined from the experience gained during dealing with these loads in past.

Wind Load: (WL) is the force on a structure induced from the impact of wind on it.The density and velocity of the air are the main characters controlling the wind’s effect on a structure; also the angle that the wind hit the structure is important.

Snow load: (SL) is the live load due to the weight of snow on a roof it depends on the building’s shape and geometry, wind exposure, and location.

Earthquake Load or Seismic Load: is the force on a structure caused by acceleration induced on its mass by an earthquake, it produce loadings on a structure through the interaction between the ground and the responses of the structure.

Temperature Loads: when structures are exposed to fires, the temperature that induced from the fire causes initial stresses and strains inside the members of the structure.

1.3 Definition of the Temperature Loading

Temperature is the measurement of average kinetic energy. Two systems are commonly used to measure the average kinetic energy; Fahrenheit and Celsius. A material’s temperature is simply the measurement of the amount of motion that the molecules or atoms have. If the molecules within a material are moving very quickly, the temperature of this material will be high; slow motion equals a low temperature [3].

Heat is the transfer of energy based on a temperature difference between two objects or regions of a single object. The flow of energy is always in the direction from a high temperature to a lower temperature. Therefore, heat will not be transferred between two systems of the same temperature.

1.4 Layout of the Thesis

In this work, it is focused on the members that affected by temperature not on the fire circumstances itself. The members behaviors inside the whole structure before the fire and after the fire, and the location of these members and how they affected by fire was a concern of this study.

In this work all the members are affected by temperatures from 20°C to 520 °C because if the temperature exceeds 550°C the members lose a lot of their strength and they reach the plastic phase that in our work all the calculations are done depending on the elastic linear analysis for solving.

The object of this thesis is to compare the deflections induced in the members to know how the members act before fire and during fire, and if the whole structure members are affected by the fire temperature or only the near ones. The layout and classification of the study is as below:

Chapter 2 is the literature review of the previous studies done on the structures that affected by fire temperature.

Chapter 3 is about the temperature loads and their types and their ways to effect on the members, it is also give a brief explanation about the STAAD pro software program that is used in this study, and how many temperature loads the program can apply to the members.

Chapter 4 is about the examples modeling and their results with comparison among them and their discussions.

Chapter 5 is the conclusions from the results of the thesis also it is about the future works that preferred to be done.

CHAPTER 2

LITERATURE REVIEW

2.1 Frames Affected by Fire Analysis

Many researchers have tried to analyze the temperature effects of 2D and 3D frames until now by applying different techniques with different methods.

The Australia and Stuttgart-Vaihingen University Germany conducted four storey large scale tests in the 1985 [4].

In 23rd June 1990, an accidental fire occurred in a partially completed 14-story office block on the Broadgate development in London. The fire started at the first floor level. The temperatures estimated to be more than 1000 °C. Because the structure was still in the construction phase, the steel frame was only partially protected and the sprinkler system and the fire detection system were not yet in service. However, despite it is subjected to very high temperatures for about 4.5 hours and experienced deflections in the composite slabs, the structure did not collapse [4].

The Broadgate accident initiated construction of an 8-story composite steel frame at the Building Research Establishment's (BRE's) test facility in Cardington near Bedford, United Kingdom in 1993. The building simulated a real commercial office building in UK. It is design was based on the British Standards and checked with the Eurocode.

The experimental studies were seven large-scale fire tests in .which the fires were started at different locations. The beams had no fire protection while the columns were fully protected. Whether the building was subjected to a number of full-scale fire tests, the building still continued to carry loads without failure [5].

Here are names of other buildings that there are papers written on large fires in tall buildings, such as Churchill plaza building Basingstoke. One Meridian Square, the World Trade Center Towers and Windsor Tower have renewed interest in studying the response of buildings under fire loading.

This work is taken from Lewis [6]; Spearpoint in (1998) compared the results of a finite element computer program, THELMA, with results from a full size fire test at the test facility in Cardington, UK. Proeet al1,(1986) explains the mechanical response of steel. Mechanical properties of steel in a member such as expansion, modulus of elasticity and the strength of the struts that are looked at by Franssen et al, (1994), compared different computer programs, analyzed three structures under different temperatures. The examined computer programs are CEFICOSS, DIANA, LENAS, SAFIR and SISMEF.

Many people in the structural engineering field thought that the fire resistance of structures based on single element behavior in standard fire tests instead of the real behavior of these elements, when they acted as part of a whole structural system.

It is accepted that conditions in standard fire tests are not like the actual conditions happening in real fires. The concept of isolated structural elements is used in the case where fire in a compartment attacks only the individual structural members, in these cases of isolated members no account is taken for the interactions between the members in the structure. If the structure was large and redundant, these interactions can change the structural response entirely.

Lien et.al. [7] studied the nonlinear behavior of steel structures under fires also the cooling phases induced by temperature.

Moss and Clifton [8] used experimental testing and advanced finite element modeling so to develop a design procedure to take account of the inelastic reserve of strength available in structure systems.

Wang [9] presented the results of analysis the global structural behavior of the 8-storey steel framed building at Cardington during the two BRE large-scale fire tests. In his tests he tried to understand the behavior of the whole building structure under realistic fire conditions.

Dong and Prasad [10] described the experimental results of a furnace test conducted on three full-scale composite frames.

In the collapse conditions, determinate and redundant structures are easier to get fail. In determinate structures when the high stressed region reaches it is strength or when the member reaches the capacity that the structure can no longer sustain any further load then the collapse occurs.

In the redundant structure there are different load paths that the additional load can be handled. Where a structure is highly redundant there are many alternative load paths and large deformations can develop without necessarily a loss of strength, and failure must be defined in a different way,sufficient reserve capacity can make a lot of structures to survive in fires with little structural damage [5].

Borst and Peeters [11] 1989 they discussed about developing a consistent algorithm that simultaneously considers the effects of thermal dilatation, degradation of the elastic properties with increasing temperature, transient creep and smeared cracking.

Franssen et.al. [12] have discussed in their paper in 1995 about a simulation of a real fire subjected to steel frame. The application of heat flow models and the basis of structural model used were considered. The effect of lateral restraint, frame continuity and thermal expansion are found by computer model.

Liew et.al. [13] in 1998 in their paper described the large-displacement inelastic behavior of building frames exposed to fire and a methodology of an advanced analysis technique.

Nwosu and Kodur [14] they discussed the effect of continuity, restraint conditions, and load ratio on the fire resistance of frame structures. They compared between the performances of a beam with different end restraints in fire.

Song et.al. [15] in their paper tried to introduce a new method for the nonlinear analysis of steel frames subject to fire and explosion loading conditions. They used the nonlinear analysis program ADAPTIC for the analysis.

Elghazouli et.al.[16] discussed about numerical models that constructed to simulate the response of composite steel/concrete building floors under fire conditions.

Bailey [17] tried to control the direction of static loads during the fire to redirect it away from the unprotected beams toward the protected beams. The design method utilizes membrane action of the composite floor slab at large vertical displacements which are typically experienced during a fire.

Becker [18] in his paper focused on the effects of longitudinally non-uniform temperature distributions on structural fire response until failure of thermally protected steel structures, composed of beams and columns in simple frame cases. Structural analysis was done by the program SAFIR.

Clancy and Young [19] their goal from the research was to provide experimental results for the evaluation of theoretical models for predicting the behavior and time-to-failure of load bearing and non-load bearing wood framed walls in fire.

Liew and Ma [20] described the use of advanced analysis that accounts for both material and geometric nonlinearity to assess the performance of steel structures exposed to a natural compartment fire.

Bernhart [21] in his report examined the behavior of reinforced concrete beams with rectangular cross-section exposed to fire from the bottom and the sides. The study was performed with 2D finite element analysis using SAFIR.

Iu and Chan [22] presented an accurate geometric and material nonlinear formulation to predict structural behavior of unprotected steel members at elevated temperatures. The effects of uniform or non-uniform temperature distribution over the section of the structural steel member are also considered.

Lim et.al. [23] described numerical modeling of the fire behavior of two-way reinforced concrete slabs using a special purpose non-linear finite-element program, SAFIR.

Bénichou and Sultan [24] presented results of measurements of thermal properties at elevated temperatures of construction materials commonly used to build lightweight wood-framed assemblies. The effects of temperature on the thermal conductivity, specific heat, mass loss and thermal expansion/contraction of these materials are discussed.

Landesmann et.al. [25] concerned with the development of an advanced analysis numerical tool capable of estimating the inelastic large-displacement behavior of plane steel-framed structures under fire conditions.

Iu et.al. [26] described a numerical procedure based on the plastic hinge concept for study of the structural behavior of steel framed structures exposed to fire. The paper presented a fire analysis procedure for predicting thermal and cooling effects on an isolated element and a multi-storey frame.

Ezekoye et.al. [27] studied the effects of PPV on compartments downstream of the fire. Two venting strategies were used: venting the fire room and venting the victim room. Also he tried test the fire environment by putting fans and without fans.

König [28] this paper gave a review of the design rules of EN 1995-1-2, the future common code of practice for the fire design of timber structures in the Member States of the EU and EFTA, and makes reference to relevant research background.

Kodur and Sultan [29] this paper presented the effect of various factors on the fire resistance of load bearing, gypsum board protected, steel stud wall assemblies. Both single row and double row steel stud configurations with installation of gypsum board on each of the exposed and unexposed sides, and with and without insulation in the cavity, were considered in the experimental program.

Franssen et.al. [30] implied that the methodologies that are used for analyzing the fire behavior of a structure that is subjected to a uniform thermal situation cannot be applied when the fire is localized. The concept of ‘‘zoning’’ can be applied in which the structure is divided into several zones in which the situation is approximated as uniform.

Junior and Creus [31] have generalized plastic hinges concept, where normal force and bending moments are considered in the plastification process, is extended to include temperature effects. Additional refinements to allow for the gradual spread of yielding on the member were included, by means of stiffness reduction factors.

Hozjan et.al. [32] presented an alternative approach to the modeling of the mechanical behavior of steel frame material when exposed to the high temperatures expected in fires. Based on a series of stress–strain curves obtained experimentally for various temperature levels, an artificial neural network (ANN) is employed in the material modeling of steel.

Garlock and Quiel [33] in 2007 examined the behavior of wide-flanged (WF) steel sections with axial load and a thermal gradient through the section depth due to uneven exposure to fire. The paper interested on the shift of the section's effective centroid because of the conditions mentioned in which it will move away from the section's geometric centroid to a cooler side. The bending moments produced from the gradient temperatures are considered too.

Duthinh et.al. [34] presented two new interfaces in fire–thermal–structural analysis. The first interface uses adiabatic surface temperatures to provide an efficient way of transferring thermal results from a fire simulation to a thermal analysis. It assigns these temperatures to surface elements of structural members based on proximity and directionality. The second interface allows the transfer of temperature results from a thermal analysis modeled with solid elements to a structural analysis modeled with beams and shells.

Santiago et.al. [35] presented a numerical parametric study of a structural system consisting of an exposed steel beam restrained between a pair of fire protected steel columns. The numerical model accounts for the initial geometrical imperfections, nonlinear temperature gradient over the cross-section, geometrical and material nonlinearity and temperature dependent material properties.

Perricone et.al. [36] presented an experimentally validated theoretical method of scale modeling which demonstrates transient and spatial accuracy of reaction rate, temperature and gas composition in ventilation limited enclosures at three model scales.

Crosti [37] in his paper focused on the structural analysis of a steel structure under fire loading. In this framework, the objective is to highlight the importance of the right choice of analyses to develop, and of the finite element codes able to model the resistance and stiffness reduction due to the temperature increase. In addition, the evaluation of the structural collapse under fire load of a real building is considered, paying attention to the global behavior of the structure itself.

Quiel and Garlock [38] in 2010 proposed a simplified closed-form methodology with which to predict the thermal and structural response of steel perimeter columns in high-rise building frames exposed to fire.

Thomas [39] discussed the suitability of the finite element heat transfer program SAFIR for modeling plaster board-lined light timber frame assemblies and its limitations.

Barowy [40] in his research study he evaluated the potential of the NIST Fire Dynamics Simulator (FDS) and Smoke view to be utilized as a part of a computer-based fire fighter trainer. This study had two objectives: 1) to determine if simulations accurately spread heat and smoke through a multi-level, multi-compartment live fire training facility 2) to determine if the simulations properly reproduce changes in the thermal environment that result from two typical fire fighter actions: opening the front door and fire suppression.

Moss et.al. [41] proposed implementation of an easy-to-use approach for the prediction of the fire resistance of bolted joints is discussed in the paper, based on an extension of the Johansen's yield equations to fire conditions, including a model for the variation of the embedment strength with temperature.

Law et.al. [42] applied a novel methodology for defining a family of possible heating regimes to a framed concrete structure using the concept of travelling fires.

Fang et.al. [43] discussed key issues that should be addressed in the robustness assessment of steel-composite structures subject to localized fire, and proposes robustness assessment approaches that offer a practical framework for the consideration of such issues.

2.2 The Codes

The first European codes were published in the 1980's. EN1993 has approximately twenty parts covering common rules, fire design, bridges, buildings, tanks, silos, pipelined piling, crane supported structures, towers, masts, and chimneys. EN1991-1-2 explains the thermal and mechanical actions in the structural design in buildings exposed to fire.

EN1993-1 also describing the principles, requirements and rules for the steel structure buildings that are affected by fire [44].

2.2.1 Eurocode 3 part 1.2

This code is concentrating on the difference between structures in the normal temperature design and after it gets exposed to high temperatures because of an accidental fire, and give principles and application rules for designing these structures [45].

2.2.2 ASTM

ASTM International, as it is known today is one of the largest, voluntary standards development organizations in the world, there are many ASTM tests on fire.

For example E119-12a which is a Standard Test Method for Fire Tests of Building Construction and Materials also E108-11 is a standard test methods for fire tests of roof coverings and E84-12c is a standard test method for surface burning characteristics of building materials.

2.2.3 AISC

The American Institute for Steel Construction (AISC). This manual first published in 1927. While structural fire design is not a primary focus of the design guides, AISC does produce 'Design Guide 19: Fire Resistance of Structural Steel Framing' in which includes:

Building Code Requirements;Fire Tests; Rated Designs; Fire Protection Materials (including gypsum, masonry, concrete, spray-applied systems, mineral fiberboard); Fire protection for steel columns; Fire protection for steel roof and floor systems; Fire protection for steel trusses; and Engineered fire protection.

2.2.4 International Standards Organization (ISO)

ISO in which founded in 1947, since then have published more than 19 000 International Standards covering almost all aspects of technology and business. Have members in 164 countries, on the basis of one member per country. ISO 9705:1993, ISO/TR 3814:1989, ISO/TR 7248:1985 are related with fire [44].

Fire resistance of structures based only on single element behavior in the standard fire tests was not the adequate answer to the needs of understanding the real structures behavior in fire, because now it has been obvious that the single element member behavior in the standard fire test is not the same if that element member is part in a whole structure.

The structures could be determinate or indeterminate, when there is a collapse load in these structures, the way the determinate structure behaving could be different from the way the indeterminate structure behaving.

In the determinate structures one load path may occur, and then the load increases until this load path cannot sustain any further load in which the collapse occur.

In the indeterminate structures, the redundant structure can find different load paths among many load paths and different load carrying mechanisms to support the additional load on this member by redirecting the load in the member's load path to another better path load in the other members.

So in the high redundant structures there may be high deformations in some parts of the structure but that does not mean necessarily that failure could happen in the structure. The redistribution of the loads in the structure has a great advantage in the fire accidents because it gives the structures more chances for remain stable and survive after the fire ends.

CHAPTER 3

TEMPERATURE LOADS THEORY AND THE SOFTWARE

3.1 General

The structure members during their expected life are faced to many different circumstances and the design precautions may not sustain these hard circumstances.

These circumstances affect on the durability and the stiffness of these members, so the fires and the high temperatures they cause should be given a special concern because of their great effect on the structures behavior.

Today keeping the structure members safe from fires by make them fire resistant has a high priority in the construction field.

To evaluate the stability of the constructions and the strength of the different structure members in a correct way it is necessary to determine the specifications of the construction materials after they get exposed to high temperatures.

In which a change happens in the physical and mechanical properties of the materials is according to the temperature degree and the duration of the fire.

Every type of structures is affected if fire incidents happen in them because the damages probably large. Although sometimes the damage is partial but in a lot of times it is necessary to demolish the building because it is not suitable for use any more.

The beams and slabs are considered as the most exposed members to the high temperatures. In the columns case the upper side is more exposed to high temperature than the lower side of the column.

3.2 Temperature Loadings

Lamont [46, 47] discussed the analytical and numerical analysis of simple beam models in fire and the effect of the temperature loads on them.

Recent experimental and theoretical researches on the composite steel framed structures showed that it is the thermal expansion and thermal bowing induced forces and displacements and not the material degradation that govern the structural response in fire until just before failure [48].

The structural members are exposed to two types of heating, first the thermal expansion, second the thermal bowing. These effects act together that result in thermal strains.

Thermal expansion happens because of increasing in the mean temperature, but in the case of the thermal bowing it happens because of a non-uniform temperature (gradient temperature) distributed over the depth of the member.

When a member is applied to uniform temperature, it means there is no difference between the temperatures from top to bottom of the member. This temperature will only cause an axially change in the member's length if this member has a free end. But if the member's ends are restrained then the temperature will cause axial stresses in the member.

$$\epsilon_{total} = \epsilon_{thermal} + \epsilon_{mechanical} \qquad (1)$$

3.2.1 Thermal expansion

The thermal expansion strain (ϵ_T) is developed because of mean temperature rise that affect on the beam.

$$\epsilon_T = \alpha \, \Delta T \qquad (2)$$

α = Thermal expansion coefficient

If the beam was not restrained then no mechanical strains happens, only thermal strains developed.

$$\epsilon_{total} = \epsilon_t = \epsilon_T = \alpha \, \Delta T \quad (3)$$

$$\epsilon_{mechanical} = \epsilon_m = 0 \quad (4)$$

If the beam was axially restrained pinned end then mechanical strains are developed which are equal and opposite in direction to the developed thermal strains. Initially a compression force F will develop but without deflections. Then the total strains (ϵ_t) in the beam are the result of thermal strains and mechanical strains which is equal to zero.

$$\epsilon_t = \epsilon_T + \epsilon_m = 0 \quad (5)$$

$$\epsilon_m = - \epsilon_T \quad (6)$$

$$F = \sigma \, A = EA\epsilon_m = - EA\epsilon_T = - EA \, \alpha \, \Delta T \quad (7)$$

E= the young's modulus of elasticity (N/mm^2)

3.2.2 Thermal bowing

If the member is applied to a gradient temperature, the temperature in one side of the member is high but in the other side is low, which means there is a difference between the temperature from top to bottom of the member. This will cause particles of the member in the higher temperature side expand, but the particles in the lower temperature side will not expand, and this phenomenon will probably cause bending in the member. (Figure 3.1)

Gradient temperatures are developed in slabs and beams that are exposed to fire temperatures.

The gradient temperature can be represented by equation :

$$T_y = \frac{T1 - T2}{d} \quad (8)$$

T_y= gradient temperature

T1= temperature of the exposed face

T2 = temperature of the unexposed face

d = depth of the section

T1 > T2

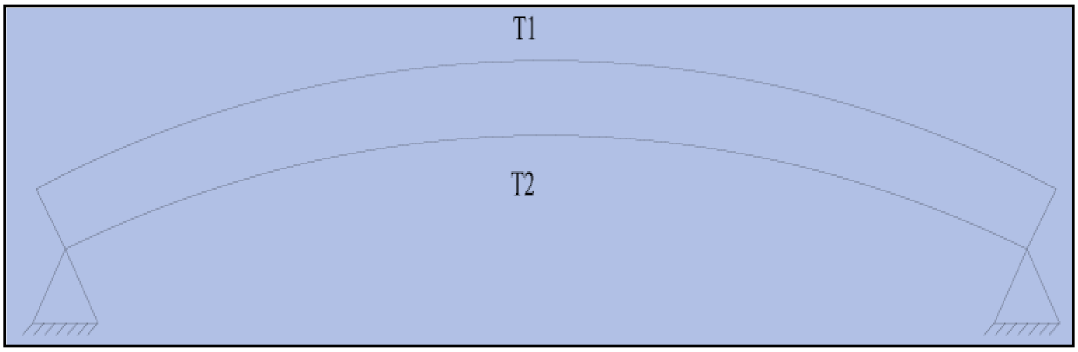

Figure 3.1 Bending caused by gradient temperature

The uniform curvature is happening along the length of the beam:

$$\text{Ø} = \alpha\ T_y \qquad (9)$$

α = thermal expansion coefficient

Ø = the curvature rate of the beam

When real fires occur in the building structures, in fact the members will be subjected to thermal expansion and thermal bowing interaction together. Because it is hard to find a member exposed to fire in all the sides with the same temperature intensity in all the sides. Both of these actions will cause thermal strains inside the members, and if these members are restrained then also mechanical stresses will develop inside these fire exposed members [47].

3.2.3 Behaviors of materials under high temperatures

The structures are facing many hazards during their life. One of these dangerous phenomena is the fire occurrence. Fires by their nature are the main cause for deteriorating of all the materials' strength and stiffness, and the changing of the physical behavior of these materials.

The materials like steel has mechanical properties, these mechanical properties are dependon the temperature change, as much as the temperature increases the mechanical properties decreases, some of these mechanical properties that depend on fire are (modulus of elasticity, yield strength, proof stress, ultimate strength and ductility) [49].

Then if the fire affects the mechanical properties of the structure members, that means it affects the stiffness of these members and reduce it. From here it can be imagined how the fire plays an important role in reducing the stiffness of the members, and this could lead to collapse of these members, which may lead also to the collapse of the whole building.

The structures could possess large reserves of strength that make them more endurance to deflections also 'it is believed that it is the thermal forces and displacements and the material degradation that govern the structural response in fire' [50].

'The degradation of structural materials' stiffness and strength at high temperatures may, in some incidents, cause the structure to collapse under severe fire conditions' [51].

The Euro code 3 part 1.2 [45, 52] made a table for (reduction factors for stress-strain relationship of steel at elevated temperatures) explains the change of the mechanical properties according to the temperature increase. So based on this table, a curve can be drawn as in the Figure 3.2, which demonstrates the decreasing of the Elasticity as the temperature increases until it gets to zero at the temperature 1200 C°.

For example if the temperature of the member affected by fire is 300 C° then according to Figure 3.2 the elasticity will be % 80 of the original Elasticity and at 800 C° the elasticity will be % 9 of the original elasticity, that means at temperature 800 C° the member lost 91% of it is elasticity.

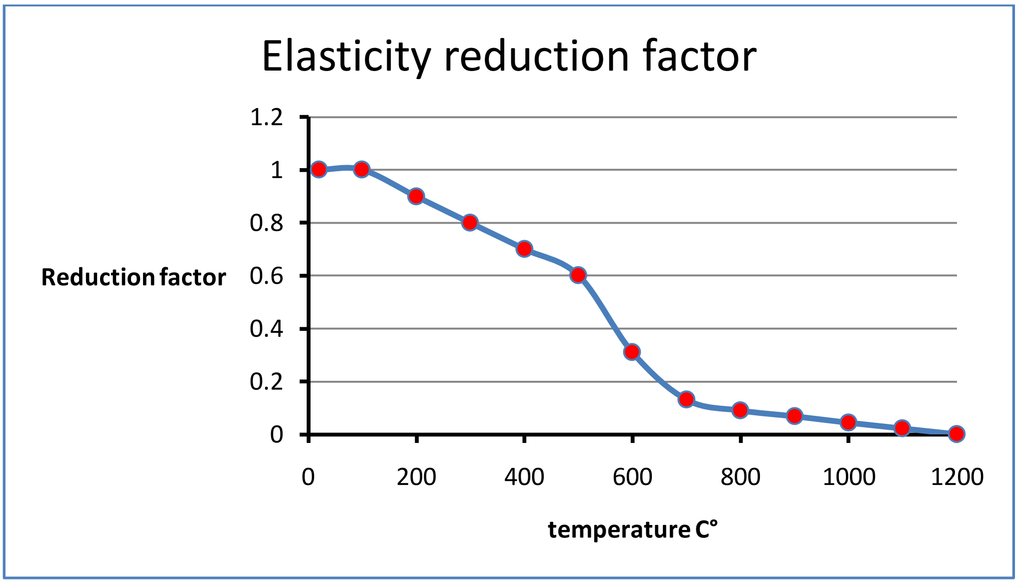

Figure 3.2 Elasticity reduction factor curve

So based on that, in the compartment in which the fire occurs, the members will lose their stiffness and see high deterioration in a fast pace, compared to the other compartments near it, the other compartment members don't lose their stiffness because they didn't exposed to fire.

3.2.4 Changing of temperature along a member

If the member is not subjected to the same amount of temperature along it is length and the temperature increased gradually from the start point of the member to the end point.

For example if there is a column subjected to fire in a compartment, then the temperature near the roof is more than the temperature in the lower parts of the column. So the temperature gradually increases along the column. "In real fire compartments, experiments have shown, for a wide range of compartment fires, that it is reasonable to assume that the room becomes divided into two distinct layers: a hot upper layer consisting of a mixture of combustion products and entrained air, and a cold lower layer consisting of air" [53].

In general, the STAAD pro cannot apply different temperatures to one element member in the same time. But to solve this problem the member can be transformed into several elements as it is needed by inserting nodes in to the member as it is shown in Figure 3.3. Then applying the temperature loads to these elements from the highest temperature to the lower temperature along the member respectively.

Single element member eight element member

Figure 3.3 Dividing a member to many elements by adding nodes

3.3 STAAD PRO

STAAD PRO is a structural analysis and design computer program; it is one of the widely used software programs in the world. This program can model the structures with the capability of modifying the input data in an easy way anywhere and anytime during working.

Also this program has the capability to do analysis of static, P-delta, pushover, response spectrum, time history, cable (linear and non-linear), buckling and steel, concrete and timber design. For this study, the linear static analysis has been adopted [54].

STAAD PRO program can also be used for modeling and analyzing structures that are affected by fire temperatures, in which the program can apply temperature loads for any member.

Coefficient of thermal expansion (ALPHA) is used to calculate the expansion of the members if temperature loads are applied. The temperature unit for temperature load and ALPHA has to be the same.

In the STAAD PRO the temperature loads come in two forms: the first one is the uniform load, and the second one is the gradient load.

a-The uniform mean temperature load in the STAAD PRO is called (temperature change for axial elongation).

b-The gradient temperature load in STAAD PRO is called (temperature differential), which it is also has two branches, one is (from top to bottom) and the other one is (from side to side) of the member. These could be used to apply gradient temperature in different axis directions depending on the member's situation. For example if the temperature load was (from top to bottom) type then it is different when it is applied to a beam than when it is applied to a column.

Also if the gradient temperature load was (from side to side) type, when it is applied to two beams, each one is aligned in one axis direction then these beams react differently to this temperature because they are not in the same axis direction. So if gradient temperatures applied to members it is wise to know previously the condition of these members. In which axis they lies, are they beams or columns. Because if a wrong temperature case applied to a member this could alternate the hot and the cold places for the member.

The program calculates the axial strain (elongation and shrinkage) due to the temperature difference for members. From this it calculates the induced forces in the member and the analysis is done accordingly. The strain intervals of elongation and shrinkage can be input directly.

The $f_{1,}$ f_2, and f_4 input parameters are shown in Figure 3.4, in the STAAD PRO these temperature load parameters are defined as below:

f_1 = the average change in temperature (from ambient "stress-free" temperature) in the member/element which will cause axial elongation in the members or uniform volume expansion in plates and solids. The temperature unit is the same as the unit chosen for the coefficient of thermal expansion ALPHA under the CONSTANT command. (Members/Plates/Solids).

f_2 = the temperature differential from the top to the bottom of the member or plate ($T_{topsurface}$-$T_{bottomsurface}$). If f_2 is omitted, no bending will be considered. (Local Y axis Members/ Local Z axis Plates). Section depth must be entered for prismatic.

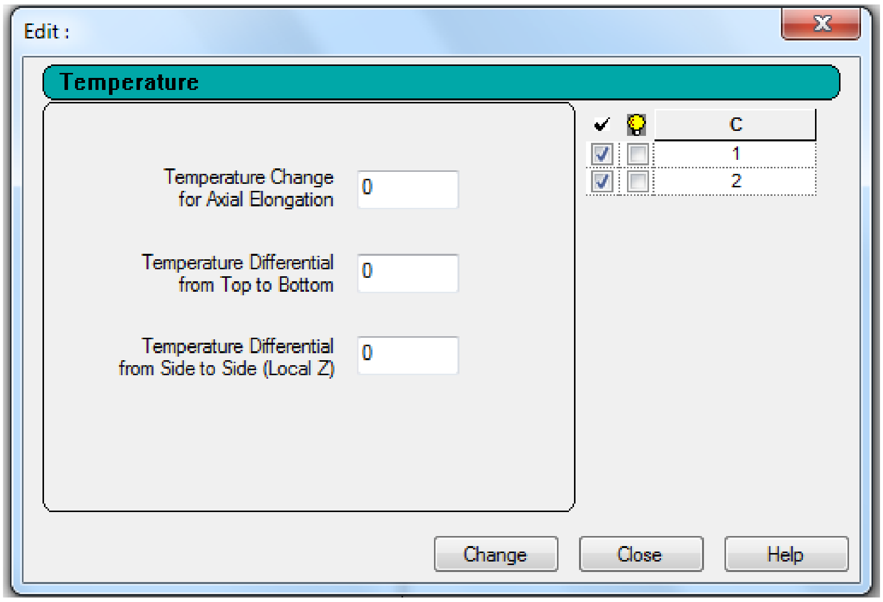

Figure 3.4 Temperature parameters in STAAD pro

f_4 = the temperature differential from side to side of the member. (Local Z axis) (Members only). Section or flange width must be entered for prismatic [54].

3.3.1 Verification example

A structure composed of two stories, each story is 3m high. The structure has three bays in x-axis direction and z-axis direction (y direction for sap2000).Every bay width is 6m. All members are of steel W14×22 sections. The material properties are Young's Modulus (E) = 1.999×10^8kN/m^2, Poisson's ratio = 0.3, Density = 76.9729 kN/m^3, Thermal coefficient = 1.17e-005/F°.

Analyzing the structure using STAAD PRO and SAP2000 programs for the comparison between the results. The structure members are not subjected to any type of loads. Except in a single member which is applied to 600°C uniform temperature and another time to 600°C gradient temperature. See Figure 3.5 and Figure 3.6. The results are shown in Tables 3.1 and 3.2.

For comparison between the two programs two joints (1 and 2) are selected in which the reaction forces and moments are compared. Figure 3.5 and Figure 3.6.

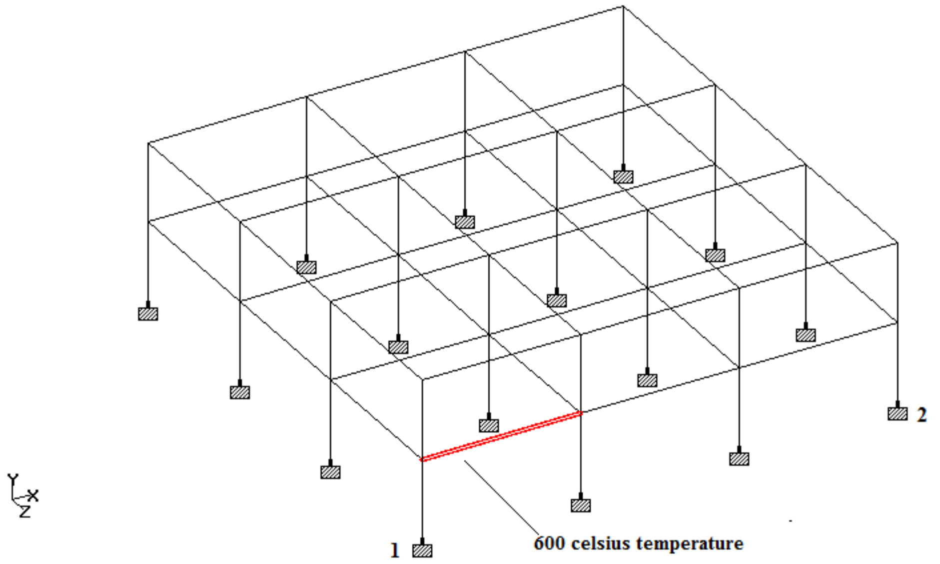

Figure 3.5 Member applied to 600°C temperature in STAAD pro

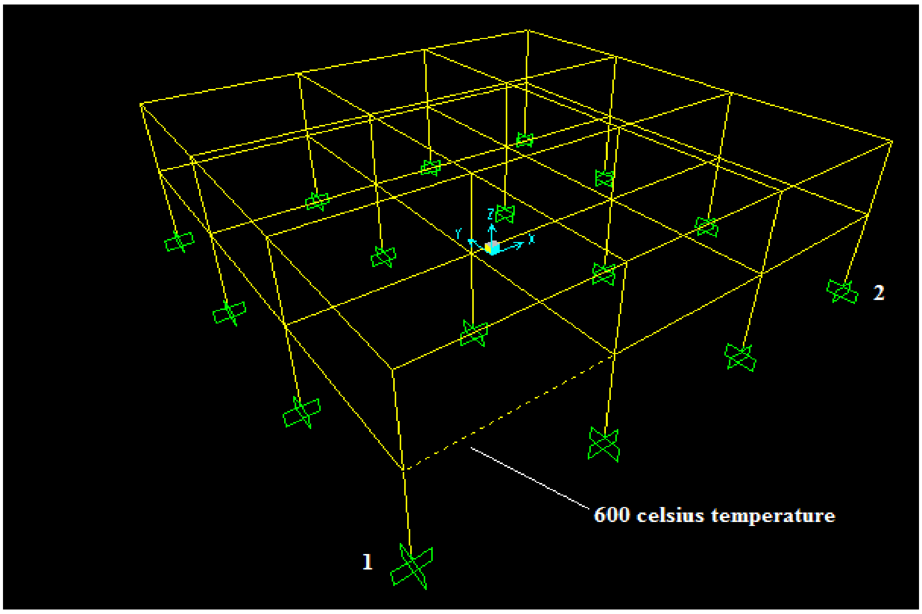

Figure 3.6 Member applied to 600°C temperature in SAP2000

Table 3.1 Uniform temperature comparison between STAAD PRO and SAP2000

Uniform temperature 600°C					
Joints	force and moments	program	X	Y	Z
Joint 1	Force (kN)	STAAD PRO	170.948	-15.357	0.047
		SAP2000	170.985	-15.361	0.047
	Moment (kN.m)	STAAD PRO	0.073	0.008	-266.676
		SAP2000	0.073	0.008	-266.734
Joint 2	Force (kN)	STAAD PRO	-50.585	-6.211	0.053
		SAP2000	-50.596	-6.213	0.053
	Moment (kN.m)	STAAD PRO	0.082	-0.002	79.923
		SAP2000	0.082	-0.002	79.941

Table 3.2 Gradient temperature comparison between STAAD PRO and SAP2000

Gradient temperature 600°C					
Joints	force and moments	program	X	Y	Z
Joint 1	Force (kN)	STAAD PRO	-69.772	3.790	0.017
		SAP2000	-69.992	3.802	0.017
	Moment (kN.m)	STAAD PRO	0.025	0.000	60.259
		SAP2000	0.025	0.000	60.447
Joint 2	Force (kN)	STAAD PRO	4.427	-2.880	-0.015
		SAP2000	4.441	-2.889	-0.015
	Moment (kN.m)	STAAD PRO	-0.023	0.000	-8.004
		SAP2000	-0.023	0.000	-8.029

CHAPTER 4

EXAMPLES

4.1 Base Example

A steel structure is composed of three floors; every floor is 3 m high, 4 bays in the x direction and 4 bays in the y direction, every bay is 4 m x 4 m, as it is shown in Figure 4.1 and Figure 4.2.

The density of steel= 76.8195 kN/m^3, thermal coefficient (α)=1.2×10^{-5}/F°, young's modulus (E)= 2.05×10^8kN/m^2 (at temperature 20C°), Yield strength of steel = 248213 kN/m^2.

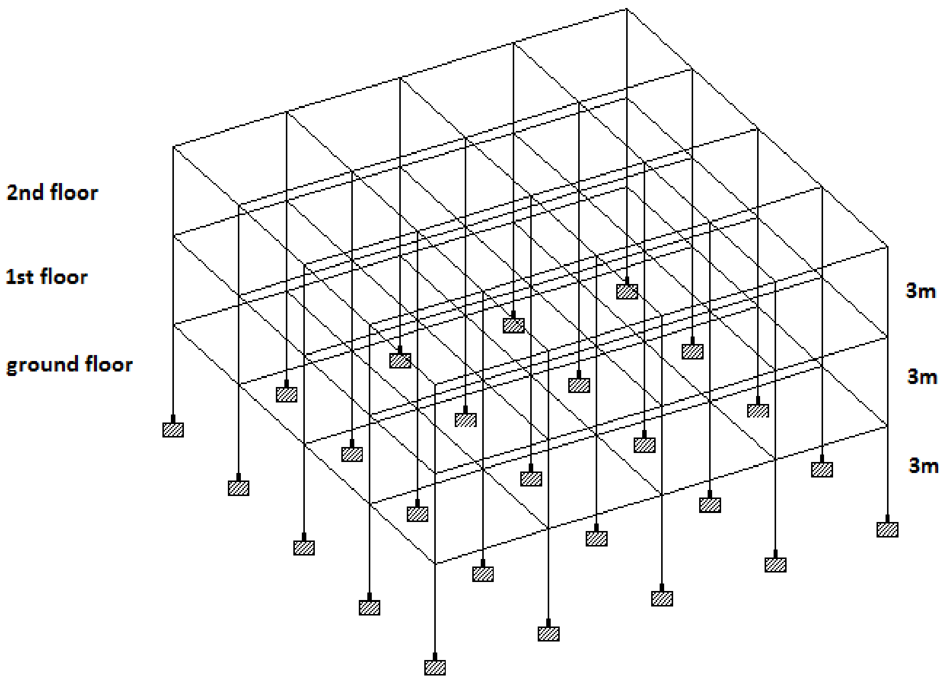

Figure 4.1 Isometric view of the 3D structure

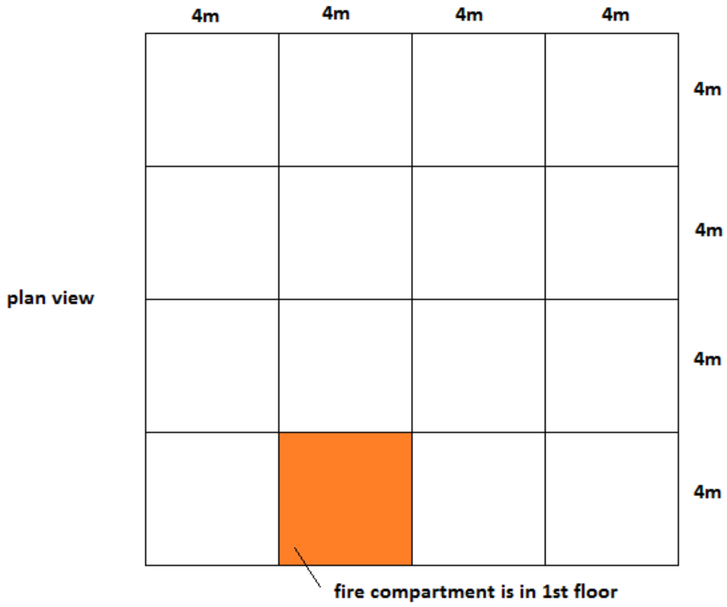

Figure 4.2 Plan view of the structure including the fire compartment in the first floor

All the beams are UB457x191x98, all the columns are UC305x305x97, besides of the self-weight of the members there are (DL=4.32 kN/m^2), (LL=2.4 kN/m^2).

As it is seen in Figure 4.3 there are 4 beams and 4 columns highlighted that are exposed to the temperature loads (uniform, gradient) in the fire compartment which is located in the first floor. Only these members are exposed to the fire in the building. All the members of the fire compartment are adopting elasticity values from Figure 3.2.

Starting from no fire in the compartment which means the temperature is equal to ambient temperature that is 20 C°. Then all the members of the fire compartment in Figure 4.3 are assumed to expose to a different temperature every time starting from 120 C° then increased to 220 C° then to 320 C° then to 420 C° then at the last they exposed to 520 C°.

The deflections and stresses that are adopted in all the members in the examples are the maximum deflections and maximum stresses in these members. For the beams the deflection is in y direction. For the columns the deflection is in two directions, x direction and z direction.

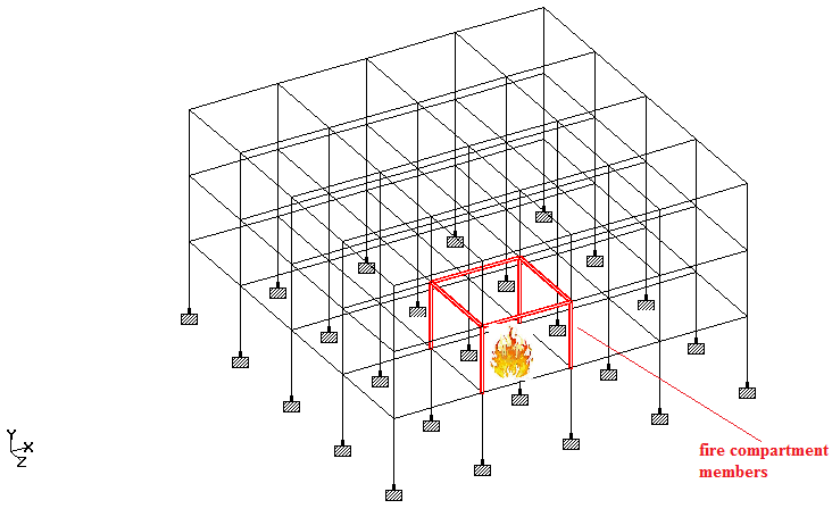

Figure 4.3 The fire compartment members in which exposed to fire temperature

4.1.1 The labeling of members

It is necessary to choose a specified member groups for observing the fire effects on that members. For the base case example (section 4.1), changing the fire location example (section 4.2) and adding two stories example (section 4.3), the same three groups (A, B and C) are considered to see the effects of fires on the members. Also there is group D (section 4.4), which is an exclusive group used to see the difference between single element members and multi element members. So the deflection and stress results developed in the members will only be observed in the groups (A, B, C and D).

The group A composed of four divisions group A1, group A2, group A3 and group A4 in which all of them are beam members as it can be seen in Figure 4.4 , Table 4.1 and Table 4.2.

The group B composed of four divisions group B1, group B2, group B3 and group B4 in which all of them are beam members as it can be seen in Figure 4.4 and Table 4.3 and Table 4.4.

The group C composed of three divisions group C1, group C2 and group C3 in which all of them are column members as it can be seen in Figure 4.4 and Table 4.5 Table 4.6 and Table 4.7.

The members of group A and B are beams where the members of group C are columns; also the group D members are beams. The members of groups and all the other members are included in the next cross section frame figures (Figure 4.4 and Figure 4.5).

The fire compartment members are composed of four beams and four columns. They are labeled as four beams B4.2.4, B8.2.4, B2.2.12, and B2.2.16 see Figures 4.4 and 4.5, with four columns C4.2.3, C4.2.4, C8.2.3, and C8.2.4 see Figure 4.4.

The labeling of fire compartment members in the section 4.2 is different from the above labeling because in that section the fire compartment location is changed.

The names of group members and the labeled members are exist in the result tables, so in the below figures (Figure 4.4 and Figure 4.5) are only the members without the groups.

All the beam deflection results are in Y-axis direction. But the columns have deflections in two axis directions X and Z.

All the members in the result tables that start with B letters are beams. All the members start with C letter are columns.

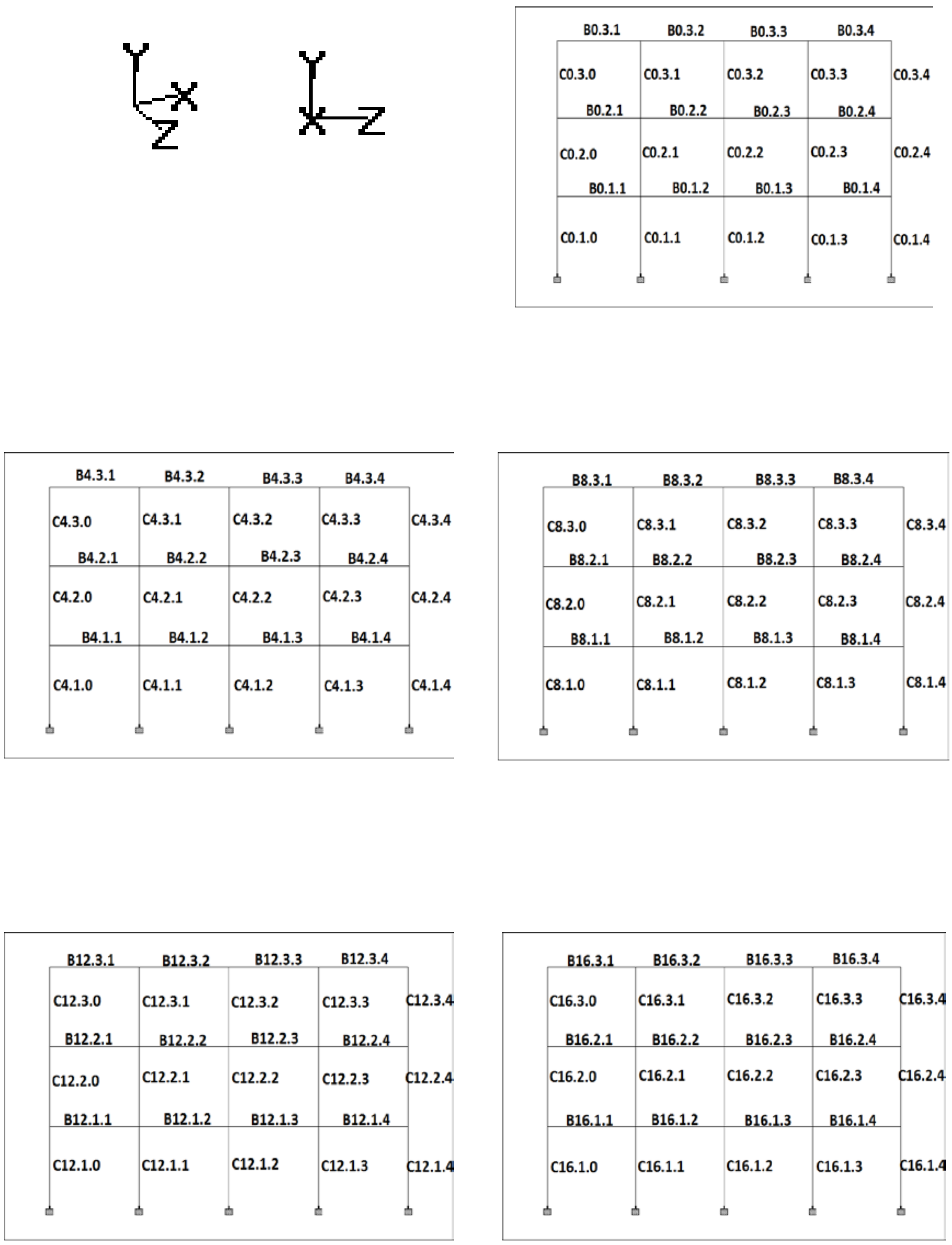

Figure 4.4 Cross sections for the structure in X-axis direction with member’s label

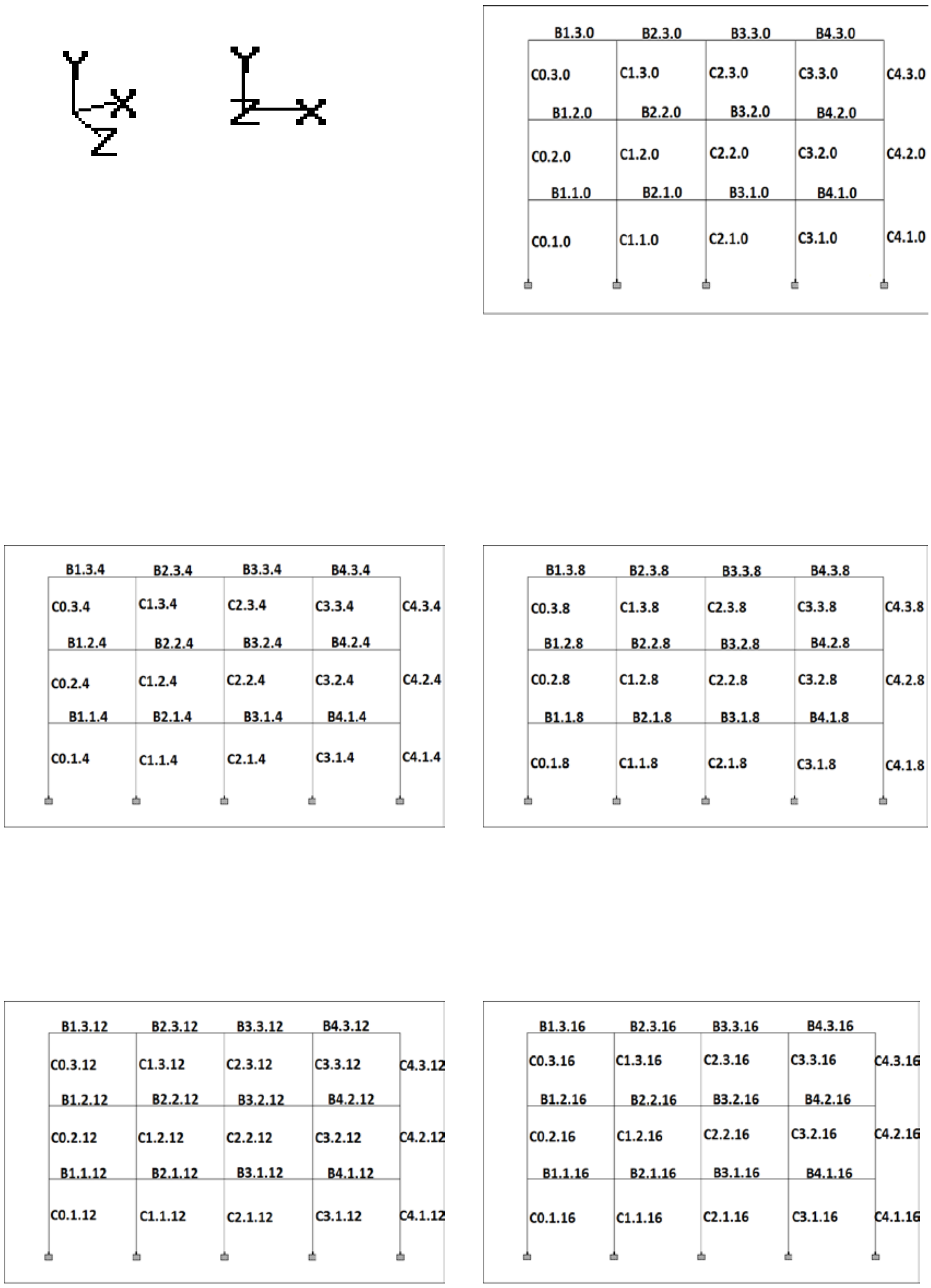

Figure 4.5 Cross sections for the structure in Z-axis direction with member's label

4.1.2 Discussions of results for the base case

The group A1 members are near the fire compartment especially the member B4.2.4 is one of the fire compartment members because of that the largest deflection and stress in the group occurs in that member. Also the member B4.3.4 which is in the second floor above the fire compartment has a considerable stress and deflection, but the member B4.1.4 in the ground floor below the fire compartment has the less deflection and stress in the group although it is directly below the fire compartment. See Tables 4.1 and 4.2.

In group A2 that is not far from the fire compartment some different pattern will be seen that the middle member B4.2.3 and the member above it B4.3.3 are showing nearly the same deflection. The lower member B4.1.3 has the lowest deflection in the group. See Table 4.1

The group A3 and group A4 that are far from the fire compartment act in a different way. In general the deflection will be small in these groups but gradually the member in the higher floor will be higher deflected if compared to the other floors beneath it, for example in the group A4 which is the farthest group from fire, the member B4.3.1 located in the higher floor has the largest deflection rationally then come member B4.2.1 in the middle floor then member B4.1.1 in the lower floor.

So what is noticed from group A pack is the near groups from fire have a higher deflection than the farthest groups. The deflection in the lower floors was less than the higher floors. In the groups that are near to the fire compartment the middle members are the members of the higher deflection, because the middle members are in the same plane of action of the fire affected beams, so because of the continuity between these beams the forces that make deflection are transferred directly and more actively between these beams.

The higher floors are not restrained while the lower floors are restrained with ground which makes the effects of the fire temperature affect more on the higher floors rather than lower floors.

The stresses in the group A1 and A2 were the higher among group A especially the members B4.2.4 and B4.2.3 exceeded the yield strength and failed, see Table 4.2.

The group B pack is far from the fire compartment, so the pattern in group A pack will not be repeated in the group B pack. In Table 4.3 it is obvious that the group B1, group B2, group B3 and group B4 will act in the same way. In all these groups the deflection in the higher floor is the maximum and then decreases in the middle floor to the minimum in the lower floor. It is noted here that besides the difference in deflection between the members of group B but all the deflections are small. The stresses of group B members are lower than yield strength so no failure happened, see Table 4.4.

For the columns in the group C pack the larger deflections happens in the group C2 especially in the two members C8.2.4, C8.2.3 because they are part of the fire compartment, see Tables 4.5 and 4.6.

The larger stresses happened in the columns C8.2.4, C8.2.3 and C8.3.3 in which exceeded the yield strength and failed, see Table 4.7.

For understanding and knowing which members are failed in groups A, B and C see Figure 4.6.

Table 4.1 Group A deflection results for the base case

The base case							
group A		Maximum deflections in members (mm) y-axis					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group A1	B4.1.4	-0.58	-0.94	-1.26	-1.55	-1.80	-2.00
	B4.2.4	-0.80	6.37	13.55	20.81	28.13	35.52
	B4.3.4	-0.96	3.07	6.54	9.99	13.41	16.79
group A2	B4.1.3	-0.56	-0.63	-0.83	-1.01	-1.18	-1.33
	B4.2.3	-0.86	2.47	5.60	8.69	11.73	14.67
	B4.3.3	-0.99	2.25	5.30	8.31	11.27	14.13
group A3	B4.1.2	-0.56	-0.51	-0.46	-0.49	-0.50	-0.52
	B4.2.2	-0.86	-1.13	-1.39	-1.63	-1.85	-2.04
	B4.3.2	-0.99	-1.12	-1.24	-1.38	-1.51	-1.66
group A4	B4.1.1	-0.58	-0.60	-0.62	-0.64	-0.66	-0.69
	B4.2.1	-0.80	-0.77	-0.8	-0.84	-0.88	-0.91
	B4.3.1	-0.96	-0.98	-1.001	-1.02	-1.05	-1.1

Table 4.2 Group A stress results for the base case

The base case							
group A		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group A1	B4.1.4	-12817	-18445	-25585	-34992	-43214	-49355
	B4.2.4	-11484	251773	448378	600398	706840	766473
	B4.3.4	12264	-30366	-48674	-66382	-83299	-99133
group A2	B4.1.3	-12640	-19272	-25073	-29885	-33558	-35911
	B4.2.3	-12570	98485	178653	250799	313324	364140
	B4.3.3	12826	-33142	-54712	-76241	-97607	-118588
group A3	B4.1.2	-12640	-15727	-18472	-20833	-22733	-24071
	B4.2.2	-12570	26817	52356	76362	98503	118331
	B4.3.2	12826	14957	-25909	-38943	-51989	-64980
group A4	B4.1.1	-12817	-11409	-16397	-20866	-24704	-27763
	B4.2.1	-11484	14547	21056	26958	32055	36161
	B4.3.1	12264	15032	17756	20327	22706	24834

Table 4.3 Group B deflection results for the base case

The base case							
group B		Maximum deflections in members (mm) y-axis					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group B1	B16.1.4	-0.30	-0.32	-0.33	-0.34	-0.36	-0.37
	B16.2.4	-0.42	-0.43	-0.44	-0.46	-0.47	-0.47
	B16.3.4	-0.5	-0.51	-0.51	-0.52	-0.52	-0.52
group B2	B16.1.3	-0.29	-0.3	-0.30	-0.31	-0.31	-0.32
	B16.2.3	-0.44	-0.45	-0.46	-0.46	-0.47	-0.47
	B16.3.3	-0.51	-0.52	-0.52	-0.52	-0.52	-0.53
group B3	B16.1.2	-0.29	-0.29	-0.29	-0.29	-0.29	-0.29
	B16.2.2	-0.44	-0.44	-0.44	-0.44	-0.44	-0.44
	B16.3.2	-0.51	-0.51	-0.51	-0.51	-0.51	-0.51
group B4	B16.1.1	-0.30	-0.30	-0.30	-0.3	-0.30	-0.3
	B16.2.1	-0.42	-0.42	-0.42	-0.42	-0.42	-0.42
	B16.3.1	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5

Table 4.4 Group B stress results for the base case

The base case							
group B		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group B1	B16.1.4	-6791	-7133	-7483	-7843	-8215	-8600
	B16.2.4	-6093	-8519	10933	13341	15698	17985
	B16.3.4	6451	6633	6698	6735	6738	6699
group B2	B16.1.3	-6677	-7335	-8185	-8991	-9747	-10444
	B16.2.3	-6626	-13662	20559	27252	33663	39705
	B16.3.3	6766	7247	7613	7825	7856	7674
group B3	B16.1.2	-6677	-6867	-7064	-7270	-7487	-7715
	B16.2.2	-6626	9334	12096	14818	17482	20061
	B16.3.2	6766	6917	7056	7183	7298	7398
group B4	B16.1.1	-6791	-6724	-6653	-6581	-6506	-6430
	B16.2.1	-6093	-6386	6677	6992	7295	7584
	B16.3.1	6541	6815	7200	7548	7852	8103

Table 4.5 Group C results for the base case deflection in the X-axis direction

The base case							
group C / X-axis direction		Maximum deflections in members (mm)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	0	0.26	0.47	0.65	0.78	0.86
	C8.1.3	0	0.28	0.51	0.71	0.86	0.96
	C8.1.2	0	-0.01	-0.02	-0.04	-0.05	-0.07
	C8.1.1	0	0	0	0	0	0.003
	C8.1.0	0	0	0.01	0.01	0.01	0.01
group C2	C8.2.4	0	5.08	10.15	15.21	20.25	25.25
	C8.2.3	0	5.12	10.24	15.36	20.46	25.54
	C8.2.2	0	-0.03	-0.05	-0.09	-0.12	-0.17
	C8.2.1	0	0	0	0.01	0.01	0.01
	C8.2.0	0	0.01	0.01	0.02	0.02	0.03
group C3	C8.3.4	0	1.74	3.43	5.06	6.61	8.05
	C8.3.3	0	1.79	3.53	5.21	6.83	8.34
	C8.3.2	0	-0.04	-0.08	-0.13	-0.19	-0.26
	C8.3.1	0	0	0.01	0.01	0.01	0.01
	C8.3.0	0	0	0.02	0.021	0.03	0.03

Table 4.6 Group C results for the base case deflection in the Z-axis direction

The base case							
group C / Z-axis direction		Maximum deflections in members (mm)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	0.11	0.31	0.5	0.66	0.79	0.88
	C8.1.3	0	0.11	0.2	0.29	0.37	0.44
	C8.1.2	0	0.08	0.15	0.22	0.28	0.32
	C8.1.1	0	0.06	0.12	0.18	0.23	0.28
	C8.1.0	-0.11	-0.05	0.06	0.09	0.12	0.16
group C2	C8.2.4	0.07	7.31	14.64	22.05	29.56	37.20
	C8.2.3	0.01	-5.35	-10.59	-15.67	-20.58	-25.28
	C8.2.2	0	-1.29	-2.47	-3.51	-4.4	-5.09
	C8.2.1	-0.01	-1.27	-2.42	-3.44	-4.31	-4.99
	C8.2.0	-0.07	-1.25	-2.39	-3.40	-4.26	-4.93
group C3	C8.3.4	0.09	3.38	6.87	10.5	14.28	18.25
	C8.3.3	-0.01	-1.34	-2.56	-3.65	-4.57	-5.29
	C8.3.2	0	-1.48	-2.84	-4.05	-5.09	-5.92
	C8.3.1	0.01	-1.27	-2.42	-3.44	-4.31	-4.99
	C8.3.0	-0.09	-1.27	-2.39	-3.42	-4.3	-4.99

Table 4.7 Group C stress results for the base case

The base case							
group C		Maximum deflections in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	23942	60420	92395	119452	141212	157230
	C8.1.3	29780	75367	115970	151130	180279	202708
	C8.1.2	29052	25045	21387	-28873	-36760	-43453
	C8.1.1	29780	34424	38837	43009	46869	50320
	C8.1.0	23942	20855	18016	16732	18633	20792
group C2	C8.2.4	23888	451489	792248	1046950	1216180	1300620
	C8.2.3	20025	382023	664401	868132	995292	1048470
	C8.2.2	19483	26314	-39716	-56789	-71443	-83090
	C8.2.1	20025	48765	75435	99421	120146	136850
	C8.2.0	23888	45233	64711	82121	97013	108800
group C3	C8.3.4	22699	88296	177260	266857	356958	447249
	C8.3.3	10194	72290	129346	179875	224647	282594
	C8.3.2	9715	47196	82341	-117142	-150840	-180745
	C8.3.1	10194	10879	11899	13561	15916	19130
	C8.3.0	22699	18939	15713	-16294	-19872	-22141

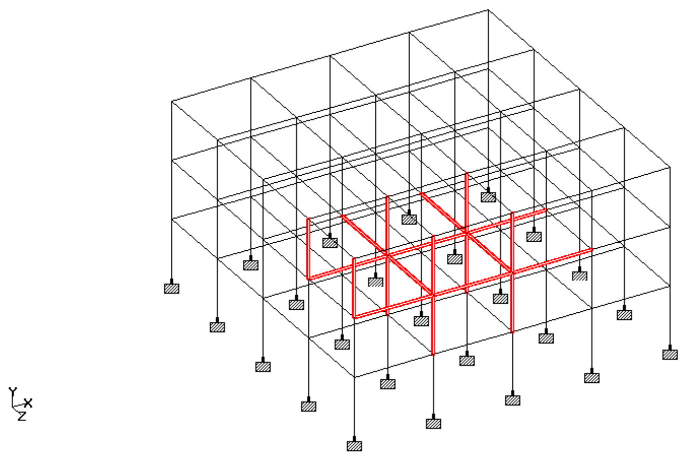

Figure 4.6 20 Failed members in 520°C temperature

4.2 Fire Compartment in a Different Location Example

In this section the location of fire compartment will be moved to see if there are any differences with the previous section results. The fire compartment assumed to be located in the corner of the first floor building as shown in Figure 4.7.

The fire compartment members are composed of four beams and four columns. The four beams labeled as B0.2.4, B4.2.4, B1.2.12 and B1.2.16 see Figure 4.4 with four columns C0.2.12, C1.2.12, C0.2.16 and C1.2.16 from Figure 4.5.

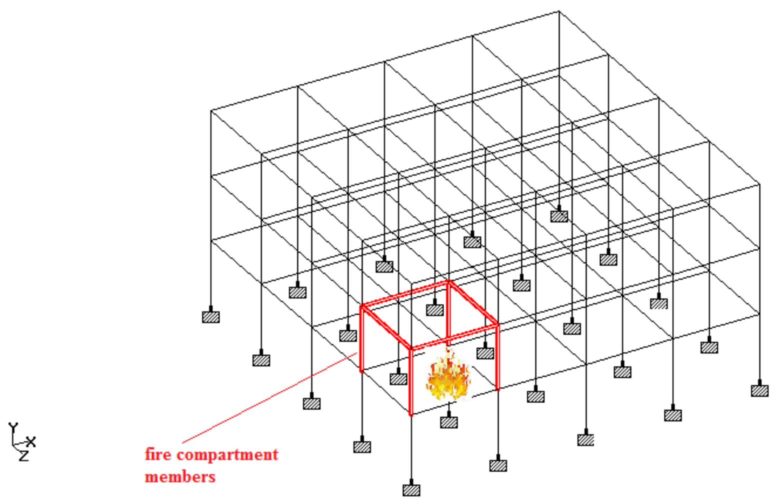

Figure 4.7 Fire compartment located in the corner of first floor

4.2.1 Discussion of results of fire compartment in a different location

From the Tables 4.8 and 4.10 it is noted that the group A pack and group B pack in this section show the same pattern as in the section 4.1. The group A again has the highest deflections in their members especially in group A1. Again the middle floors have the highest deflections in groups that are near to the fire compartment, because their members are parallel and located in the same line of effect with the members of the fire compartment. For the members of group B that are far from fire compartment again the highest floors have the larger deflections then comes the middle floor then the lower floor. For the columns the members of fire compartment are the most affected by the temperature.

For the group A the highest stress happened in the members B4.2.4 and B4.2.3 in the group A1 and group A2 in which exceeded the yield strength of the steel.

But for the group B members the stresses were not large enough to make any threats to the structure.

For the columns the maximum deflections happened in the x-axis direction in the group C3 in the members C8.3.4 and C8.3.3 then in the group C2 in the members C8.2.4 and C8.2.3, in general the deflections were not high.

About stresses in the columns large stresses observed, but no column from group C has reached yield strength and failure.

For understanding and knowing which members are failed in groups A, B and C see Figure 4.8.

Table 4.8 Group A deflection results for the different fire compartment

Different fire compartment							
group A		Maximum deflections in members (mm) (y-axis)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group A1	B4.1.4	-0.58	-0.97	-1.37	-1.76	-2.15	-2.54
	B4.2.4	-0.80	6.30	13.39	20.48	27.57	34.66
	B4.3.4	-0.96	2.99	6.40	9.81	13.22	16.63
group A2	B4.1.3	-0.56	-0.67	-0.91	-1.15	-1.38	-1.62
	B4.2.3	-0.86	2.56	5.85	9.15	12.46	15.76
	B4.3.3	-0.99	2.17	5.21	8.24	11.28	14.31
group A3	B4.1.2	-0.56	-0.51	-0.47	-0.49	-0.51	-0.52
	B4.2.2	-0.86	-1.13	-1.40	-1.67	-1.94	-2.22
	B4.3.2	-0.99	-1.11	-1.23	-1.36	-1.48	-1.60
group A4	B4.1.1	-0.58	-0.60	-0.62	-0.64	-0.67	-0.69
	B4.2.1	-0.80	-0.77	-0.80	-0.84	-0.88	-0.92
	B4.3.1	-0.96	-0.98	-1.00	-1.02	-1.06	-1.11

Table 4.9 Group A stress results for different fire compartment

Different fire compartment							
group A		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group A1	B4.1.4	-12817	-18520	-26985	-38055	-50740	-63425
	B4.2.4	-11484	254746	498155	741563	984972	1228380
	B4.3.4	12264	-31192	-51093	-70994	-90895	-110796
group A2	B4.1.3	-12640	-19406	-26173	-32939	-39705	-46472
	B4.2.3	-12570	90108	168800	247492	326184	404876
	B4.3.3	12826	-36213	-61064	-85915	-110766	-135617
group A3	B4.1.2	-12640	-15569	-18497	-21425	-24354	-27282
	B4.2.2	-12570	25348	50697	76045	101394	126742
	B4.3.2	12826	16399	-27460	-41190	-54920	-68650
group A4	B4.1.1	-12817	-11224	-16450	-21676	-26902	-32128
	B4.2.1	-11484	14953	22599	30246	37892	45538
	B4.3.1	12264	15638	19012	22387	25761	-29202

Table 4.10 Group B deflection results for the different fire compartment

Different fire compartment							
group B		Maximum deflections in members (mm) (y-axis)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group B1	B16.1.4	-0.30	-0.32	-0.33	-0.35	-0.36	-0.37
	B16.2.4	-0.42	-0.43	-0.45	-0.47	-0.48	-0.5
	B16.3.4	-0.5	-0.51	-0.53	-0.54	-0.55	-0.57
group B2	B16.1.3	-0.29	-0.30	-0.30	-0.31	-0.32	-0.32
	B16.2.3	-0.44	-0.45	-0.46	-0.47	-0.48	-0.49
	B16.3.3	-0.51	-0.52	-0.53	-0.54	-0.54	-0.55
group B3	B16.1.2	-0.29	-0.29	-0.29	-0.29	-0.30	-0.30
	B16.2.2	-0.44	-0.45	-0.45	-0.45	-0.45	-0.45
	B16.3.2	-0.51	-0.52	-0.52	-0.52	-0.52	-0.52
group B4	B16.1.1	-0.30	-0.30	-0.30	-0.30	-0.3	-0.3
	B16.2.1	-0.42	-0.42	-0.42	-0.42	-0.42	-0.42
	B16.3.1	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5

Table 4.11 Group B stress results for different fire compartment

Different fire compartment							
group B		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group B1	B16.1.4	-6791	-6994	-7251	-7508	-7765	-8022
	B16.2.4	-6093	-7557	-9020	-10484	-11948	-13412
	B16.3.4	6451	6971	7546	8121	8696	9272
group B2	B16.1.3	-6677	-6890	-7338	-7786	-8235	-8683
	B16.2.3	-6626	-11285	-15944	-20602	-25261	-29920
	B16.3.3	6766	9940	13135	16331	19527	22723
group B3	B16.1.2	-6677	-6959	-7256	-7553	-7850	-8146
	B16.2.2	-6626	-7917	9210	10543	11876	13209
	B16.3.2	6766	7366	7988	8611	9233	9855
group B4	B16.1.1	-6791	-6664	-6592	-6520	-6447	-6375
	B16.2.1	-6093	-6007	-5954	-5901	-5849	5813
	B16.3.1	6541	6949	7356	7764	8172	8579

Table 4.12 Group C different fire compartment deflection in the X-axis direction

Different fire compartment							
group C / X-axis direction		Maximum deflections in members (mm)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	0	0.14	0.28	0.42	0.56	0.70
	C8.1.3	0	0.08	0.16	0.24	0.32	0.40
	C8.1.2	0	0.06	0.1	0.2	0.2	0.3
	C8.1.1	0	0.01	0.02	0.03	0.05	0.06
	C8.1.0	0	0.02	0.04	0.06	0.08	0.09
group C2	C8.2.4	0	1.50	2.99	4.49	5.99	7.48
	C8.2.3	0	1.33	2.65	3.98	5.31	6.64
	C8.2.2	0	0.15	0.29	0.44	0.58	0.73
	C8.2.1	0	0.02	0.04	0.06	0.07	0.09
	C8.2.0	0	0.04	0.08	0.13	0.17	0.21
group C3	C8.3.4	0	1.58	3.15	4.73	6.31	7.89
	C8.3.3	0	1.39	2.78	4.17	5.57	6.96
	C8.3.2	0	0.23	0.46	0.69	0.92	1.14
	C8.3.1	0	0.02	0.04	0.06	0.07	0.09
	C8.3.0	0	0.05	0.11	0.16	0.21	0.27

Table 4.13 Group C different fire compartment deflections in the Z-axis direction

Different fire compartment							
group C / Z-axis direction		Maximum deflections in members (mm)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	0.11	-0.07	-0.14	-0.22	-0.30	-0.37
	C8.1.3	0.00	-0.07	-0.15	-0.22	-0.30	-0.37
	C8.1.2	0.00	-0.08	-0.15	-0.23	-0.30	-0.38
	C8.1.1	0.00	-0.08	-0.16	-0.23	-0.31	-0.39
	C8.1.0	-0.11	-0.16	-0.22	-0.28	-0.35	-0.42
group C2	C8.2.4	0.1	-0.2	-0.4	-0.6	-0.8	-1.0
	C8.2.3	0.0	-0.2	-0.4	-0.6	-0.8	-1.0
	C8.2.2	0.0	-0.2	-0.4	-0.6	-0.8	-1.0
	C8.2.1	0.0	-0.2	-0.4	-0.6	-0.8	-1.0
	C8.2.0	-0.1	-0.3	-0.5	-0.6	-0.8	-1.0
group C3	C8.3.4	0.09	-0.33	-0.65	-0.97	-1.29	-1.60
	C8.3.3	-0.01	-0.33	-0.64	-0.96	-1.28	-1.59
	C8.3.2	0.00	-0.32	-0.63	-0.95	-1.27	-1.59
	C8.3.1	0.01	-0.31	-0.62	-0.94	-1.26	-1.57
	C8.3.0	-0.09	-0.39	-0.70	-1.00	-1.31	-1.61

Table 4.14 Group C stress results for the different fire compartment

Different fire compartment							
group C		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	23942	18869	-24839	-37258	-49677	-62096
	C8.1.3	29780	27795	25813	-33729	-44972	-56215
	C8.1.2	29052	30196	31339	32482	33626	34769
	C8.1.1	29780	31233	32979	34724	36469	38214
	C8.1.0	23942	25298	26654	28010	29366	30722
group C2	C8.2.4	23888	26650	-36902	-55353	-73804	-92255
	C8.2.3	20025	26176	-33807	-50711	-67614	-84518
	C8.2.2	19483	20604	21726	22847	23969	25090
	C8.2.1	20025	22259	24845	27431	30017	32603
	C8.2.0	23888	25996	28105	30213	32321	34429
group C3	C8.3.4	22699	40548	-60707	-84858	-110969	-138711
	C8.3.3	10194	30391	-51129	-76694	-102259	-127823
	C8.3.2	9715	10023	10330	10638	10945	11253
	C8.3.1	10194	12410	14626	16884	19248	21611
	C8.3.0	22699	24488	26277	28066	29855	31644

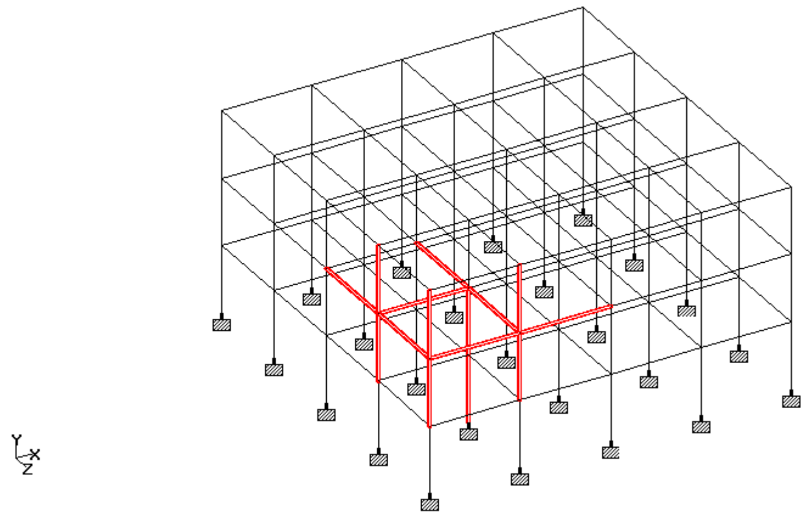

Figure 4.8 15 Failed members in 520°C temperature

4.3 Adding Two Stories to the Base Example

The same base structure (section 4.1) and the location of the fire compartment is the same. But one thing is changed; further two stories are added to it. Also the location of the groups A, B, C does not change, the same members are observed. Figure 4.9 is the structure before adding two stories. Then two stories added to the structure as seen in the Figure 4.10.

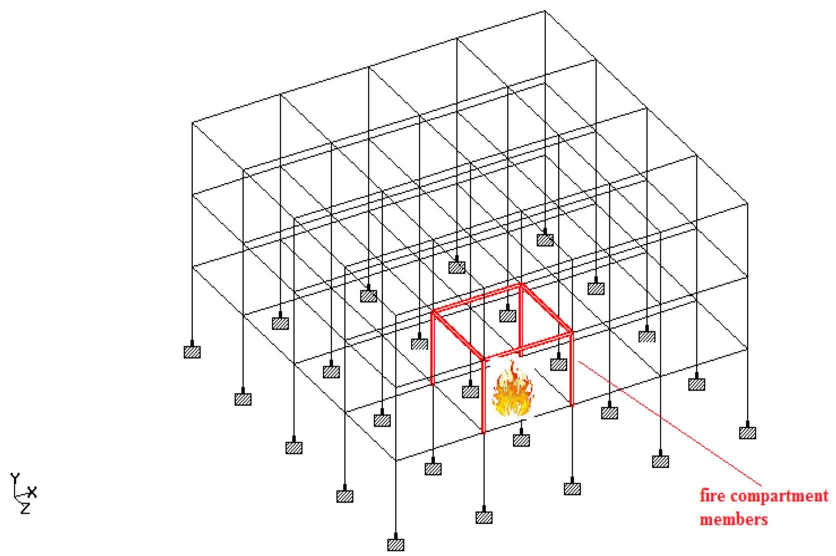

Figure 4.9 Structure and fire compartment before adding two stories

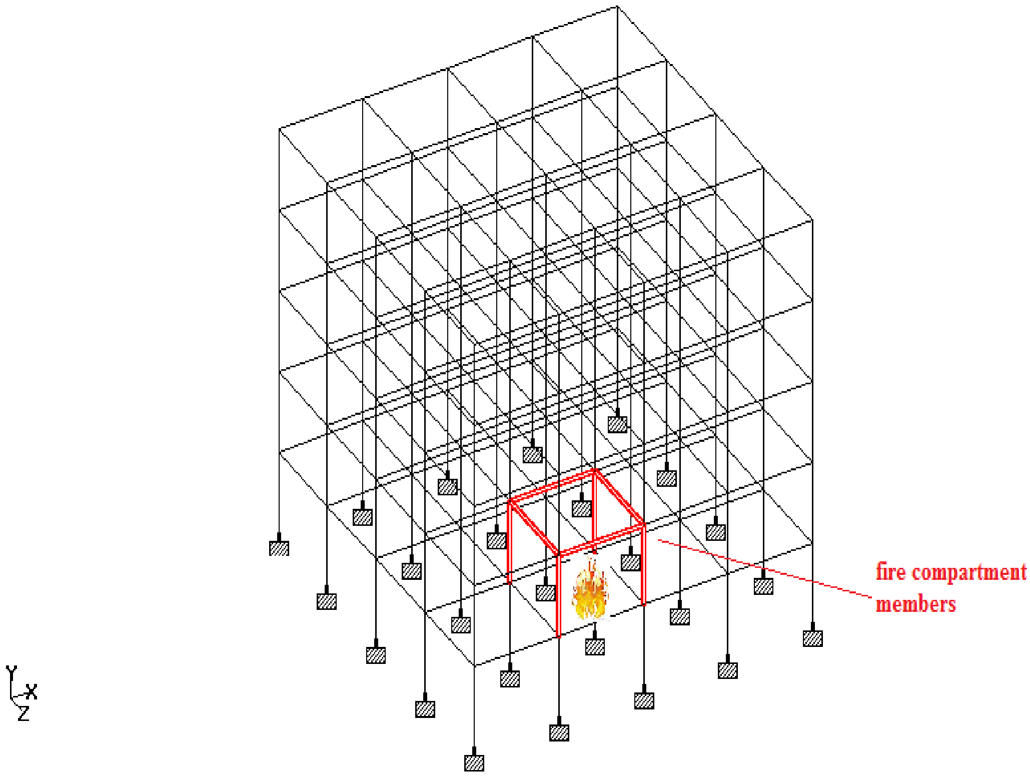

Figure 4.10 Structure and fire compartment after adding two stories

4.3.1 Discussions of results for adding two stories

From the Tables 4.15 and 4.17 it is noted that the group A pack and group B pack in this section show the same deflection pattern as in the section 4.1 and section 4.2. The group A again has the highest deflections in their members especially in group A1.

In group A again the middle floors have the highest deflections in groups that are near to the fire compartment, because its members parallel and located in the same line of effect with the members of the fire compartment. See Table 4.15

Also for the members of the group B that are far from fire compartment again the highest floors have the larger deflections then the middle floor then the lower floor. For the columns the members of fire compartment are the most affected by the temperature. Table 4.17.

The stresses in group A is larger than stresses in group B, especially the members B4.2.4 in group A1 and B4.2.3 in group A2 are failed because they exceeded the yield strength of steel. See Table 4.16.

No large deflections detected and no members failed in group B members see Tables 4.17 and 4.18. For the group C members the maximum deflections happened in the Z-axis direction in general, especially in group C2 the members C8.2.4 and C8.2.3. See Table 4.19 and Table 4.20.

The larger stresses happened in the group C2 then C3 then C1. That in group C2 the members C8.2.4 and C8.2.3 failed. In group C3 the members C8.3.4 and C8.3.3 failed. In group C1 only the member C8.1.3 failed. See Table 4.21.

For understanding and knowing which members are failed in groups A, B and C see Figure 4.11.

Table 4.15 Group A deflection results for the two stories added

Two stories added							
group A		Maximum deflections in members (mm) (y-axis)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group A1	B4.1.4	-0.77	-1.22	-1.62	-1.98	-2.29	-2.55
	B4.2.4	-1.2	5.82	12.84	19.91	27.02	34.16
	B4.3.4	-1.55	2.50	5.81	9.09	12.32	15.48
group A2	B4.1.3	-0.81	-0.97	-1.23	-1.48	-1.7	-1.9
	B4.2.3	-1.34	1.81	4.76	7.65	10.83	13.48
	B4.3.3	-1.74	-1.34	4.04	6.77	9.44	11.99
group A3	B4.1.2	-0.81	-0.76	-0.72	-0.74	-0.75	-0.77
	B4.2.2	-1.34	-1.62	-1.84	-2.03	-2.21	-2.37
	B4.3.2	-1.74	-1.84	-1.89	-1.94	-2	-2.06
group A4	B4.1.1	-0.77	-0.82	-0.85	-0.87	-0.9	-0.93
	B4.2.1	-1.2	-1.26	-1.31	-1.34	-1.38	-1.42
	B4.3.1	-1.55	-1.68	-1.73	-1.77	-1.82	-1.86

Table 4.16 Group A stress results for two stories added

Two stories added							
group A		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group A1	B4.1.4	-12193	-18737	-27369	-38476	-47601	-54472
	B4.2.4	-10155	247910	442224	592230	696983	755314
	B4.3.4	-10324	-22611	-36100	-49289	-62021	-74046
group A2	B4.1.3	-12418	-18515	-23832	-28184	-31437	-33426
	B4.2.3	-12460	95470	173089	242751	302880	351425
	B4.3.3	-12589	-29990	-48988	-67645	-85809	-103223
group A3	B4.1.2	-12418	-15142	-17545	-19570	-21144	-23255
	B4.2.2	-12460	25644	49986	72766	93642	112156
	B4.3.2	-12589	-12360	-17969	-27022	-36091	-45118
group A4	B4.1.1	-12193	-11893	-16725	-21022	-24673	-27533
	B4.2.1	-10155	14231	19608	24370	28392	31508
	B4.3.1	-10324	-10803	-12108	-13328	-14443	-15425

Table 4.17 Group B deflection results for the two stories added

Two stories added							
group B		Maximum deflections in members (mm) (y-axis)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group B1	B16.1.4	-0.42	-0.45	-0.47	-0.48	-0.49	-0.50
	B16.2.4	-0.65	-0.71	-0.72	-0.74	-0.75	-0.76
	B16.3.4	-0.85	-0.93	-0.94	-0.95	-0.96	-0.96
group B2	B16.1.3	-0.44	-0.47	-0.47	-0.48	-0.48	-0.49
	B16.2.3	-0.74	-0.79	-0.79	-0.8	-0.81	-0.81
	B16.3.3	-0.95	-1.02	-1.03	-1.04	-1.04	-1.04
group B3	B16.1.2	-0.44	-0.46	-0.46	-0.46	-0.46	-0.46
	B16.2.2	-0.74	-0.78	-0.78	-0.78	-0.78	-0.77
	B16.3.2	-0.95	-1.02	-1.02	-1.02	-1.01	-1.01
group B4	B16.1.1	-0.42	-0.43	-0.43	-0.43	-0.43	-0.43
	B16.2.1	-0.65	-0.69	-0.69	-0.69	-0.69	-0.69
	B16.3.1	-0.85	-0.91	-0.91	-0.91	-0.91	-0.91

Table 4.18 Group B stress results for two stories added

Different fire compartment							
group B		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group B1	B16.1.4	-6425	-7037	-7665	-8277	-8869	-9437
	B16.2.4	-5326	-7979	10704	13383	15984	18480
	B16.3.4	-5419	-5596	-5743	-5868	-5966	-6033
group B2	B16.1.3	-6551	-7539	-8575	-9553	-10465	-11297
	B16.2.3	-6550	-13887	21037	28008	34694	41008
	B16.3.3	-6603	-7690	-8597	-9354	-9930	-10289
group B3	B16.1.2	-6551	-6963	-7378	-7781	-8168	-8536
	B16.2.2	-6550	-9123	11762	14398	17006	19564
	B16.3.2	-6603	-6589	-6529	-6654	-6893	-7023
group B4	B16.1.1	-6425	-6013	-5657	-5495	-5978	-6352
	B16.2.1	-5326	-5453	5596	6107	6572	6970
	B16.3.1	-5419	-5781	-6432	-6999	-7466	-7814

Table 4.19 Group C results for the two stories added deflection in the X-axis direction

Two stories added							
group C / X-axis direction		Maximum deflections in members (mm)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	0	0.26	0.49	0.67	0.80	0.89
	C8.1.3	0	0.29	0.53	0.73	0.89	1.00
	C8.1.2	0	-0.01	-0.02	-0.04	-0.06	-0.08
	C8.1.1	0	0.00	0.00	0.00	0.00	0.00
	C8.1.0	0	0.01	0.02	0.03	0.04	0.05
group C2	C8.2.4	0	5.09	10.18	15.26	20.33	25.36
	C8.2.3	0	5.14	10.29	15.43	20.56	25.67
	C8.2.2	0	-0.03	-0.05	-0.09	-0.13	-0.18
	C8.2.1	0	0.00	0.01	0.01	0.01	0.00
	C8.2.0	0	0.02	0.05	0.07	0.09	0.11
group C3	C8.3.4	0	1.76	3.47	5.12	6.70	8.18
	C8.3.3	0	1.81	3.58	5.29	6.93	8.48
	C8.3.2	0	-0.04	-0.08	-0.13	-0.19	-0.26
	C8.3.1	0	0.01	0.01	0.01	0.00	-0.01
	C8.3.0	0	0.04	0.07	0.10	0.13	0.16

Table 4.20 Group C results for the two stories added deflection in the Z-axis direction

Two stories added							
group C / Z-axis direction		Maximum deflections in members (mm)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	0.12	0.37	0.61	0.81	0.97	1.10
	C8.1.3	0.01	0.17	0.32	0.45	0.58	0.68
	C8.1.2	0.00	0.13	0.25	0.37	0.47	0.56
	C8.1.1	-0.01	0.11	0.22	0.32	0.42	0.50
	C8.1.0	-0.12	0.08	0.17	0.25	0.33	0.40
group C2	C8.2.4	0.09	7.45	14.90	22.43	30.05	37.79
	C8.2.3	0.02	-5.24	-10.37	-15.37	-20.20	-24.83
	C8.2.2	0.00	-1.15	-2.18	-3.09	-3.85	-4.42
	C8.2.1	-0.02	-1.12	-2.13	-3.02	-3.75	-4.31
	C8.2.0	-0.09	-1.12	-2.10	-2.98	-3.71	-4.26
group C3	C8.3.4	0.10	3.53	7.16	10.93	14.84	18.93
	C8.3.3	0.02	-1.20	-2.28	-3.23	-4.02	-4.62
	C8.3.2	0.00	-1.32	-2.52	-3.57	-4.47	-5.16
	C8.3.1	-0.02	-1.12	-2.13	-3.02	-3.75	-4.31
	C8.3.0	-0.10	-1.11	-2.10	-2.98	-3.71	-4.26

Table 4.21 Group C stress results for two stories added

Two stories added							
group C		Maximum stresses in members (kN/m^2)					
subdivision	members	20°C	120°C	220°C	320°C	420°C	520°C
group C1	C8.1.4	34150	76768	113234	144630	170525	190393
	C8.1.3	46749	98365	143528	182975	216046	241894
	C8.1.2	47464	43967	39180	-36190	-46152	-54656
	C8.1.1	46749	53767	59323	64512	69233	73351
	C8.1.0	34150	32131	29007	31286	34318	36873
group C2	C8.2.4	35310	470396	816444	1075900	1249290	1337180
	C8.2.3	38062	405325	693587	902988	1035480	1093450
	C8.2.2	37879	43885	47262	-60979	-76759	-89367
	C8.2.1	38062	66949	91569	113503	132191	146908
	C8.2.0	35310	57243	76019	92753	106929	117972
group C3	C8.3.4	32511	83501	168107	253815	340529	427957
	C8.3.3	29947	92108	152305	205768	251349	306089
	C8.3.2	28104	64943	97589	127636	-158452	-189448
	C8.3.1	29947	30539	34968	38631	41360	42936
	C8.3.0	32511	28558	23769	22960	27604	31568

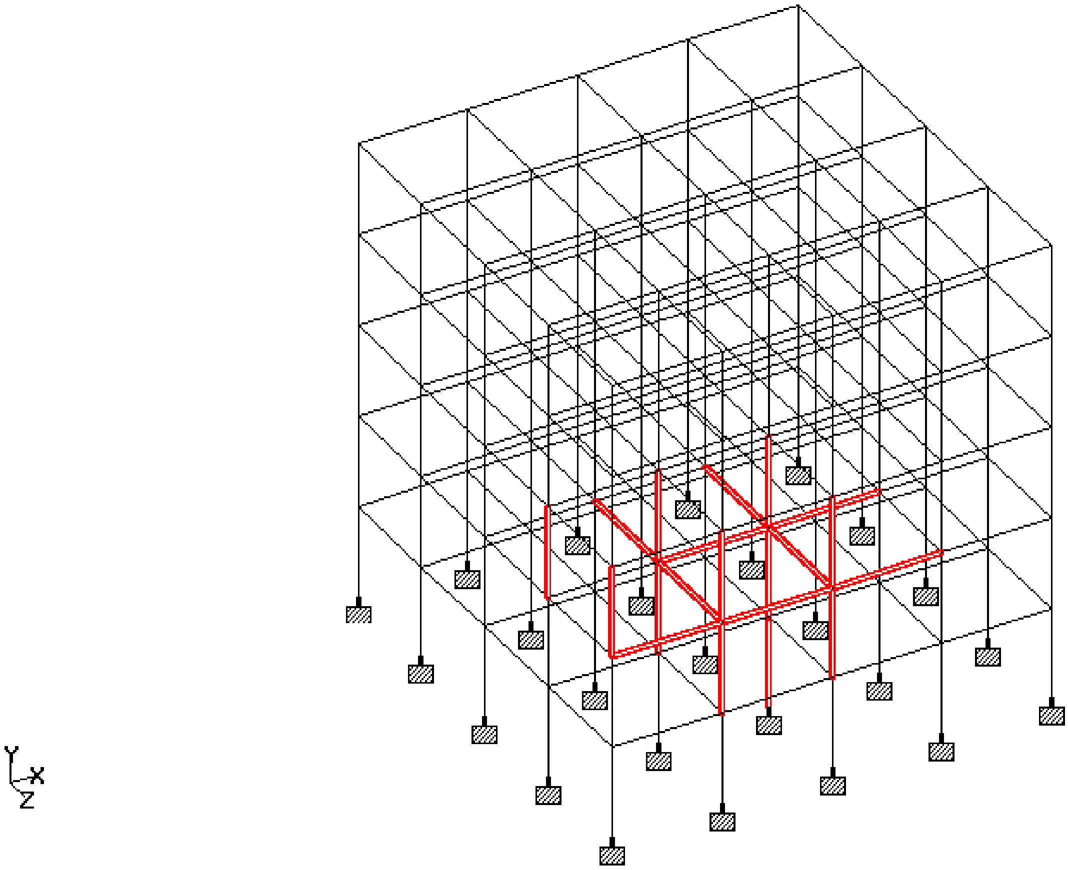

Figure 4.11 20 Failed members in 520°C temperature

4.4 Comparison of Groups in 500°C Temperature

In case the temperature of compartment members are 500°C then starting from group A1 see Figure 4.12 it can be noticed that the beams are deflected in the same pattern generally, but the (base case) a bit more deflected for the beam B4.2.4 which is part of the fire compartment. The beam B4.1.4 was not change in the three cases, but for the beam B4.3.4 it deflected less in the (two stories added).

The stresses in the beam B4.2.4 was the higher in group A1 and exceeded the yield strength of steel obviously especially for the different fire compartment case. See Figure 4.13.

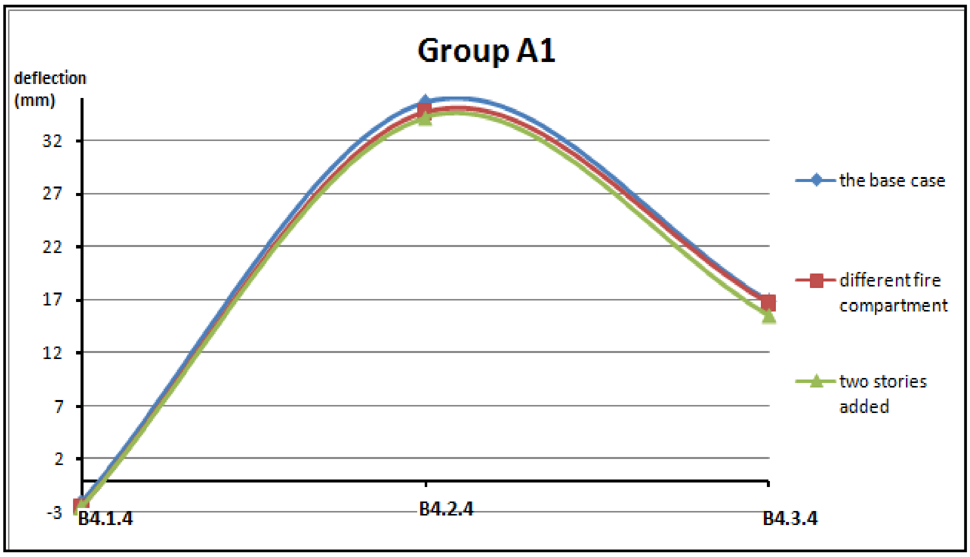

Figure 4.12 Deflection comparison of group A1

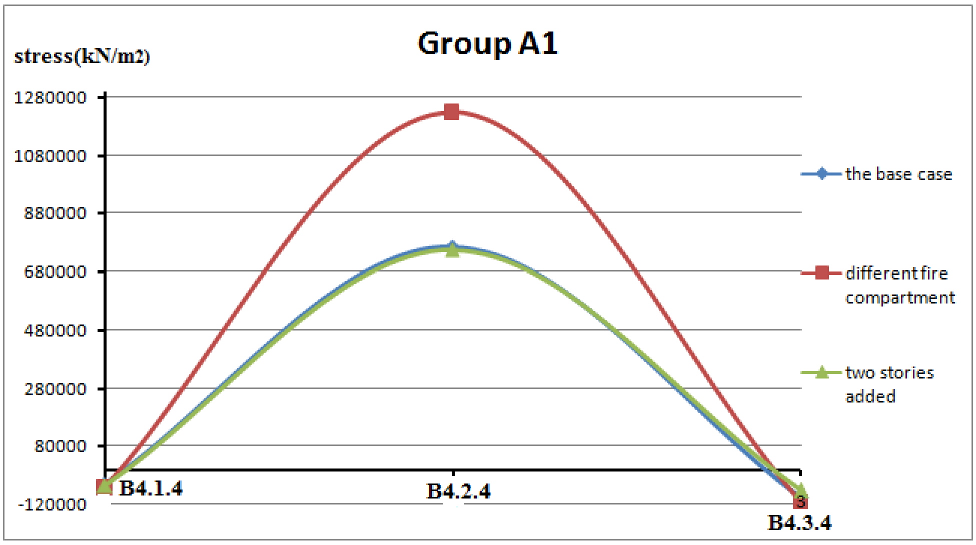

Figure 4.13 Stress comparison of group A1

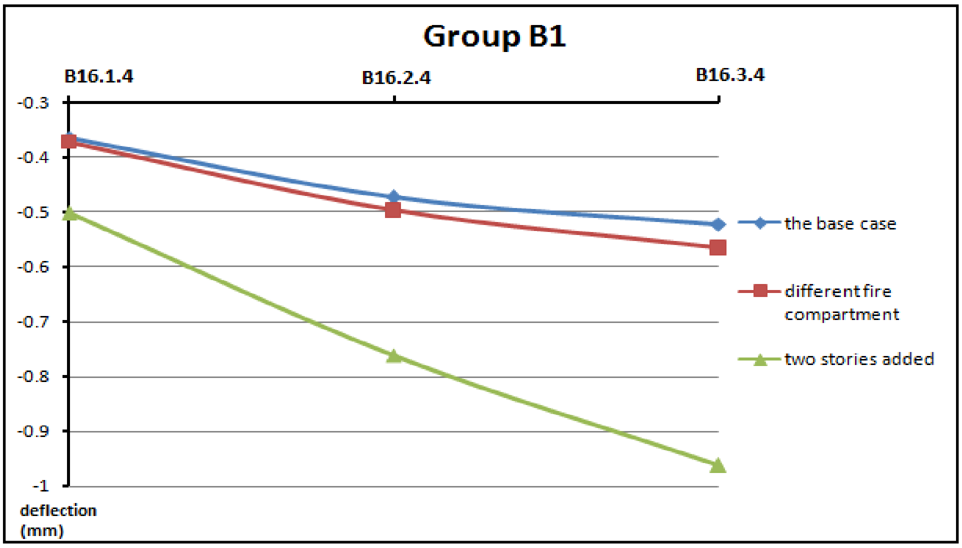

Figure 4.14 Deflection comparison of group B1

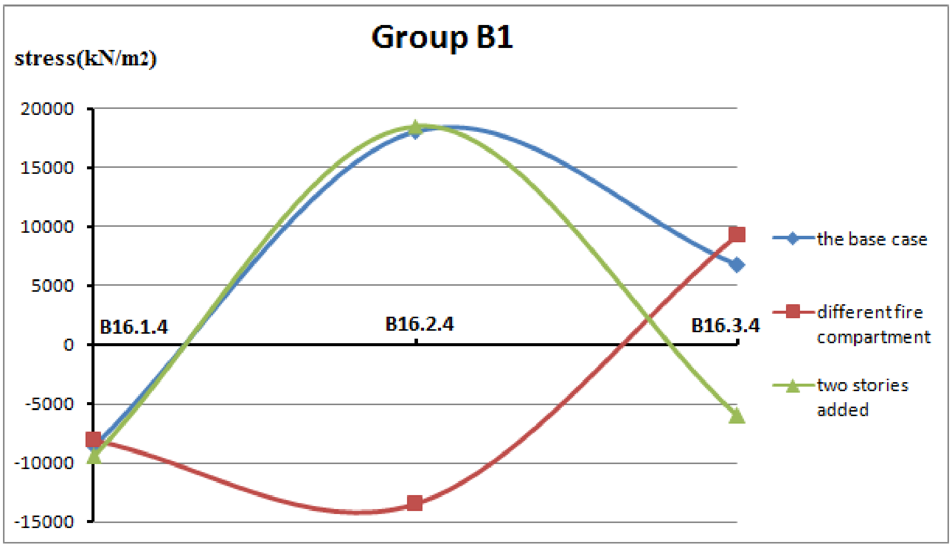

Figure 4.15 Stress comparison of group B1

In the group B1 in Figure 4.14 which is a group far from fire compartment it can be seen that (two stories added) has the larger deflection then come (the different fire compartment) then the (base case). So the deflection increased from the base case after adding two stories to the structure. In the three members of group B1 no member has reached the yield strength of steel so no failed members in this group. See Figure 4.15.

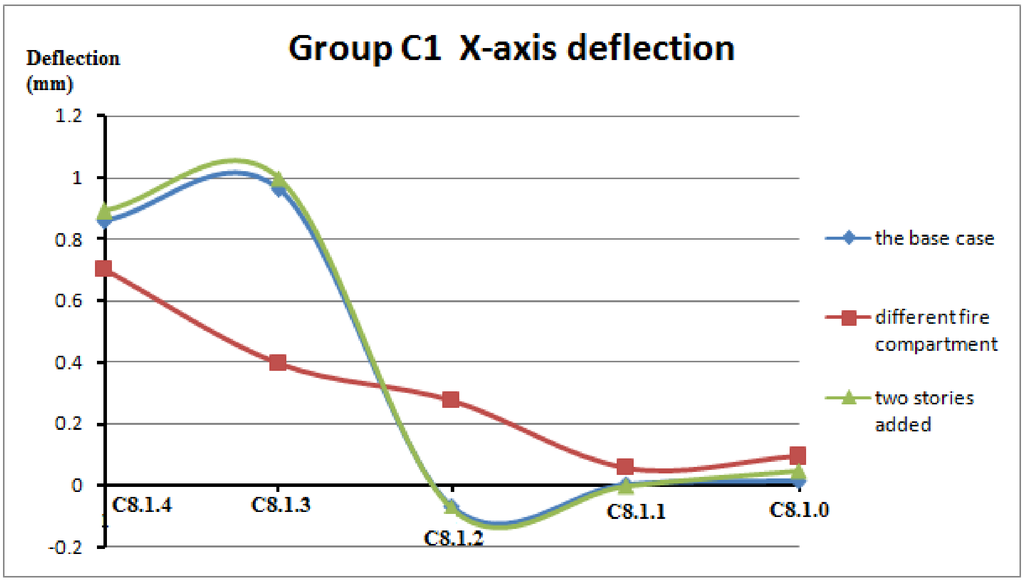

Figure 4.16 Deflection comparison of group C1 in X-axis direction

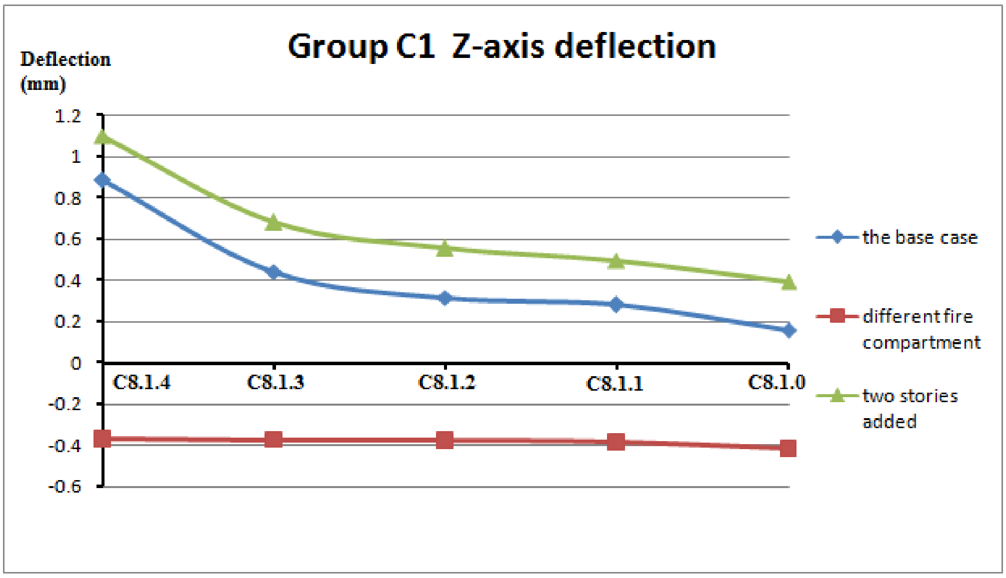

Figure 4.17 Deflection comparison of group C1 in Z-axis direction

For the Figures 4.16 and 4.17 it shows deflection in the group C1 columns in two axis directions one in X-axis direction and one in Z-axis direction. In the both Figures the columns C8.1.4 and C8.1.3 have the larger deflections rationally because these two columns located exactly below the fire compartment columns. As it is noticed from Figure 4.18 the stresses in member C8.1.3 only in the two stories added case has exceeded the yield strength of steel.

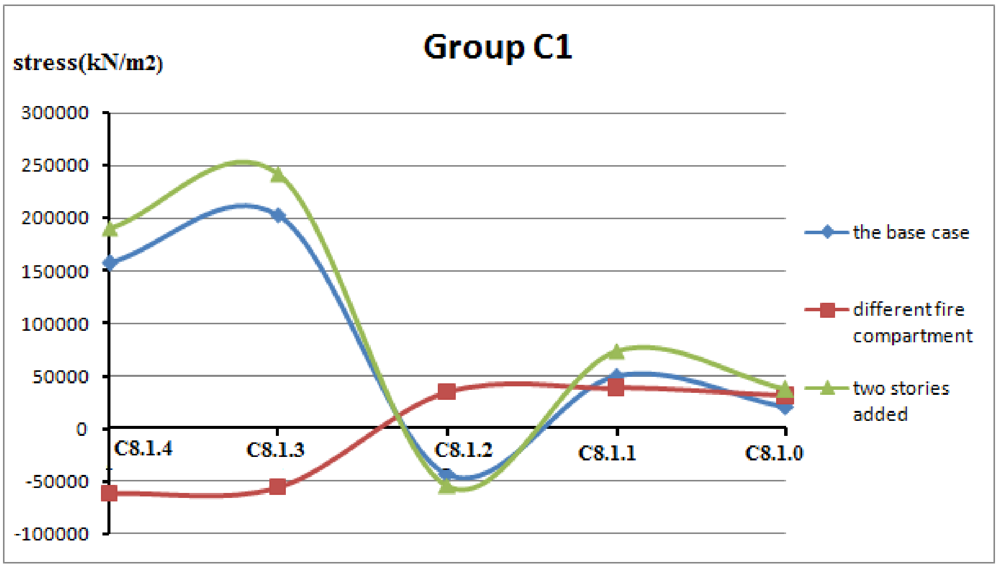

Figure 4.18 Stress comparison of group C1

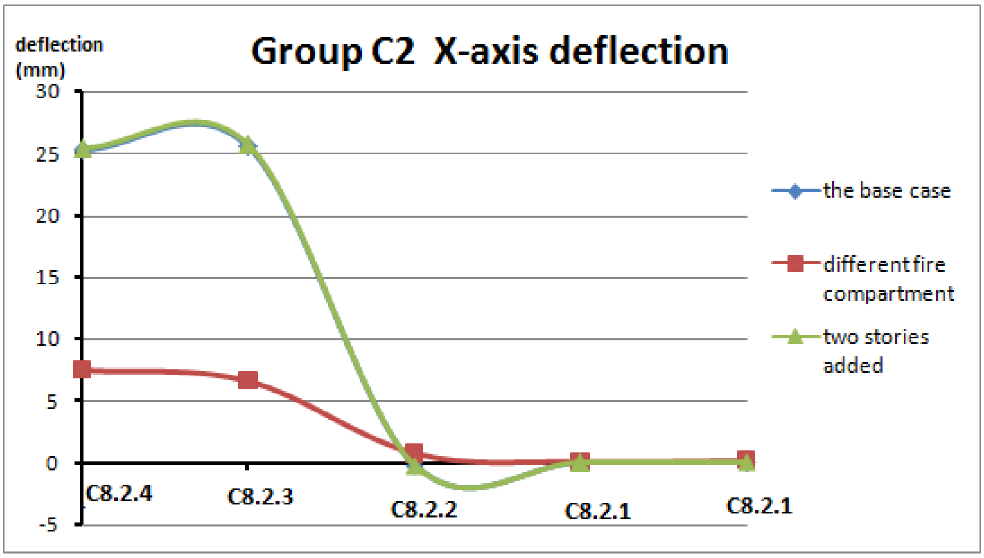

Figure 4.19 Deflection comparison of group C2 in X-axis direction

In the Figures 4.19 and 4.20 it shows deflection for group C2, the columns C8.2.4 and C8.2.3 are part of the fire compartment because of that large deflection happened in these columns especially in the (base case) and (two stories added).

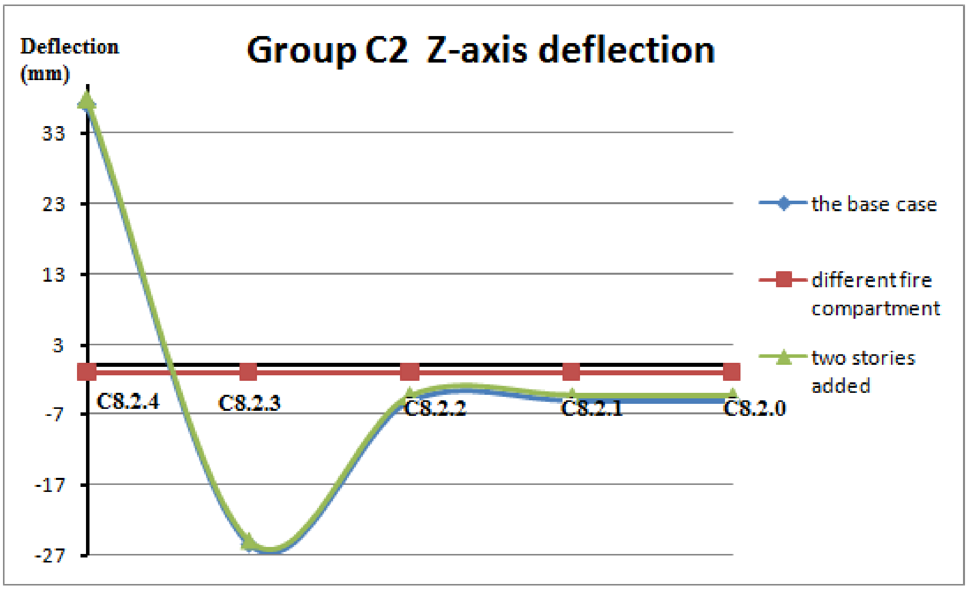

Figure 4.20 Deflection comparison of group C2 in Z-axis direction

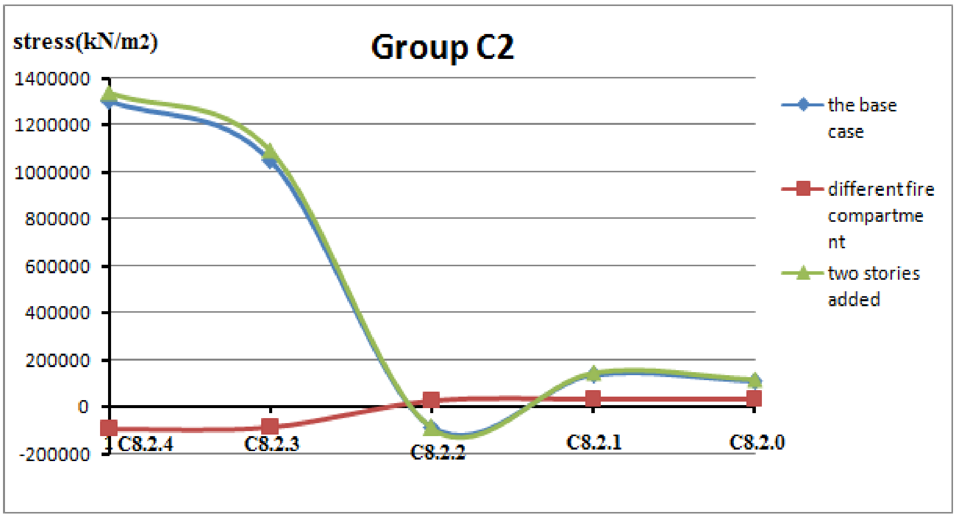

Figure 4.21 Stress comparison of group C2

In (different fire compartment) the deflection was less because when location of fire compartment changed the columns C8.2.4 and C8.2.3 were not remain parts of fire compartment any more. The stresses in group C2 for the different fire compartment case did not reached yield strength in any members. But for the base case and two stories added case only the two members C8.2.4 and C8.2.3 have exceeded yield strength of steel. See Figure 4.21.

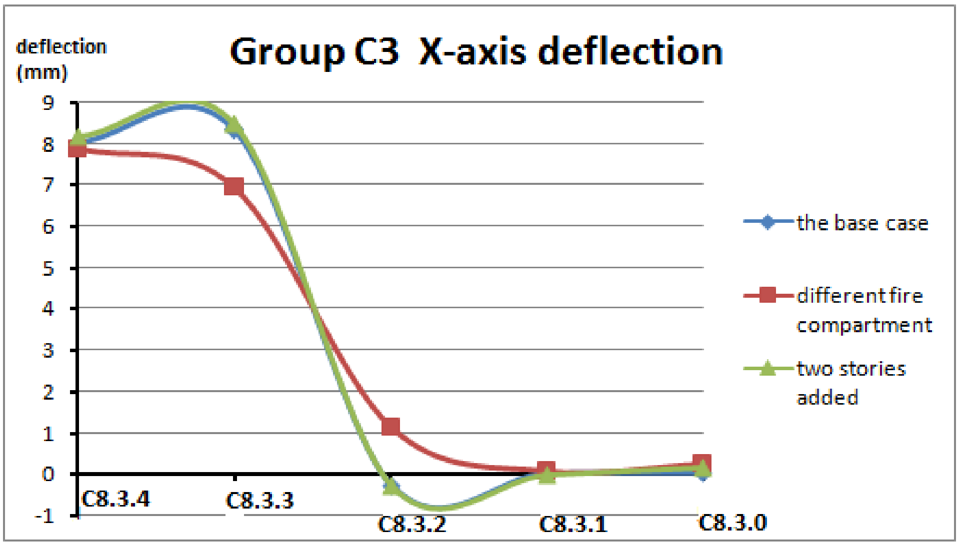

Figure 4.22 Deflection comparison of group C3 in X-axis direction

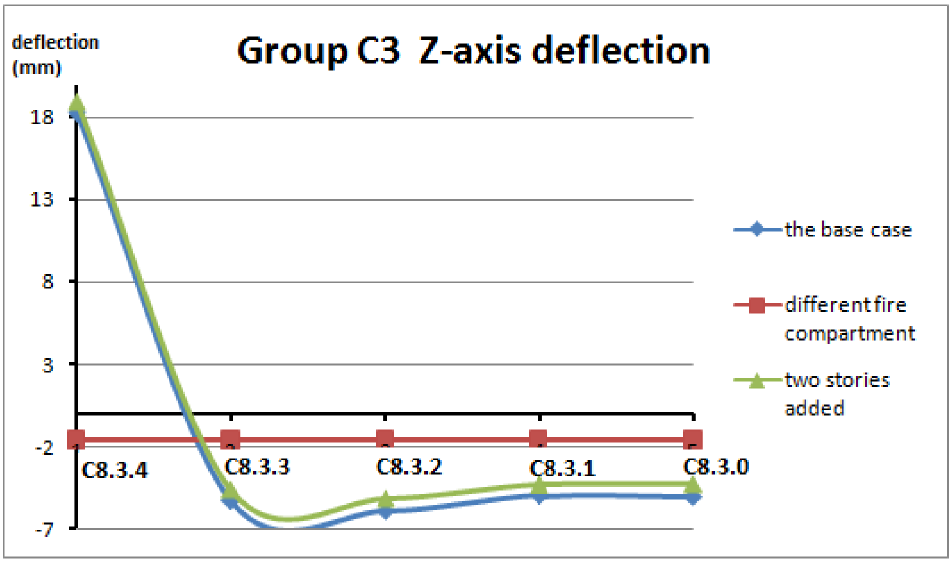

Figure 4.23 Deflection comparison of group C3 in Z-axis direction

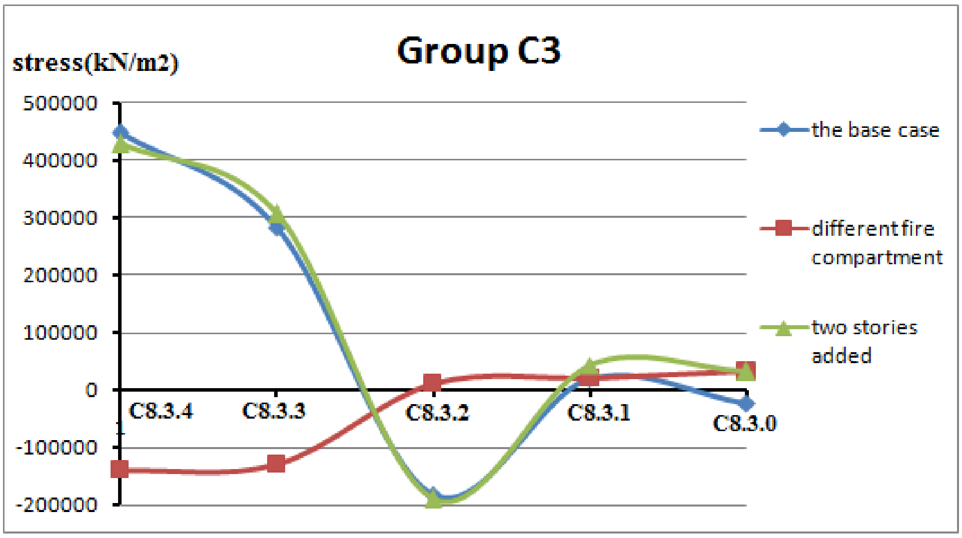

Figure 4.24 Stress comparison of group C3

In the Figures 4.22 and 4.23 the deflections for group C3 which is located in the higher floor, all the three cases have larger deflection in columns C8.3.4 and C8.3.3 because these columns are located above the fire compartment columns exactly.

The stresses in group C3 for the different fire compartment case did not reached yield strength in any members. But for the base case and two stories added case only the two members C8.3.4 and C8.3.3 have exceeded yield strength of steel. See Figure 4.24.

4.5 Single Element and Multi-element Members

In the section 3.2.4 of this paper, it is mentioned about dividing the member into several elements by inserting nodes into the member, then applying the temperature loads to these elements gradually from the highest temperature to the lower temperature along the member.

So for explaining this, two cases are taken:

In the first case no nodes will be added to the fire compartment members (beams &columns) and all of the members remain single element members.

In the second case the beams of the fire compartment are considered as single element members as usual. But the nodes will be added to the columns of the fire compartment and the columns converted from single element to ten-element members as shown in Figure 4.25.

Assume the temperature 520 °C is applied on the members of both two cases in this comparing.

Case 1: the temperature is 520 °C along all the beams and columns lengths in the fire compartment because the members are single element members. See Figure 4.3.

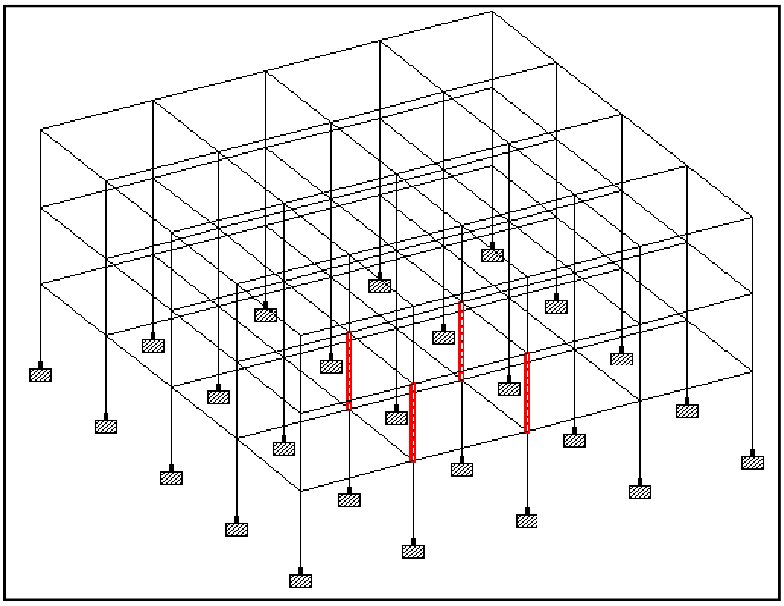

Figure 4.25 Fire compartment composed of columns divided to ten-elements

Case 2: the temperature is 520 °C along all the beams as in the case 1, but for the columns in Figure 4.25 the temperature 520 °C is gradually divided along (assume 10 elements) the elements linearly as described below:

n= total number of elements in a member = 10

i= element number

T_i = cumulative temperature of the element

Ambient temperature = 20 °C

T_{top}-T_{bottom} = 520-20 = 500°C

$\Delta = \frac{T_{top} - T_{bottom}}{n} = \frac{500}{10} = 50$ °C the increasing rate of temperature for the ten elements.

For element one, i = 1 then:

$T_1 = i \times \Delta = 1 \times 50 = 50$ °C

For the other elements in the same way:

For element two, i= 2 then

$T_2 = 2 \times 50 = 100$ °C

Then T_3 = 150 °C, T_4 = 200 °C, T_5 = 250 °C, T_6 = 300 °C, T_7 = 350 °C, T8= 400 °C, T_9 = 450 °C, T_{10} = 500 °C, see Figure 4.26

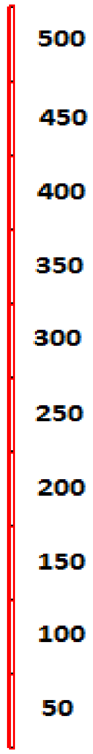

Figure 4.26 500°C Temperature division on the ten elements column (case 2)

The lower part of the column is the coolest part (50°C), the higher part of the column is the hottest part (500°C).

So to see the effects of these two different cases of loading on the other members, a selected group of elements is chosen and is called group D.

The group D as shown in Figure 4.27 is composed of three beam members. The results are shown in Table 4.22 and Table 4.23.

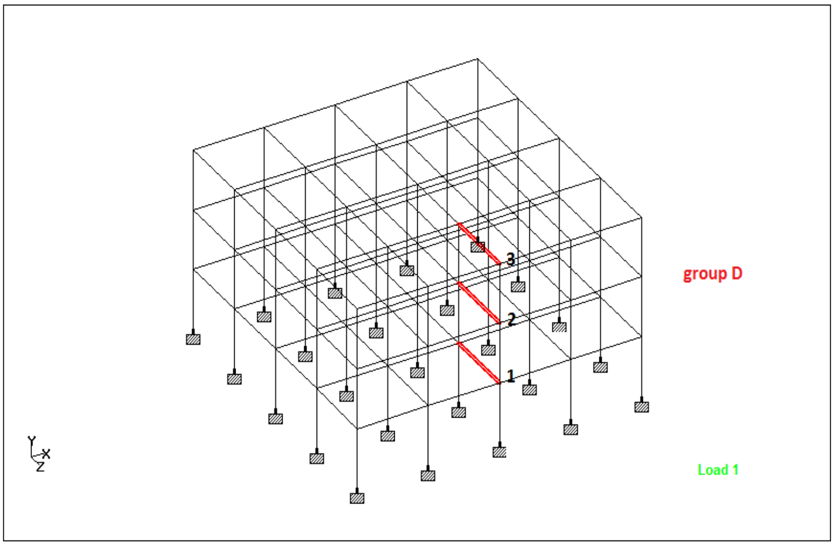

Figure 4.27 Group D members

4.5.1 Discussions of results for single element and multi-element members

As it can be seen from the results of Table 4.22 that the same members exposed to the same temperature, but in the multi element case which is closer to the reality the deflections were smaller than the one element member case. Because in the multi element members there are cooler elements and hotter elements and their average is a moderate temperature. But in the one element members there is only one heat distributed equally to all the elements because of that the deflection is more in the one element members. This means the predictions of fire sustaining of buildings need more accuracy in the techniques and the ways of analyzing their members.

Table 4.22 Deflection results for the group D members

Group D	Deflections at temperature 520°C (mm)	
	Single element members	Multi element members
B8.1.4	-2.05	-0.893
B8.2.4	35.46	27.23
B8.3.4	16.73	8.5

The stress results for group D also give less stresses in the case of multi element members in which in the member B4.2.4 the stresses of single element are much larger than multi element member. See Table 4.23.

Table 4.23 Stress results for the group D members

Group D	Stresses at temperature 520°C (kN/m^2)	
	Single element members	Multi element members
B8.1.4	-49917	-47688
B8.2.4	769197	408740
B8.3.4	-98699	-78344

CHAPTER 5

CONCLUSIONS

5.1 Conclusions

Four example cases are considered, the first one is the base case in which there is fire in a specific compartment and the structure has three floors. In the second one the same base example but the only change is the location of the fire compartment. In the third one the same base example but the only change is two stories added to the base example. In the fourth one the same base example but the columns of the fire compartment are transformed from single element to ten elements, for the first three examples the same group of beams and columns are chosen to see the effects of the temperature loads, only for the fourth example a special group are chosen to see effects.

In general it can be implied from the tables and figures that no great deflection and stresses happened in the ground floor, the larger deflection and stress happened in the fire compartment members and then happened in the compartment above the fire compartment and after them it happened in the compartments besides the fire compartment.

So the higher floors tends to more deflect and develop more stresses than the lower floors during fire, the cause of that may be the higher floor members are more free than the lower floor members in which they are more restrained because their closeness to the ground.

The case 2 of the group D is more similar to the reality because the temperature is distributed gradually in the columns. In the case 1 the members are taken as single element and the temperature is assumed to be distributed along the member equally as one value which is not similar to the reality, and because of that the results of the deflections and stresses for the case 1 may be overestimated.

Maybe after the fire ends, the only part that needs renovation in the building is the compartment that the fire occurred in, or just some of the members around that compartment in the other neighboring compartments, because except for the members of fire compartment large deflections and stresses not detected in the other members generally.

So when a fire occurs in a building compartment, and the compartment collapsed, it doesn’t mean necessarily that all the building is collapsing or the building is reaching failure, because the members behavior in the fire tell us that the structure can maintain the deflections and stresses occurred in part of it by redistributing the forces and moments that the failed members had supported before, so the connected net of members can sustain the defects happen in some members if the number of these deflected and failed members were not that large to reach the critical point of collapse.

5.2 Future Works

In future it needs to consider more parameters for different conditions:

1. The restrain condition of the members is necessary to be considered if the members are fixed how the effects of the fire changes from if the members are pinned and so on.
2. The situation of fire is needed to be considered how the duration of the fire act on the structure’s stability, when the fire gets its maximum intensity in the start of fire or in the middle or at the end of it, the effect of that on the structures.
3. More specific software program to be used to handle the complicated environment of the fire compartment and maintain the temperature parameters.

If the researchers and experts go through and making more steps in this field in the future, they may get more answers of how these members act during fire , in which it will have great benefit in predicting structures’ behavior, strength , time of collapse, and the stability of the structure.

REFERENCES

[1] Meriam, J. L. and Kraige, L.G. (2002). Engineering mechanics –statics. John Wiley & Sons, New York.

[2] Hibbeler, R. C. (2012). Structural analysis. Pearson Prentice Hall,New Jersey.

[3] Gorbett, G. E. and Pharr, J. L. (2011). Fire dynamics.Prentice Hall.

[4] A European joint research programme. (1999). The behavior of multi-storey steel framed buildings in fire. British steel plc, Swinden Technology centre.

[5] PIT Project main report (2000). Behaviour of steel framed structures under fire conditions. The University of Edinburgh, School of Civil & Environmental Engineering.

[6] Lewis K. R. (2000). Fire design of steel members, report, University of Canterbury, Christchurch, New Zealand.

[7] Lien, K.H., Chiou, Y.J., Wang, R.Z. and Hsiao, P.A. (2009). Nonlinear behavior of steel structures considering the cooling phase of a fire, Journal of Constructional Steel Research, **65**, 1776-1786.

[8] Moss P. j. and Clifton, G. C. (2002). Modelling of the cardington LBTF steel frame building fire tests, Fire Mater., **28**, 177–198.

[9] Wang, Y.C. (2000). An analysis of the global structural behaviour of the Cardington steel-framed building during the two BRE fire tests, Engineering Structures, **22**, 401-412.

[10] Dong, Y. and Prasad, K. (2009). Thermal and structural response of a two-story, two bay composite steel frame under fire loading, *Proceedings of the Combustion Institute*, **32**, 2543–2550.

[11] BORST, R.D. and PEETERS, P.P.J.M. (1989). Analysis of concrete structures under thermal loading, *Computer Methods in Applied Mechanics and Engineering*, **77**, 293-310.

[12] Franssen, J.M., Cooke, G.M.E. and Latham, D.J. (1995). Numerical Simulation of a Full Scale Fire Test on a Loaded Steel Framework, *J. Construct. Steel Research,* **35**, 377-408.

[13] Liew, J.Y.R., Tang, L.K., Holmaas, T., Choo, Y.S. (1998). Advanced analysis for the assessment of steel frames in fire, *Journal of Constructional Steel Research*, **47**, 19–45.

[14] Nwosu, D.I. and Kodur, V.K.R. (1999). Behaviour of steel frames under fire conditions, *Can. J. Civ. Eng.*, **26**, 156-167.

[15] Song, L., Izzuddin, B.A., Elnashai, A.S. and Dowling, P.J. (2000). An integrated adaptive environment for fire and explosion analysis of steel frames — Part I: analytical models, *Journal of Constructional Steel Research*, **53**, 63–85.

[16] Elghazouli, A.Y., Izzuddin, B.A. and Richardson, A.J. (2000). Numerical modelling of the structural fire behaviour of composite buildings, *Fire Safety Journal*, **35**, 279-297.

[17] Bailey, C. (2002). Structural fire design of unprotected steel beams supporting composite floor slabs, *II International Conference on Steel Construction – II CICOM,* São Paulo, SP, Brazil.

[18] Becker, R. (2002). Structural behavior of simple steel structures with non-uniform longitudinal temperature distributions under fire conditions, *Fire Safety Journal*, **37**, 495–515.

[19] Clancy, P. and Young, S.A. (2004). Full scale experiments for evaluating theoretical fire wall models, *Fire Mater.*, **28**, 431–458.

[20] Liew, J.Y.R. and Ma, K.Y. (2004). Advanced analysis of 3D steel framework exposed to compartment fire, *Fire Mater.*, **28**, 253-267.

[21] Bernhart, C.I.D. (2004). The Effect of Support Conditions on the Fire Resistance of a Reinforced Concrete Beam, *Fire Engineering Research Report 04/5*, University of Canterbury.

[22] Iu, C.K. and Chan, S.L. (2004). A simulation-based large deflection and inelastic analysis of steel frames under fire, *Journal of Constructional Steel Research*, **60**, 1495–1524.

[23] Lim, L., Buchanan, A., Moss, P. and Franssen, J.M. (2004). Numerical modelling of two-way reinforced concrete slabs in fire, *Engineering Structures*, **26**, 1081–1091.

[24] Bénichou, N. and Sultan, M.A. (2005). Thermal properties of lightweight-framed construction components at elevated temperatures, *Fire Mater.*, **29**, 165–179.

[25] Landesmann, A., Batista, E.D.M. and Alves, J.L.D. (2005). Implementation of advanced analysis method for steel-framed structures under fire conditions, *Fire Safety Journal*, **40**, 339–366.

[26] Iu, C.K., Chan, S.L. and Zha, X.X. (2005). Nonlinear pre-fire and post-fire analysis of steel frames, *Engineering Structures*, **27**, 1689–1702.

[27] Ezekoye, O.A., Lakshminarasimhan, K., Seers, P. and Nicks, R. (2005). Effects of PPV Attack on Thermal Conditions in a Compartment Downstream of a Fire, *Fire Technology*, **41**, 193–208.

[28] König, J. (2005). Structural fire design according to Eurocode 5-design rules and their background, *Fire Mater.*, **29**,147–163.

[29] Kodur, V.K.R. and Sultan, M.A. (2006). Factors Influencing Fire Resistance of Load-bearing Steel Stud Walls, *Fire Technology*, **42**, 5–26.

[30] Franssen, J.M., Pintea, D. and Dotreppe, J.C. (2007). Considering the effects of localised fires in the numerical analysis of a building structure, *Fire Safety Journal*, **42**, 473–481.

[31] Junior, V.S. and Creus, G.J. (2007). Simplified elastoplastic analysis of general frames on fire, *Engineering Structures*, **29**, 511–518.

[32] Hozjan, T., Turk, G., Srpčič, S. (2007). Fire analysis of steel frames with the use of artificial neural networks, *Journal of Constructional Steel Research*, **63**, 1396–1403.

[33] Garlock, M.E.M. and Quiel, S.E. (2007). Mechanics of Wide-flanged Steel Sections that Develop Thermal Gradients Due to Fire Exposure, *Steel Structures*, **7**, 153-162.

[34] Duthinh, D., McGrattan, K. and Khaskia, A. (2008). Recent advances in fire–structure analysis, *Fire Safety Journal*, **43**, 161–167.

[35] Santiago, A., Silva, L.S.D., Real, P.V. and Veljkovic, M. (2008). Numerical study of a steel sub-frame in fire, *Computers and Structures*, **86**, 1619–1632.

[36] Perricone, J., Wang, M. and Quintiere, J. (2008). Scale Modeling of the Transient Thermal Response of Insulated Structural Frames Exposed to Fire, *Fire Technology*, **44**, 113–136.

[37] Crosti, C.(2009). Structural Analysis of Steel Structures under Fire Loading, *Acta Polytechnica*, **49**, No.1.

[38] Quiel, S.E. and Garlock, M.E.M.(2010). Closed-Form Prediction of the Thermal and Structural Response of a Perimeter Column in a Fire, *The Open Construction and Building Technology Journal*, **4**, 64-78.

[39] Thomas, G. (2010). Modelling thermal performance of gypsum plasterboard-lined light timber frame walls using SAFIR and TASEF, *Fire Mater.*, **34**, 385–406.

[40] Barowy, A.M. (2010). Heat and Smoke Transport in a Residential-Scale Live Fire Training Facility: Experiments and Modeling, *M.Sc. Thesis*, Worcester Polytechnic Institute.

[41] Moss, P., Buchanan, A., Fragiacomo, M. and Austruy, C. (2010). Experimental Testing and Analytical Prediction of the Behaviour of Timber Bolted Connections Subjected to Fire, *Fire Technology*, **46**, 129–148.

[42] Law, A., Gottfried, J.S., Gillie, M. and Rein, G. (2011). The influence of travelling fires on a concrete frame, *Engineering Structures*, **33**, 1635–1642.

[43] Fang, C., Izzuddin, B.A., Elghazouli, A.Y. and Nethercot, D.A. (2011). Robustness of steel-composite building structures subject to localised fire, *Fire Safety Journal*, **46**, 348–363.

[44] Collette, K. A. (2007). Comparisons of Structural Designs in Fire, M.Sc. Thesis, Worcester Polytechnic Institute.

[45] BSI (2001). Eurocode 3: Design of steel structures Part 1.2: General rules — Structural fire design.

[46] Lamont S. (2000). PIT project research report:TM3, Study of thermal expansion and bowing in a restrained beam. The University of Edinburgh, School of Civil & Environmental Engineering.

[47] Lamont S. (2002). The behavior of multi-storey composite steel framed structures in response to compartment fires, *Ph.D Thesis*, The University of Edinburgh.

[48] Yang, X. J. and Gao, R.Factors affecting the behavior of steel structures in fire. College of Civil & Architecture Engineering, Beijing Jiaotong University, Beijing, 100044, China

[49] Kallerová, P. and Wald, F. (2007). Mechanical properties of steel at elevated temperatures. Czech Technical University in Prague.

[50] Usmani, A.S., Rotter, J.M., Lamont, S., Sanad, A.M. and Gillie M. (2001).Fundamental principles of structural behaviour under thermal effects, Fire Safety Journal, **36**, 721–744.

[51] Al-Jabri, K. S. (2007). Behavior of Steel-framed Buildings in a Fire, Steel Structures, **7**, 227-237.

[52] Hoang, H. (2010). Structural Continuity Effects in Steel Frames under Fire Conditions, *M.Sc. Thesis*, Worcester Polytechnic Institute.

[53] Karlsson, B. and Quintiere, J. G. (2000). Enclosure Fire Dynamics. CRC Press LLC, Florida.

[54] Research Engineers International, Bentley Solutions Center. (2008). STAAD Pro 2007: Technical Reference Manual. Bentley Systems, Incorporated.