# EFFECT OF TEMPERATURE ON THERMAL AND MECHANICAL PROPERTIES OF HIGH STRENGTH STEEL A325 AND A490 BOLTS

By

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**ABSTRACT**

EFFECT OF TEMPERATURE ON THERMAL AND MECHANICAL PROPERTIES OF HIGH STRENGTH A325 AND A490 BOLTS

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Bolted connection is a very popular connection type as it is the most economical, versatile, and reliable method of connection. Although, these bolted connections are effectively designed to withstand various loads under ambient conditions, there is a lack of specific methodology for fire design. Limited information is available on high temperature properties of steel bolts. The heat treatment and metallurgical composition of high strength steel used in bolts governs the material properties of these bolts. Thus an experimental study is carried out on high strength steel bolts to evaluate high temperature thermal and mechanical properties.

Thermal conductivity and specific heat tests in 20 to 735°C temperature range and thermal expansion tests on 20 to 1000°C range were conducted on A36, A325 and A490 steel in heating as well as cooling phase. Result from these tests indicates that chemical composition and phase transition that occurs in steel has significant influence on its thermal properties.

For mechanical property characterization, high temperature steady state single shear and tension tests were carried out on A325 and A490 bolts in the temperature range of 20 to 800°C. Data from these tests were utilized to deduce high temperature stress–strain curve, strength and stiffness reduction factors. Test results indicate that the strength and stiffness properties of A325 and A490 bolts are more temperature sensitive than conventional steel. Also A490 bolts maintain higher strength and stiffness properties than A325 bolts throughout 20 to 700°C temperature range. Data from property tests was utilized to develop high temperature relations for thermal and mechanical properties of A325 and A490 steel in 20 – 800°C temperature range.

## Dedication

I dedicate this thesis to my grace of soul of my father. I also would like to dedicate my work to my family Lata Kand, Monali Kand, Prasad Kand and Abhishek Gawasane for their love, support and encouragement through out my education.

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## Introduction

### 1.1 General

Steel as a construction material is widely used in high-rise buildings due to numerous advantages, such as high ductility, high stiffness, strength to weight ratio, speed and ease of construction, it offers over other construction materials. In buildings fire is one of the most severe hazards and the temperatures in these fires can reach as high as 1000$^{o}$C. At these temperatures steel softens and loses much of its strength and stiffness properties. Therefore steel structural members are to be designed for appropriate fire safety measures.

Research from previous fire incidents and fire tests have shown that unprotected steel structural member fails in about 20 minutes under severe fire conditions. Building codes stipulate a minimum period, ranging from 30 minutes to 4 hours, during which a structural member should continue to have load-bearing function under fire. This requirement, referred to as fire resistance, is to ensure safe evacuation of occupants and firefighters, limit the spread of fire, and minimize property damage due to fire exposure. Since unprotected steel structural members lose much of their strength and stiffness very quickly, fire insulation is often applied to these steel structural members to meet the fire resistance requirements specified in building codes.

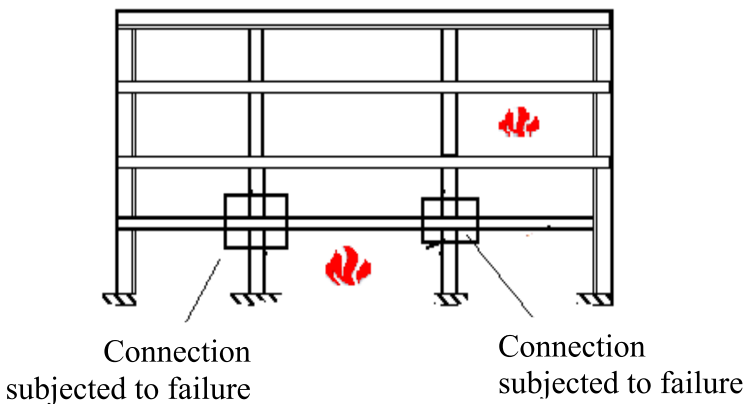
Fire safety can be achieved through different strategies, such as active or passive fire protection systems. Active fire protection systems include provision of sprinklers, fire doors, smoke detection and fire alarm systems. Passive fire protection can be achieved, by applying proper fire insulation to structural members such as beams, columns and connections. Fire insulation improves fire resistance of structural members by limiting temperature rise to a minimum so as to limit the reduction in strength and stiffness. Typical insulation material includes gypsum wallboard, plaster, concrete filling, spray applied material and intumescent coating. Spray applied insulation materials are popular due to their ease of application and good adhesion characteristics.

Beams and columns are primary load carrying members in a steel framed structure. To facilitate load transfer between these members, they are connected together by structural connections. Structural connections can be classified as welded and bolted connections. Bolted connections are economical, versatile and are relied for their fast and easy erection, and hence are preferred over welded connections. Strength of these bolted connection often govern the stability of the structural members.

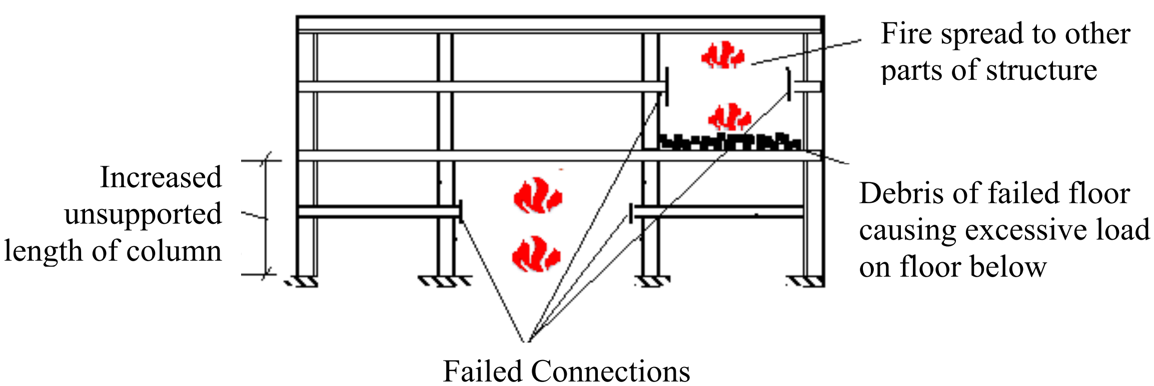
Connections (also referred to as joints) are an integral part of the structural framing system. Failing of connections can lead to failure of connected structural members as shown in Figure 1.1 even though these connected members (beams, columns) may have considerable strength and stiffness to carry the loads. Also connections facilitate load transfer from highly stressed region to less stressed region, which significantly enhances survival time of the structure. In most of the previous fire incidents where partial or full collapse of a steel structure occurred, the failure was initiated at the connections. As connections play an important role in maintaining integrity and stability of the structure, fire design of these connections to withstand loads under fire conditions is necessary to improve global performance of the structure.

For fire design of connections, knowledge of high temperature properties of bolts is necessary. Recent development in performance based fire design has increased the use of computer-based modeling to simulate fire response of structural components and systems. These computer-based models require high temperature properties of materials including bolts, to evaluate fire response.

The temperature dependent properties of bolts that are critical for modeling behavior of connections include thermal properties such as thermal conductivity, specific heat, and thermal expansion, and mechanical properties such as strength and stiffness. Currently there is lack of information on these properties of bolts at elevated temperatures. Hence an extensive experimental study is proposed to determine high temperature thermal and mechanical properties of bolts.




(a) Typical steel structural frame subjected to fire




(b) Influence of failure of connection on other components of a structure

Figure 1.1 – Influence of connection failure on global performance of a steel-framed structure (For interpretation of the references to color in this and all other figures, the reader is referred to the electronic version of this thesis.)

## 1.2 Bolted connections in steel frames

Bolted connections can be classified as simple connections and eccentric connections. Based on the load applied on the bolted connection these connections can further be classified as simple connections that includes tension member connection, beam end simple connection, and hanger connection and eccentric connections that includes eccentricity in the plane of the faying surface, and eccentricity normal to the plane of the faying surface.

These bolted connections compose of a bolt, a nut and a washer. Steel bolts can be classified as common bolts and high strength bolts. Grade A307 bolts are common, low carbon steel bolts produced as per ASTM A307 standard specification with minimum tensile strength of 415MPa. Grade A325 and A490 bolts are high strength, medium carbon steel bolts used in structural steel connections. These bolts are manufactured as per requirements of ASTM A325/A325M-04a and ASTM A490/A490M-04b specifications, with minimum tensile strength of 830MPa and 1030MPa respectively. These high strength steel bolts are widely used for structural steel construction due to its strength advantages and efficiency over common bolts.

## 1.3 Behavior of bolted connection under fire

At ambient temperature, bolted connections are generally subjected to shear force and flexural tension or compression force depending on the type of the connection. These forces are generated in the connection through the load transfer that occurs from connected members. To withstand these forces, design manuals provide relevant equations to design bolted connections at ambient temperature.

When fire occurs in steel framed building, structural members not only lose their strength and stiffness, but also undergo significant expansion. Such expansion leads to development of fire-induced forces in the structural members and connections. This phenomenon of development of fire-induced force is called catenary action of the beam as shown in Figure 1.2. During initial stage of fire, due to axial restraint of thermal expansion, compressive forces develop in the connections. In later stage of fire, as temperature increases the beam undergoes extensive sagging and during cooling phase due to thermal shrinkage of the beam, significant tensile forces develop in the connections. The magnitude of these fire-induced forces depends on thermal expansion of the steel in heating and cooling phase, and axial restraint of beam.

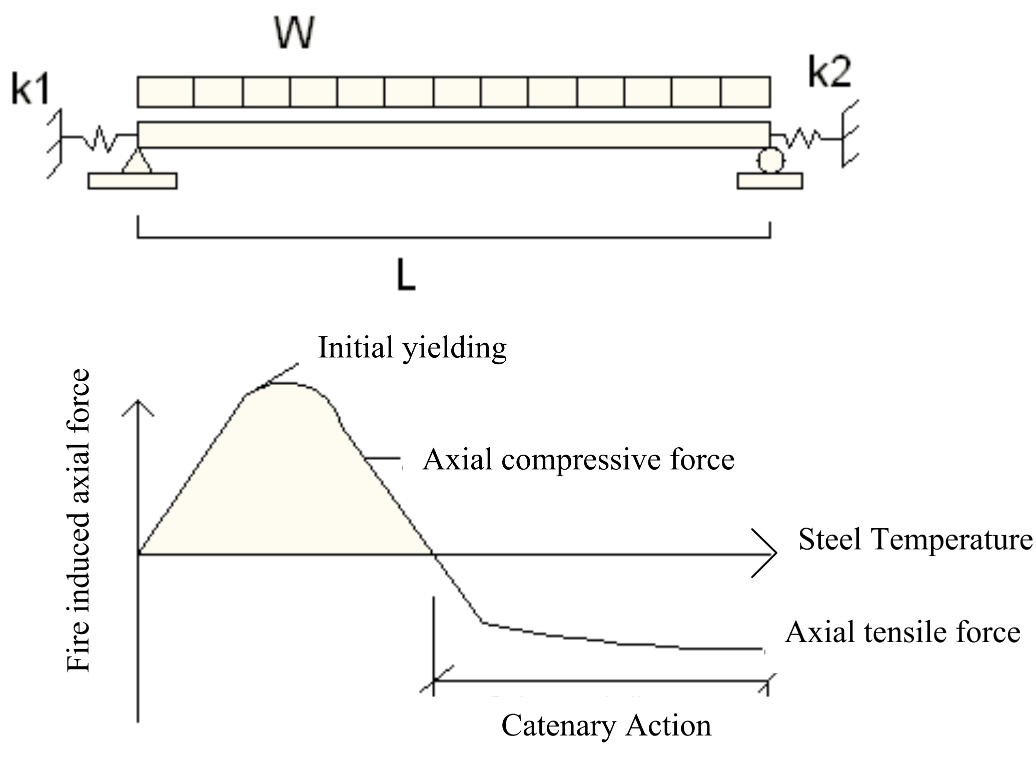



Figure 1.2 – Development of a catenary action in typical fire response of a restrained steel beam that fails by yielding

As performance of bolted connections exposed to fire depends on the development of fire-induced forces due to catenary action of beams, designing bolted connections for sufficient strength and stiffness to withstand these unforeseen loads becomes necessary. Strength and stiffness properties of bolts determine the strength of connection. As these properties of bolts are temperature dependant properties; accurate assessment of temperatures in bolts of a connection is essential. The properties that influence temperatures in steel structural member include thermal conductivity, specific heat and thermal expansion. Hence to accurately analyze and design bolted connections to withstand loads under a fire condition, it becomes necessary to study high temperature thermal and mechanicalproperties of the bolts.

### 1.4 High temperature properties of bolts

A325 and A490 bolts are manufactured by quenching and tempering with a designed chemical composition to attain minimum ultimate strength of 830MPa and 1030MPa respectively at ambient temperature. Similar to other types of steel, strength and stiffness properties of bolts decrease with increase in temperature. Althoughthese properties at ambient temperature are well established, very little is known about the high temperature properties of bolts. The British code for instance assumes the strength of bolts as 80% of strength of the conventional structural steel at a particular temperature. Eurocode provides constitutive relations for determining properties of bolts at elevated temperature. However these relations are based on very limited experimental data. Kirbey (1995), Yu (2006) and Gonzalez (2009) performed experimental studies to evaluate high temperature properties of high strength steel bolts. These studies clearly indicate that the heat-treated bolts are far more temperature sensitive. No specific study has been carried out to determine high temperature thermal properties of high strength steel bolts. Thermal properties of high strength steel are often assumed to be same as conventional steel. Furthermore, no data is available in literature or codes of practice on thermal properties of steel in cooling phase of fire and these properties are often assumed similar to that of heating phase, which may lead to unrealistic analysis and design of structural members. To summarize, very limited experimental work is done to evaluate high temperature properties of bolts. Hence to fill this knowledge gap, a series of high temperature thermal and mechanical property tests was planned on A325 and A490 bolts, as a part of this study.

### 1.5 Objective

From above discussion it is clear that there is lack of information on high temperature properties of steel bolts. At present, high temperature properties of bolts arededuced based on very limited experimental data or assumed to be same as that of conventional steel properties. Also, most of the current information on high temperature material properties of structural steel is suitable for the heating phase of fire. To address these knowledge gaps, a comprehensive experimental study in undertaken on A325 and A490 bolts with the ultimate objective of developing relations for high temperature thermal and mechanical properties. The specific objectives of the research are:

- Conduct a detailed state-of-the art review on the high temperature properties of structural steel bolts. The comprehensive review will cover study of bolted connection failure in the past fire incidents, high temperature properties of bolts and current provisions in codes and standards.
- Conduct tests on A325 and A490 bolts to determine high temperature thermal properties, whichinclude thermal conductivity, specific heat and thermal expansion under both heating and cooling phase.
- Conduct tests on A325 and A490 bolts to determine high temperature mechanical properties,which include shear strength, tensile strength, and elastic modulus.

- Compare high temperature properties of high strength bolt steel with those of conventional structural steel.
- Utilize data generated from experiments to develophigh temperature relations for thermal and mechanical propertiesof A325 and A490 bolts.

### 1.6 Thesis Outline

The research, undertaken to address the above objectives, is presented in five Chapters. Chapter 1 gives a general background to the fire response of steel connections and high temperatureproperties of bolts that influence performance of connection under fire conditions. Also the objective of the study isdescribed.

Chapter 2 provides a state-of-the-art review on the high temperature properties of steel bolts. A summary of previous studies and provisions in current codes of practice on high temperature properties of bolts is presented in this chapter. Chapter 3 presents thermal property tests and results on A36, A325 and A490 steel. Data from these tests was utilized to develop relationships for thermal conductivity, specific heat and thermal expansion as a function of temperature. Chapter 4 presents a description of shear and tension tests together with discussion on the effect of temperature on strength properties of bolts. The test data was utilized to develop high temperature strength and stiffness reduction factors and stress-strain curves for A325 and A490 bolts. Chapter 5 summarizes the main findings arising from the current study and lays out recommendations for further research.

# Chapter 2

## Literature review

### 2.1 General

Significant research and development activities in the field of metallurgy of steel has produced new types of steel. By increasing the carbon content of steel by a nominal amount, high strength properties can be achieved in steel. Use of high percent carbon steel originated in 300 B. C. in subcontinents of India and has made marvels in construction industry since then. A325 and A490 bolts, which are commonly used in the bolts, are made from this high strength steel.

High temperature properties of steel govern the performance of bolted connections exposed to fire. Current method of fire resistance evaluation of connections utilize ambient temperature properties of steel without any consideration to the variations in high temperature properties of high strength steel. As conventional steel and high strength bolt steel have different chemical composition, and microstructure, their strength properties are considerably different at ambient temperature, so can be at high temperatures. Thus high temperature properties of conventional steel may not be applicable for bolts. Limited experimental studies carried out on bolt steel over the last decade. These studies emphasize the need for further research to understand the behavior of bolt steel at elevated temperature.

This chapter provides a state-of-the-art review and previous experimental and analytical studies on steel bolts at elevated temperature. The high temperature properties of steel bolts as provided by codes and standards are also reviewed.

## 2.2 Bolted connection failure in past fire incidents

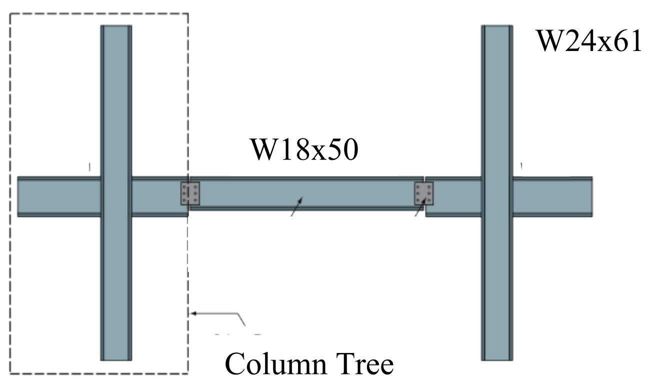
A review of fire incidences indicated that in six major fire incidents, partial or full collapse of steel framed buildings occurred (Beitel and Iwankiw, 2005). Out of these six severe fire events in steel structures, two of the buildings namely The World Trade Center 5 (2001) and One New York Plaza (1970), suffered full or partial collapse due to failure of connections. The following section briefly describes the fire-induced damage in these buildings.

### 2.2.1 World Trade Center 5

On September 11, 2001, two hijacked jets were deliberately flown into the World Trade Center (WTC) – Towers 1 and 2. Impact of the jets resulted in the break out of fire in both the buildings, ultimately resulting in their collapse. The debris from WTC 2, some of which were on fire, rained down on WTC 5. This caused a fire in WTC 5, eventually resulting into partial collapse of a number of floor systems and structural members. WTC 5, a nine storied, structural steel frame office building suffered severe structural damages due to failure of bolted connections. The roof and the $9^{th}$ floor of the building were constructed with a regular beam column connection, whereas the $8^{th}$ floor and the floors below were connected with a column tree connection (Figure 2.1(b)). 3/4" diameter high strength steel bolts were used in these bolted connections. The structural members were provided with spray applied mineral fiber insulation and sprinkler were installed in the building to enhance fire performance. The columns and floor assembly (slab and floor beams) of WTC 5 had fire resistance rating of 3 hours and 2 hours respectively. Exterior non-load bearing walls did not appear to have any fire resistance.

The investigation report concluded that the collapse of floor systems originated at the column tree connections (FEMA 403). After the collapse of the 5th through 8th floors, the columns were observed to be standing straight (Figure 2.1(d)), but the shop fabricated beam stubs and column assembly was damaged at the connections (FEMA 403). This failure of connection was attributed to the fire weekend steel, unanticipated tensile force generated due to the catenary sagging of the beams, and excessive shear force from the debris of the collapsed floors. The performance analysis of damaged connections showed that due to degradation of steel at elevated temperature, the strength of the connection at 550°C was only 200 kN (50%) of that of room temperature capacity. In addition, shear force and catenary tension force together, produced a diagonal tension mechanism in the beam web. The diagonal tension transferred to connections from the connected beam and the reduced strength of the steel resulted in the collapse of 5th through 8th floors of WTC 5. Lessons learned from this incident emphasizes the need to understand behavior of bolted connection under fire-induced forces.

(a) WTC 5 on fire




(b) Typical beam column section and column tree connection

Figure 2.1 – Illustration of failure of WTC5 column tree connections


(c) Failed column tree connections and after fire structural damage

(d) Failed column tree connection and floor assembly without significant damage to column

Figure 2.1 (cont'd)– Illustration of failure of WTC5 column tree connections

Figure 2.2 – Details of failed connection due to web tear out in 6th, 7th and 8th floors of WTC5

### 2.2.2 One New York Plaza

Another building that collapsed as a result of failure of bolted connection was the One New York Plaza building. In this 50 storied steel framed structure, the beams and columns were protected with spray applied asbestos fire resistive material (SFRM) but no sprinkler system. On August 5$^{th}$, 1970 a fire broke on the 33$^{rd}$ floor of the building. This fire lasted for 5 hours resulting in local damage that was restricted to 33$^{rd}$ and 34$^{th}$ floors. Some of the filler beams on these floors were observed to have dropped onto their supporting girders due to failure of the connections. Post fire investigations revealed that the fire insulation on the connection was locally damaged leaving some areas of connection directly exposed to fire. Therefore these isolated unprotected connections experienced very high temperatures and also were subjected to excessive fire-induced tensile force generated from twisting and expansion of the connected beams. The failure of connection observed in this building was similar to the connection failure in the World Trade Center Tower 5 fire and the Alexis Nihon Plaza fire (Beitel and Iwankiw [2005]).

Above studied fire incidents clearly indicate the role of bolted connection on the fire performance of steel framed buildings.

## 2.3 Types of steel

Modern steel is produced from iron and various alloy metals through different heat treatments. Thus by controlling these two factors, variety of steel can be produced and material properties of these steels depends on chemical composition, microstructure and heat treatment. The following section describes these factors and different types of steel used as a construction material.

### 2.3.1 Factors influencing properties of steel

#### 2.3.1.1 Chemical composition of steel

Pure iron is a weak, soft and ductile material and in its purest form it is not fully suitable for engineering use. To improve mechanical and physical properties of this pure iron, it is mixed with suitable alloys to form steel, a commonly used construction material. Alloys added to iron include carbon, manganese, aluminum, nickel, chromium, molybdenum, silica, etc. Addition of these alloy elements improves strength, ductility, hardness, corrosion resistance, machinability, and other properties of steel. Mechanical and physical properties of steel vary with the amount of these alloying elements. Carbon content of steel has the most effect on properties of steel. The tensile strength of steel with carbon content of 1.4% is approximately twice as that of steel with 0.4% carbon content (Adebiyi et.al., 2008). Hence high strength capacity can be obtained by increasing carbon content of steel; however it also increases brittleness. Thus the optimal carbon content of the steel must be limited to control the brittleness. High temperature performance of steel can be improved by providing molybdenum and chromium. These alloys improve hardenebility, strength and resistance to fracture of steel at elevated temperature.

#### 2.3.1.2 Heat treatment of steel

Steel, which is an iron alloyed with carbon and a few other alloying elements, can be heat treated to achieve a wide range of strength, toughness and ductility. Most of the heat treatments of steel are based primarily on controlling the distribution of carbon and formation of a desired microstructure such as pearlite, baintite or martensite.

Commonly used heat treatment processes are:

- Annealing

- Normalizing
- Quenching and tempering
- Special heat treatments such as surface hardening, hardening and tempering of tool steels, martempering, and austempering

Figure 2.2 illustrates crystalline structure of steel for various carbon contents and temperatures. Each of above method produces different crystalline structure and hence physical and mechanical properties of steel vary with the heat treatment type. A typical heat treatment of steel starts with heating steel to the austenitization temperature where $\alpha$ ferrite (body centered cubic structure) phase transforms to the $\gamma$ austenite (face centered cubic structure) and all carbides are dissolved in the austenite. This austenite is cooled down to room temperature either slowly or rapidly according to any of the above stated heat treatments to produce desired microstructure and different strength properties in steel.

Annealing typically produces low carbon steels generally used for conventional structural steel. In annealing and normalizing steel is heated to austenite and carbon. This austenite is cooled down slowly. At this stage the austenite cools down to form ferrite, which has no ability to dissolve carbon, thus carbon precipitates to form cementite. Thus pearlite is formed which is a microstructure made of alternate lamellae of ferrite and cementite. Figure 2.3 illustrates the effect of carbon content and microstructure on the hardness of steel. This is comparatively a coarse grained crystalline structure also the effect of carbon on strength is comparatively less due to its precipitation and hence only limited strength properties can be achieved in these types of steels. On the other, rapid cooling of austenite results in formation of martensite. Maternsite a fine grain, feathery, unstable crystalline microstructure. This martensite is the hardest form of steel crystalline structure, hence imparts high strength properties on steel. Heat treatment such as quenching and tempering is designed to produce martensite microstructure in steel. Hence this heat treatment produces high strength steel that is commonly used for manufacturing high strength steel used for making A325 and A490 bolts.

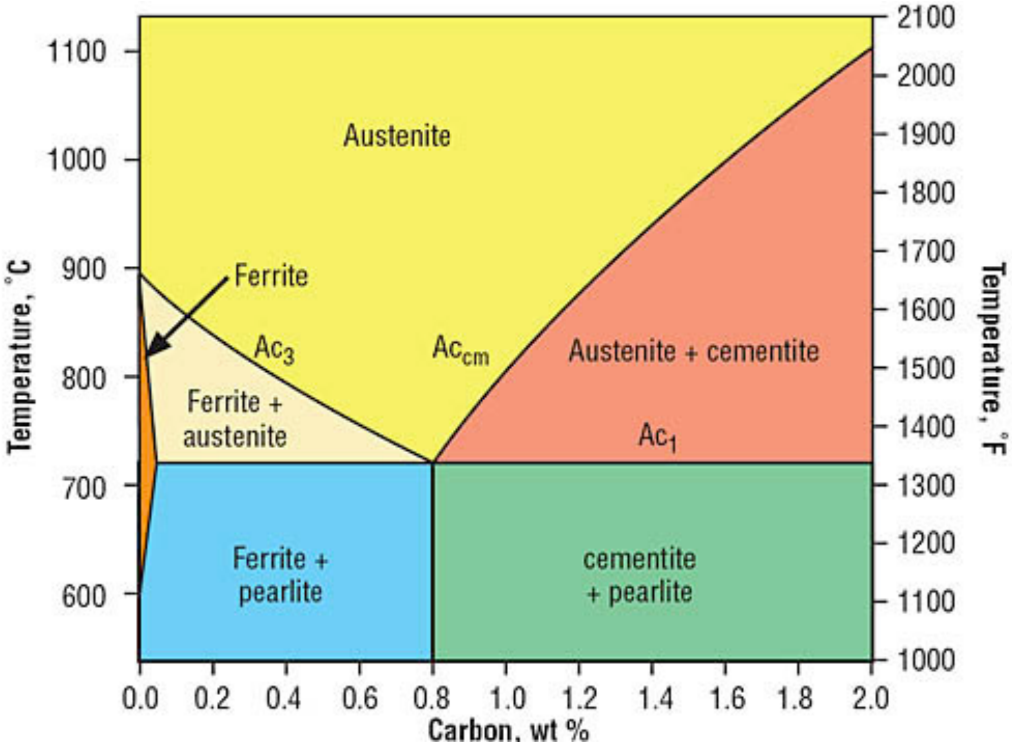



Figure 2.3 – Iron-carbon phase diagram

**2.3.1.3 Type of steel**

The types of steel used in construction can be broadly classified as normal strength and high strength steel.

**Conventional structural steel**

Three basic types of steel used for structural steel include plain carbon Steel, low-alloy steel, and high-alloy specialty steel. The most commonly used structural steel includes Grade A36 and A992 steel. A36 steel is a mild steel produced to have yield strength of 250MPa as per ASTM standard A36 / A36M - 08 (Standard Specification for Carbon Structural Steel). A36 steel is plane carbon steel and its composition constitutes of maximum of 0.26% carbon, and 0.04%

phosphorus. A572 Gr. 50 and A992 steels are low carbon alloy steel with maximum carbon content of 0.23% and small amount of molybdenum, chromium, vanadium and boron to improve the strength characteristics. Both these steels show yield strength of 350MPa and ultimate strength of 450MPa. Generally, the conventional structural steels and annealed to achieve required strength. Required cross section shapes are achieved by hot or cold rolling or forging for use in structural members.

**High strength steel bolts**

Grade A325 and A490 bolts are two most commonly used bolts made with high strength steel. ASTM A325/A325M and A490/A490M specify the composition and heat treatment requirement of these bolts. Basic alloying elements found in A325 and A490 bolts are carbon, manganese, phosphorus, sulfur and silicon. Occasionally molybdenum, chromium, vanadium and boron are added to these bolts in order to improve their response to the heat treatment. The percent carbon content in A325 and A490 steel can vary from 0.3 - 0.52%. A325 and A490 bolts are manufacture by quenching and tempering to attain minimum tensile strength of 830MPa and 1030MPa at room temperature, respectively. The high strength properties of these bolts can be attributed to the formation of martensite crystal structure produced after the heat treatment and to the comparatively higher carbon content of these steel types.

## 2.4 High temperature properties of bolt steel

A review of literature shows that there are a limited number of experimental studies performed on steel bolts at high temperatures. This section provides a brief review of experimental and analytical studies on high temperature properties of bolts. Also a review of constitutive relations for steel bolts provided in codes and standards namely EC 2005, ASCE 1992 and BS 5950 is presented.

### 2.4.1 Experimental studies

A review of literature shows that there are only a limited number of experimental studies on the high temperature behavior of steel bolts. A brief review of these experimental studies is presented here.

#### 2.4.1.1 Strength properties

Kirby (1995) performed high temperature strength tests on Grade 8.8 bolts to study the degradation of properties with temperature. A series of double shear and tension tests were conducted to study the properties of three types of M20 bolts namely A, B, and C with two types of nuts namely A and B for temperatures ranging from 20 to 800°C. Test bolts A and B were manufactured by hot forging, whereas bolt C was manufactured by cold forging to meet BS3692: Grade 8.8 strength requirements. Nut A was manufactured as per requirements of BS970: Part l: Grade 080M30 and B was manufactured as per BS3111: Part l: Type O. Mechanical properties of these test nuts and bolts are given in Table 2.1.

Table 2.1 - Mechanical properties of Grade 8.8 bolts used in high temperature tests by Kirby

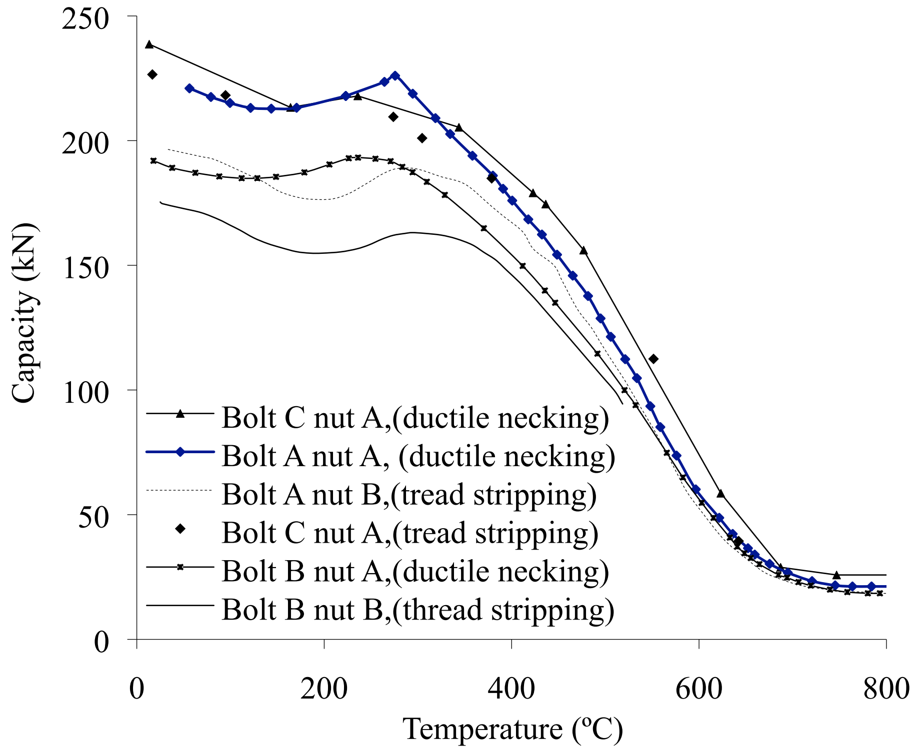
| Test bolts A, B and C manufacture as per BS3692: Grade 8.8 Specifications | | | |
|---|---|---|---|
| Mechanical property | Tensile strength | 0.2% Proof stress | Elongation after fracture |
| Requirement | Min: 785MPa<br>Max: 981 MPa | Min: 628 MPa | Min: 12% |
| Nuts A and B manufacture as per to BS3692 Specifications | | | |
| Mechanical property | Proof load stress | Brinell hardness HB | Rockwell/Vickers hardness HRC |
| Requirement | 785MPa | Max 302 | Max 30/130 |

A series of tension tests were planned on five sets of nut and bolt combination as shown below. Figure 2.3(a) illustrates the results of the tension test. The figure shows that depending on the fit between the test nut and bolt, two different failure mechanisms can occur in bolts subjected to tension namely ductile necking and thread stripping. Double shear test was performed on test bolt Type A and C for two different shear paths. These shear paths include both shear planes acting on shank and shear plane acting shank and threads. Despite the shear path, failure was observed in the shank due to the tensile stresses caused by the prying action. Figure 2.3(b) illustrates the influence of temperature on the shear capacity of bolts.

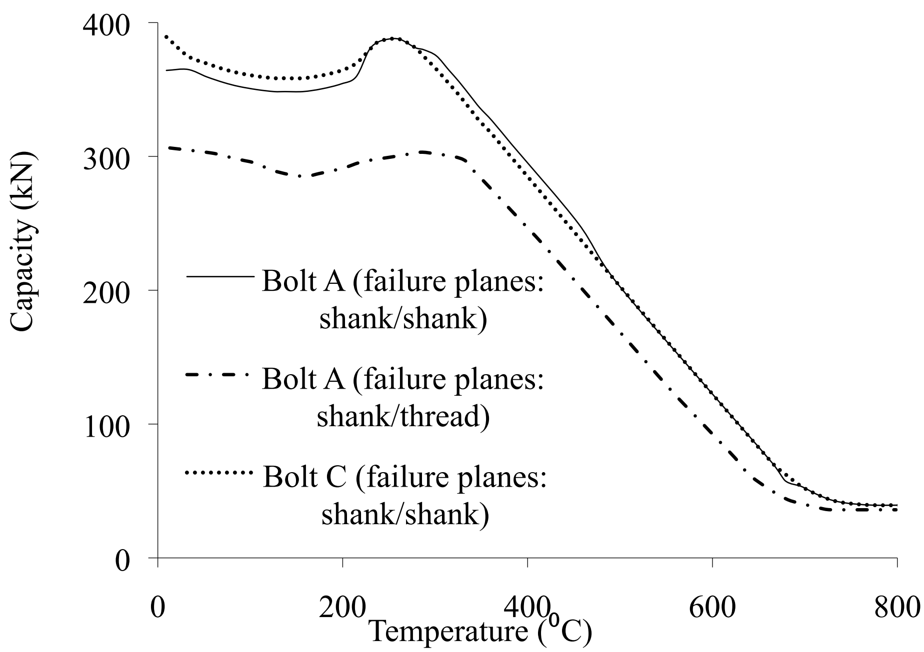
Both tension and shear test results indicate that all bolt and nut assemblies showed a marked loss in strength between 300 to 700°C. Test result highlighted that the present design curves based on the BS5950: Part 8 'Code of Practice for Fire Resistant Design' are conservative. Other notable conclusion of the study is that hot forged bolts are more sensitive to temperature variation when compared to cold forged bolts.

Based on these test results a tri-linear formula to determine tensile and shear strength reduction factors was proposed as follows;

$$\text{Strength reduction factor} = \begin{cases} 1.0, & T \leq 300^\circ C \\ 1 - (T - 300) \times 0.2128 \times 10^{-2} & 300^\circ C < T \leq 680^\circ C \\ 0.170 - (T - 680) \times 0.5312 \times 10^{-3} & 680^\circ C < T \leq 1000^\circ C \end{cases}$$




(a) Tensile strength of Grade 8.8 bolts




(b) Shear strength capacity of Grade 8.8 bolts

Figure 2.4 – High temperature tension and shear test results of Grade 8.8 bolts (Kirby, 1995)

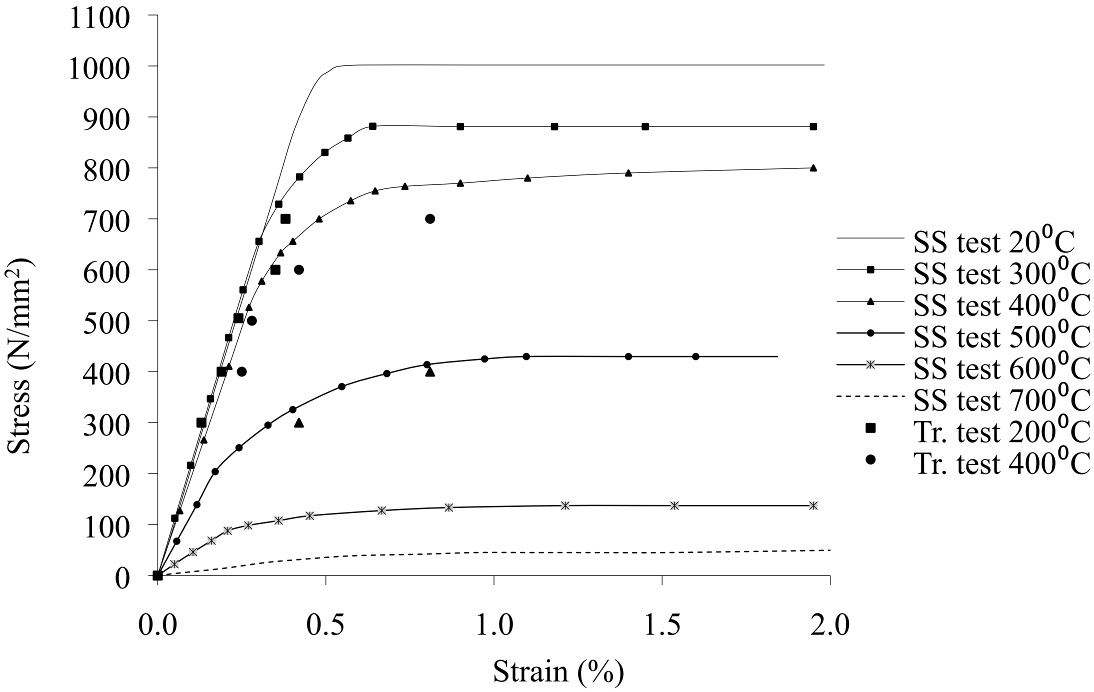
Gonzalez (2009) performed tension test on grade 10.9 bolts at elevated temperature. The test specimen, 16mm diameter was manufactured to meet the chemical composition requirement of DIN EN ISO 898 – (01). Table 2.2 illustrates the strength properties of the tested bolts.

Table 2.2 - Mechanical properties of Grade 10.9 bolts used for high temperature tension tests by Gonzalez (2009)

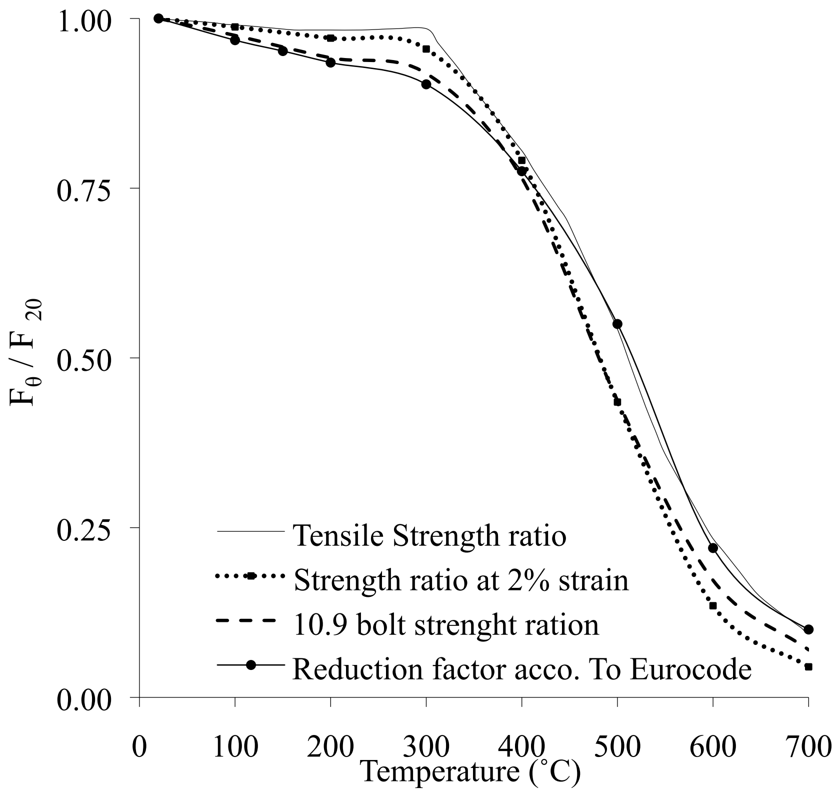
| Steel grade | Maximum tensile strength (MPa) | Strength at 0.2% strain (MPa) | Minimum expansion | Minimum lateral contraction | Vickers hardness HN; F≥ 98N |
|---|---|---|---|---|---|
| 10.9 | 1090 | 940 | 9% | 48% | 320-380 |

Strain controlled steady state tension tests for temperature range of 25 to 700°C and transient tests at temperatures of 200, 400 and 500°C were conducted on the bolts. Figure 2.4 (a) illustrates the high temperature stress-strain curve obtained from the steady state as well as transient test results. For given test temperature strength reduction factors for Grade 10.9 bolts were proposed as shown in Figure 2.4 (b).

These relationships indicate that influence of temperature on the strength of bolts is negligible till 300°C. However as temperature increases above 300°C, strength drastically reduces and was noted to be only 5% of its original value at 700°C. Comparison of test result with Eurocode provision indicates the conservative nature of these provisions at temperature above 400°C. Other notable conclusion from this study is that the high temperature creep has significant influence on the behavior of bolts at or above 300°C.




(a) Stress-strain relation for steady state (SS) and Transient (Tr.) test for Grade 10.9 bolt




(b) – Strength reduction factor for Grade 10.9 bolts

Figure 2.5 - Tension test results for Grade10.9 bolts performed by Gonzalez (2009)

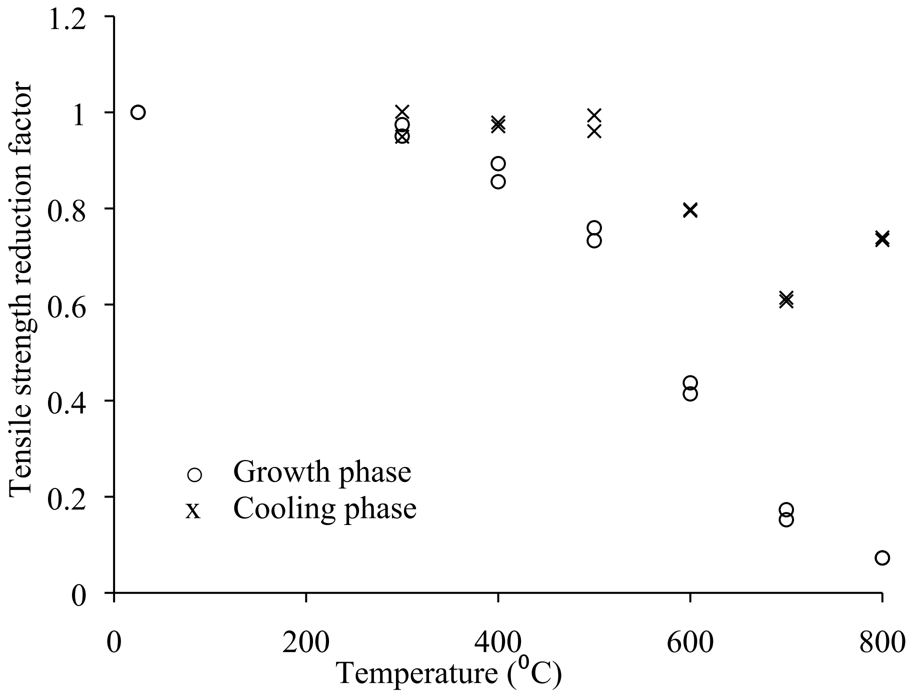
Sakumoto (1992) performed series of high temperature mechanical property tests and residual strength tests to verify the high temperature properties of fire resistant (FR) bolts. FR bolts are developed to maintain 2/3 of their original strength at 600°C and they perform comparatively better than the conventional steel bolts at elevated temperature.

Test specimen, F10T-type torque-control bolts and nuts with conventional NSW20B plain washers were prepared in accordance with the JSS II-09-1981. Mechanical properties of FR bolts are given in the following table.

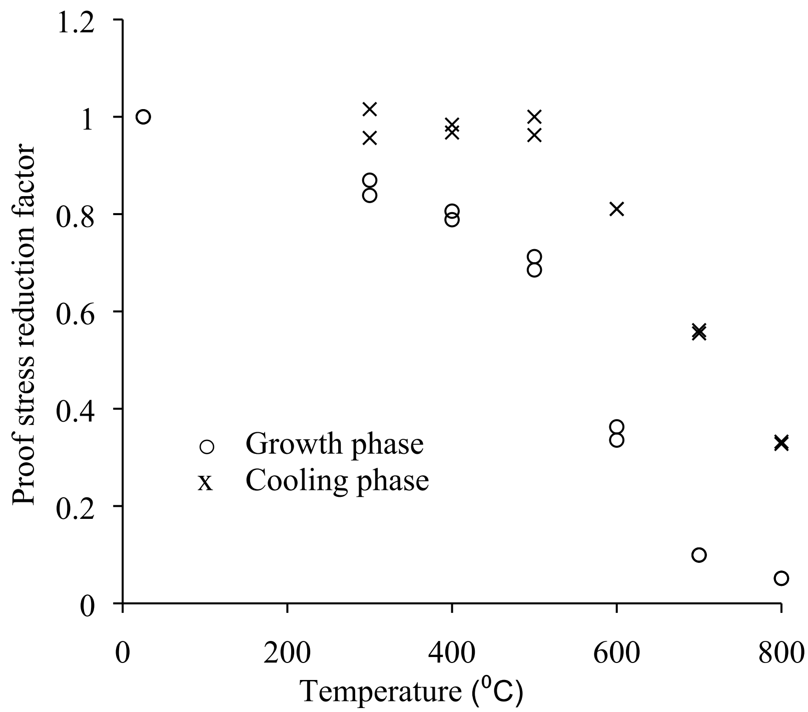
Table 2.3 - Mechanical properties of fire resistance bolts used high temperature tests by for Sakumoto (1992)

| Steel grade | Tensile Strength (MPa) | Proof Stress (MPa) | Elongation (%) | (%)Reduction in area |
|---|---|---|---|---|
| BOLTEN110-FR | 1130<br>Min - (980.7-1176.8) | 1090<br>Min - 882.6 | 19<br>Min - 14 | 65<br>Min - 40 |

The study concluded that tensile strength, proof stress and elastic modulus for FR bolts are not influenced by temperature up to 500°C as shown in Figure 2.5 (a) and (b). Although the strength of FR bolts reduced with increase in temperature, their performance was still superior to the conventional bolts (Figure 2.5 (d) and (e)). The same is true about the elastic modulus of FR bolts as indicated in Figure 2.5(c). Looking at the residual strength of FR bolts, it can be said that no significant reduction in strength of FR bolts is observed when the bolt is subjected to heating and cooling phases up to 500°C. The elastic modulus decreased rapidly with temperature. However it was restored to the room temperature value when the bolt was cooled down to room temperature. The test also concluded that the bolts and connection plates lose the clamping force with reduction in elastic modulus and relaxation of metal may result in increase in slip tendency of the friction joint.

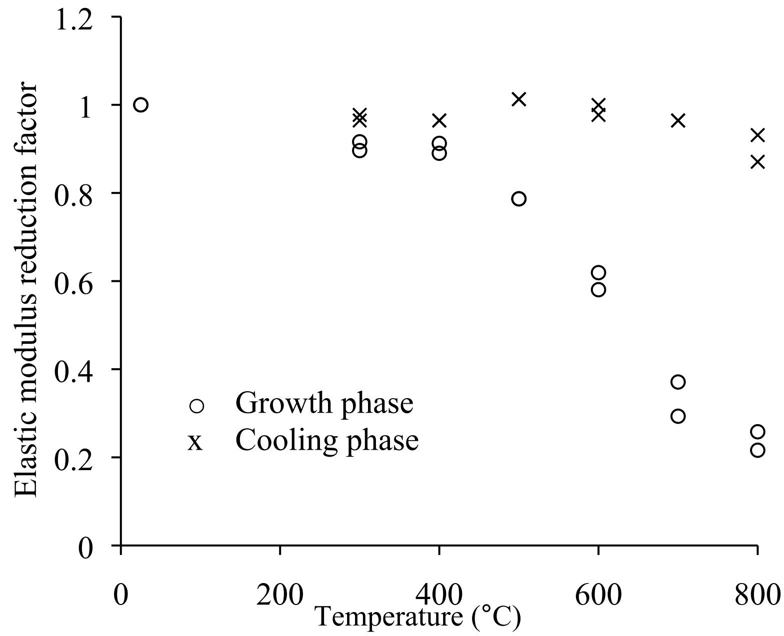



(a) Reduction in tensile strength for FR bolts

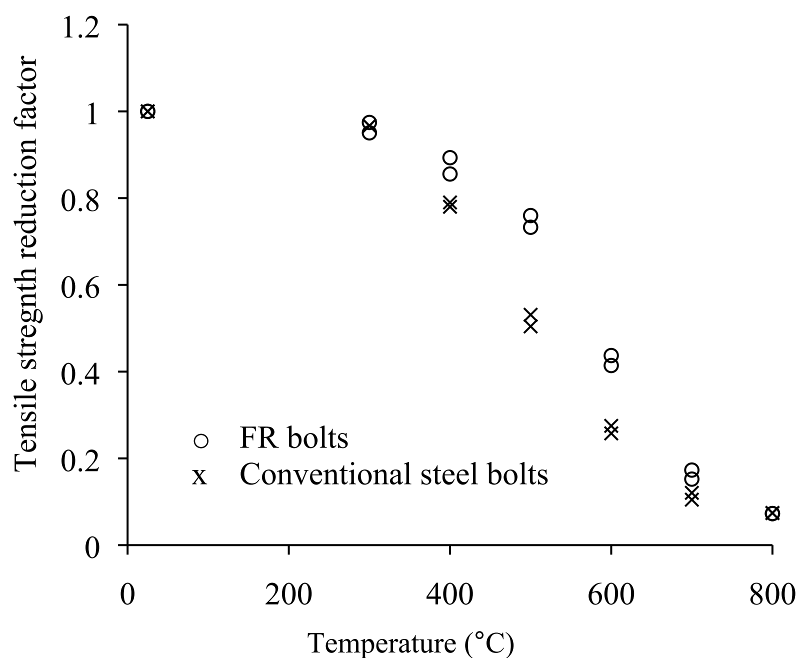



(b) Reduction in proof stress for FR bolt

Figure 2.6 – High temperature property test results of (FR) bolts by Sakumoto (1992)

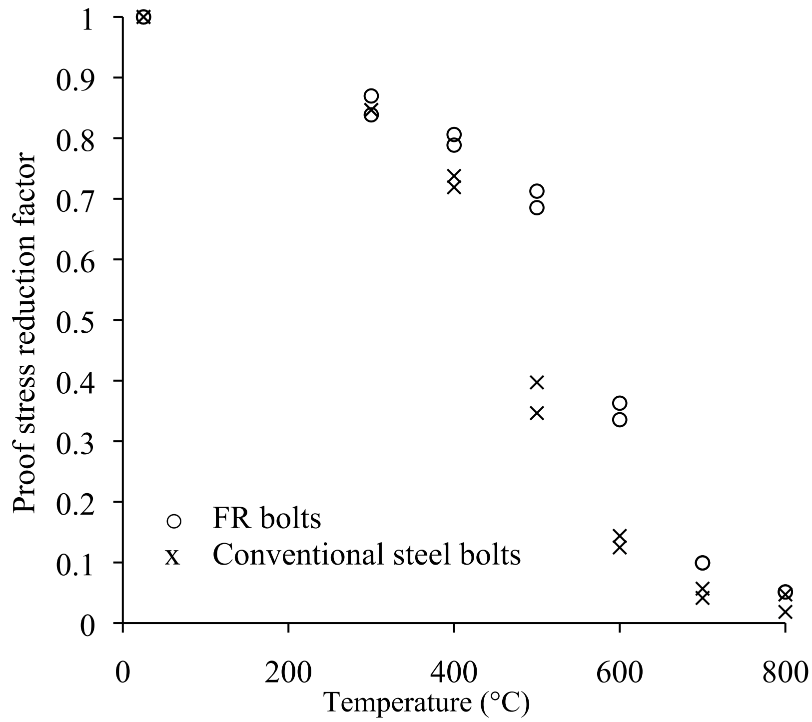



(c) Elastic modulus for FR bolts




(d)Tensile strengths of FR and conventional bolt

Figure 2.6 (cont'd) – High temperature property test results of (FR) bolts by Sakumoto (1992)




(e) Proof stress of FR and conventional bolt

Figure 2.6 (cont'd) – High temperature property test results of (FR) bolts by Sakumoto (1992)

Yu (2006) tested A325 and A490 bolts at elevated temperature. Double shear tests, single shear residual strength tests, slip load tests, and hardness tests were performed on 7/8" dia., A325 and A490 bolts. Mechanical properties of these bolts used in the tests are reproduced in the table below.

Table 2.4 Mechanical properties of A325 and A490 bolts used in high temperature tests by Yu(2006)

| Bolt grade | Hardness | Tensile strength (MPa) |
|---|---|---|
| A325 | 35 | 927 |
| A490 | 28-32 | 1126.5 |

Yu evaluated high temperature shear strength of A325 and A490 bolt, which was used to calculate the strength reduction factors (see Table 2.5). The study concluded that the shear strength of A325 and A490 bolts is not affected by temperatures up to 300°C, but decreases gradually with increase in temperature between 300 to 700°C to reach about 5% of its room temperature capacity. Comparatively, A325 bolts lose strength at a faster rate than that of the A490 bolts. Untested segments of bolt from the double shear test were cooled down to room temperature to perform residual strength test. Figure 2.7 clearly shows that A325 and A490 bolts regain their full strength when subjected to heating and cooling cycles up to 500°C. However if the temperature is increased beyond 500°C, on cooling, these bolts do not gain their full strength capacity.

Table 2.5 - Shear strength reduction factors (Yu, 2006)

| Temperature (°C) | A325 bolt | A490 bolt |
|---|---|---|
| 20 | 1.00 | 1.00 |
| 200 | 1.10 | 0.95 |
| 300 | 1.05 | 1.01 |
| 400 | 0.61 | 0.83 |
| 500 | 0.36 | 0.60 |
| 600 | 0.21 | 0.34 |
| 700 | 0.12 | 0.16 |
| 800 | 0.10 | 0.14 |

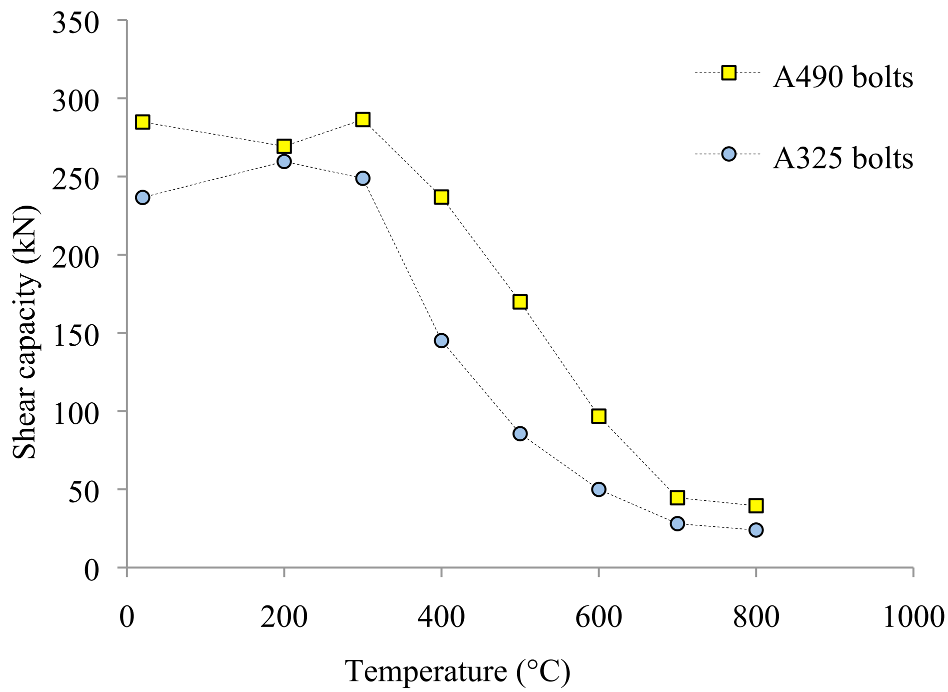



Figure 2.7 – High temperature shear strength of Grade A325 and A490 bolts by Yu (2009)

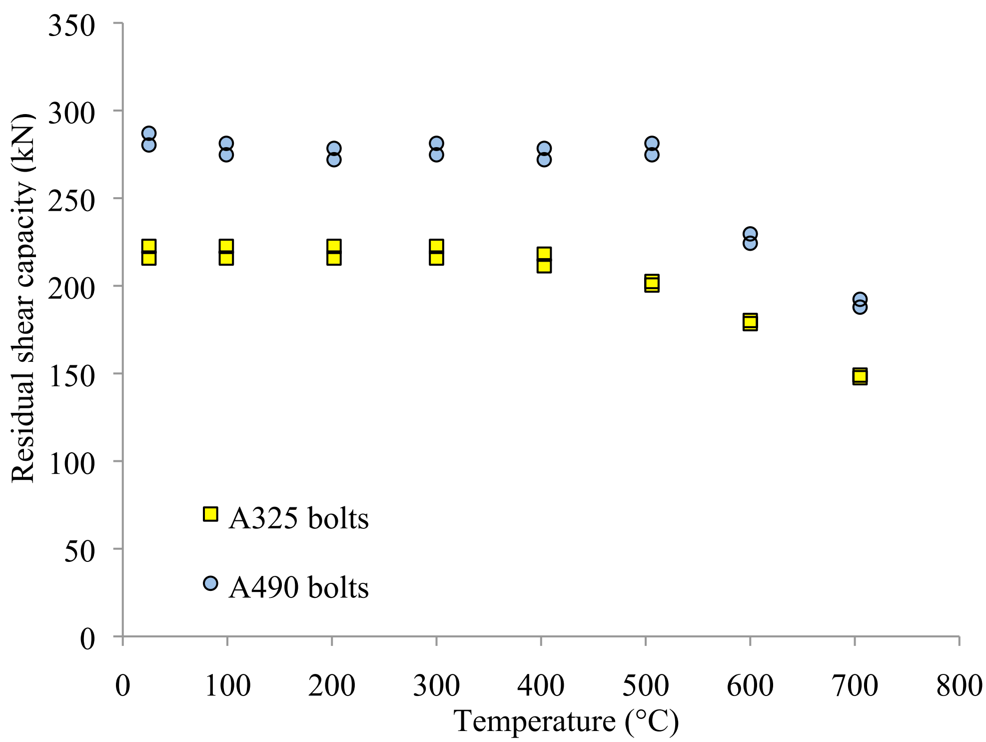
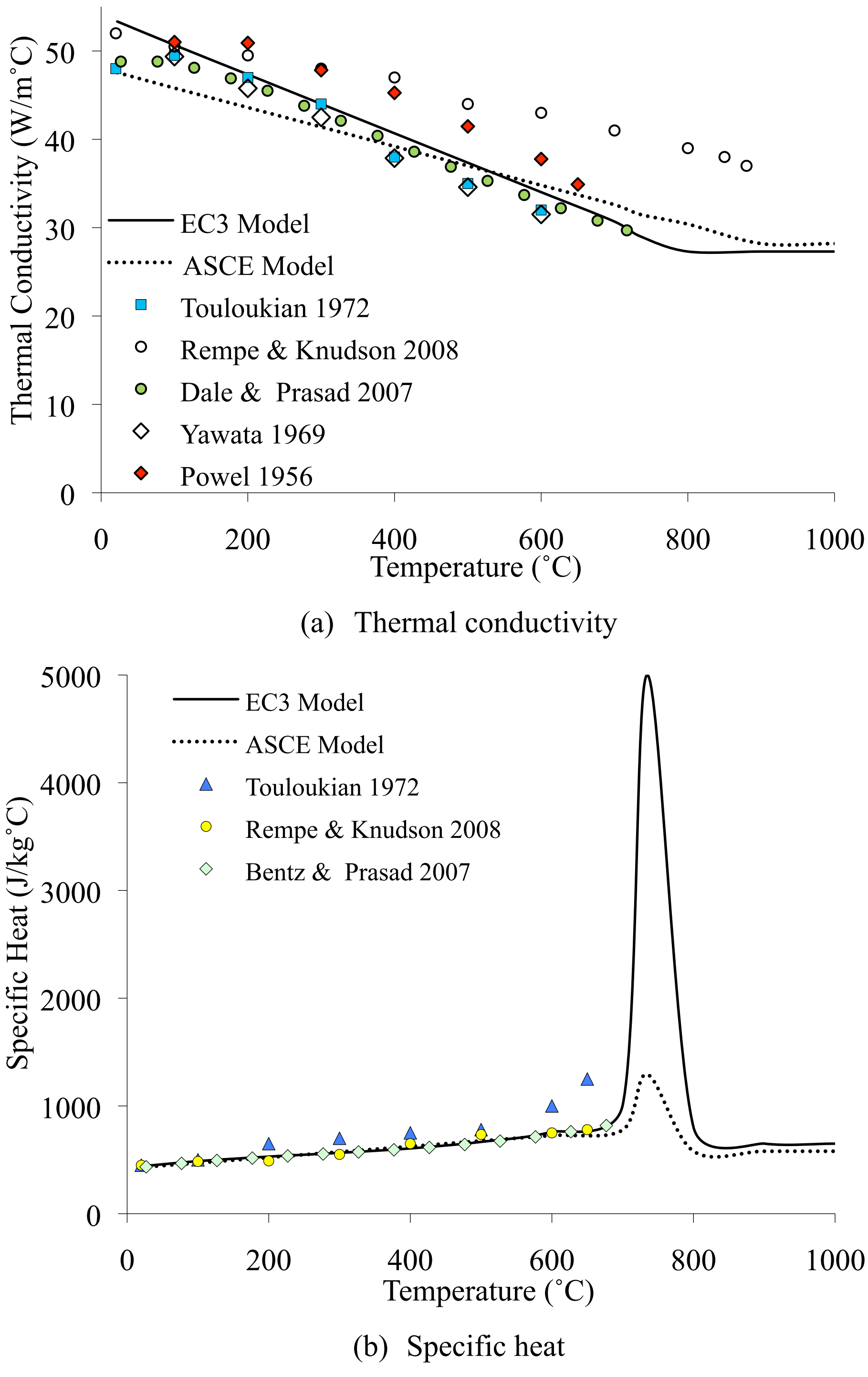



Figure 2.8 - Residual strength of Grade A325 and A490 bolts by Yu (2009)

#### 2.4.1.2 High temperature thermal properties of bolts

The thermal properties which influence the temperature rise in steel are thermal conductivity, specific heat and thermal expansion. These properties of steel vary with variation in temperature. Literature review shows that no specific studies have been carried to evaluate high temperature variations in thermal properties of high strength steel A325 and A490 bolts. However, quiet often the thermal properties of conventional structural steel are used for these high strength steel bolts. Furthermore, no studies are carried out to evaluate thermal properties of steel in the cooling phase of fire.

Kodur and Dwaikat (2010), reviewed thermal and mechanical properties of structural steel at high temperatures. The paper compiles different test models and EC3 (2005), ASCE (1992) standard provisions for high temperature material properties of structural steel. Figures 2.8 (a), (b) and (c), illustrate the thermal conductivity, specific heat and thermal expansion predicted by different test programs, EC3, and ASCE 1992 models. The comparison shows that up to 700°C thermal properties predicted by different models vary slightly. This variation can be attributed partly to the fact that majority of the existing data on thermal properties originates from studies carried out on iron and non-structural steel alloys. The thermal properties of steel being structure-sensitive, they vary with physiochemical changes that occur in steel at temperatures above 700°C. However measurement of these properties becomes complex beyond 700°C, when steel undergoes phase change. Hence majority of test programs predict thermal properties till 700°C thus not highlighting the influence of phase change on steel thermal properties.

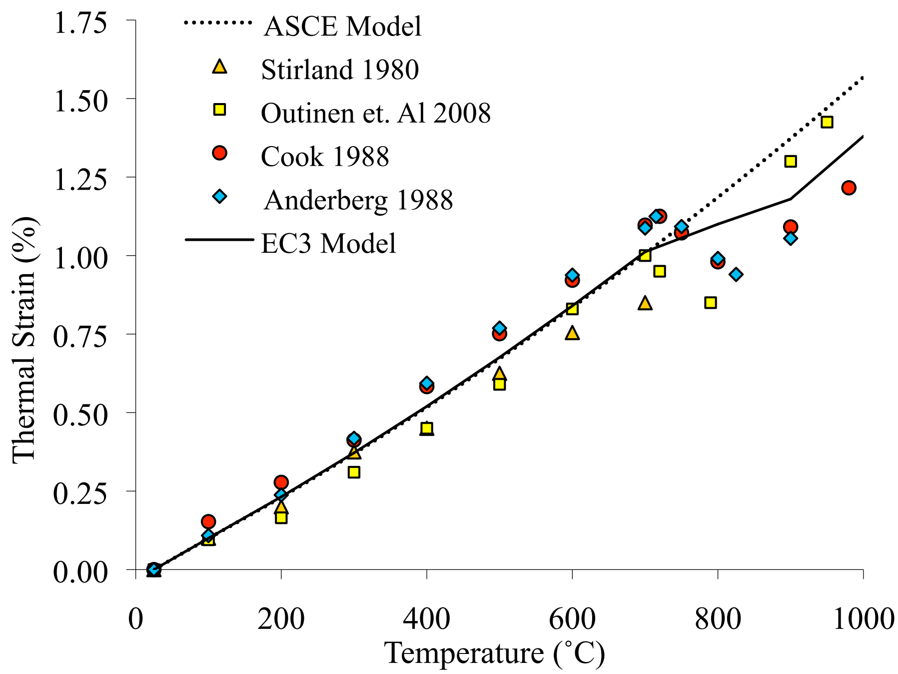



(a) Thermal conductivity



(b) Specific heat

Figure 2.9 - Thermal properties of steel predicted by different test programs and property relations in Eurocode and ASCE manual




(c) Thermal strain

Figure 2.9(cont'd) - Thermal properties of steel predicted by different test programs and property relations in Eurocode and ASCE manual

### 2.4.2 Provision in codes and standard

EC3 (2005), ASCE (1992) and BS5905 provide high temperature property relationships for evaluating thermal and strength properties of bolts at elevated temperature.

#### 2.4.2.1 Mechanical properties

Eurocode EC3 and British code BS 5950 provides the temperature dependence of mechanical properties of steel bolts.

EC3 considers that during a fire incidence, temperature in the connection can be lower than the other components of the structure because of the presence of large steel mass of connecting members. Thus it is considered that the net section failure of bolts will not occur. Shear and bearing are considered as the governing failure modes of fasteners at elevated temperature and are explained in Appendix B.1.

To determine fire resistance of steel bolts subjected to shear or bearing loads, empirical formulas are provided in EC3 part 1 and 2. These formulas relate high temperature strength of bolts with the ambient strength with strength reduction factors. These strength reduction factors denoted as $k_{b,\theta}$ (Table 2.6) represent the reduction in load carrying capacity of bolts with increase in temperature and are applicable for all types of bolts.

BS 5950 part 8: Code of Practice for Fire Resistant Design provides strength reduction factor for bolt steel at elevated temperature (Table 2.6). Due to lack of suitable experimental data these strength reduction factors at a specified temperature are derived as 80% of strength of structural steel, which correspond to 0.5% strain.

Table 2.6 – Strength reduction factors for bolts as per EC3 and BS 5905 part 8

| Temperature ($^{\circ}$C) | Strength reduction factor as per EC3 part 1 and 2 | Strength reduction factor as per BS 5950 part 8 |
|---|---|---|
| 20 | 1.000 | 0.8 |
| 100 | 0.968 | 0.776 |
| 150 | 0.952 | 0.767 |
| 200 | 0.935 | 0.757 |
| 300 | 0.903 | 0.683 |
| 400 | 0.775 | 0.638 |
| 450 | 0.550 | 0.577 |
| 550 | 0.220 | 0.39 |
| 700 | 0.100 | 0.15 |
| 800 | 0.067 | 0.06 |
| 900 | 0.033 | 0.02 |

#### 2.4.2.2 Thermal properties

EC3 and ASCE provides constitutive relations to determine thermal conductivity, specific heat and thermal expansion of steel at high temperatures.

Constitutive relations to determine that thermal conductivity, specific heat, and thermal expansion of steel provided by EC3-part 1 and 2 are given in appendix B.1. ASCE (1992) standard provides empirical formulas to determine the thermal properties of steel at elevated temperature. These formulas are applicable for all types of steel and are explained in Appendix B.2.

## 2.5 Summary

This chapter presents an overview on high temperature thermal and mechanical properties of steel bolts. Review of experimental studies indicates that unlike structural steel, information on high temperature thermal and mechanical properties of high strength bolts is quite limited. The results from these studies indicate that the strength of heat treated high strength bolts is far more temperature sensitive than hot forged conventional steel. Also heat treatment of bolts can significantly improve its high temperature properties and bolts can be produced for improved fire performance. Review of literature shows that studies carried out to evaluate thermal properties of steel are either based on iron or nor-structural steel. Hence may not be applicable for the different types of high strength steel used for manufacturing bolts. Furthermore, no data is available on the thermal properties of steel in cooling phase of fire. Since current guidelines provided by codes and standards to determine high temperature thermal and mechanical properties of bolts are either assumed from the properties of conventional steel or are based on very limited experimental studies, these guidelines are conservative and may lead to unrealistic design on bolted connection. For realistic and accurate fire design of connection, there is a clear need of detailed knowledge of high temperature properties of bolts.

## Chapter 3

## High temperature thermal Properties

### 3.1 General

Past fire incidents and research have shown that temperatures in unprotected steel members can reach up to 800°C in about 45 minutes. These high temperatures can lead to significant strength degradation in steel after about 400°C and at 800°C steel possess only 10% of its original strength. As the strength degradation is a function of temperature, accurate prediction of temperatures in steel is critical. Such a prediction requires detailed high temperature thermal properties of steel.

The three thermal properties that influence the temperature rise and fire induced forces in steel are thermal conductivity, specific heat and thermal expansion. These properties can vary in the heating and cooling phase of the fire. At present, design codes and standards provide thermal property relations as a function of temperature derived from the experimental studies on conventional structural steel. These properties of conventional steel are often used for high strength steel, the bolts are made of. Furthermore there is a lack of data on thermal properties of steel in cooling phase of fire, hence these properties are assumed to be same as that of heating phase. No specific study has been conducted on A325 and A490 high strength steel bolts to determine high temperature thermal properties in heating and cooling phase. Hence to fill the knowledge gaps a series of thermal property tests were conducted to evaluate high temperature thermal properties of Grade A325, A490 and A36 steel in both heating and cooling phases. A detailed description of the tests and discussion of test results on thermal properties are presented in this chapter.

### 3.2 Thermal conductivity and specific heat

Thermal conductivity ($k_t$) is defined as the ratio of heat flow rate to temperature gradient. It represents the ability of steel to conduct heat. Specific heat is the amount of heat required to change the temperature of a unit mass material by unit degree temperature. It represents the energy absorbed by the steel during heating or cooling and accounts for sensible heat and latent heat involved during temperature change. Sensible heat ascribes to the heat involved in thermodynamic reaction during temperature change. Latent heat refers to the energy absorbed or released by the material during phase transition. The material absorbs considerable amount of energy during physiochemical changes and hence specific heat values are highly influenced by the phase transition in materials. These two properties influence the temperature rise and its distribution within the steel structural members and hence form important input data parameters for thermal analysis. The following section discusses the test procedure and results in detail.

#### 3.2.1 Test specimen

Three types of steel namely A36, A325 and A490 were used to fabricate test specimen for undertaking thermal property tests. A36 is low carbon steel produced by annealing or forging to attain normal strength properties and structural members such as beams and columns are made of this steel. A325 and A490 are medium carbon steels produced by quenching and tempering to attain high strength properties and are used in bolts. Chemical composition and mechanical properties of these steels are given in Table 3.1.

Specimens of size 40mm in diameter and 25mm in height as shown in Figure 3.1 were used for conducting the tests. These specimens were machine cut from factory-manufactured bolts of size 40mm in diameter and 510mm long. As the accuracy of test result greatly depends on the specimen-to-sensor contact, these specimens were ground smooth to ensure uniformity of the surface and enhance specimen-to-sensor contact.

Figure 3.1 – Typical test specimen for undertaking thermal conductivity and specific heat tests

Table 3.1 - Chemical composition and mechanical properties of A36, A325 and A490 test steel

(a) Chemical composition

| Steel | C | Mn | P | S | Si | Cr | Mo | Cu | B | Al | Ni |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A36 | 0.16 | 0.80 | 0.10 | 0.027 | 0.27 | 0.12 | 0.01 | 0.22 | - | 0.031 | 0.07 |
| A325 | 0.25 | 0.6 | 0.02 | 0.05 | 0.17 | - | 0.025 | - | <0.0005 | - | 0.06 |
| A490 | 0.3 | 0.80 | 0.40 | 0.04 | 0.27 | 0.12 | 0.02 | 0.04 | <0.0005 | - | 0.06 |

(b) Mechanical properties

| Steel | Tensile strength (MPa) | Yield strength (MPa) | Elongation (%) | Reduction in area (%) |
|---|---|---|---|---|
| A36 | 500 | 290 | 28 | 52 |
| A325 | 830 | 630 | 14 | 35 |
| A490 | 1030 | 895 | 14 | 40 |

### 3.2.2 Test apparatus

Thermal conductivity and specific heat properties were measured using commercially available thermal constants analyzer apparatus Hot Disk "TPS2500" shown in Figure 3.2a as per ISO/DIS22007 standard. This state-of-the-art equipment is based on transient plane source (TPS) technique and determines thermal properties of material. Hot disk can be used to perform the thermal conductivity and specific heat tests at temperatures ranging from room temperature to 730°C. It utilizes a flat plate sensor to determine thermal properties of material. These transiently heated plate sensors are made-up of double spiral, electricity-conducting pattern etched out from thin nickel foil. These sensors are insulated with 25 μm thick layer of kapton (Figure 3.2) or mica (Figure 3.5). Kapton sensor is used for conducting tests at temperatures up to 200°C, whereas mica sensor is used for measurements in the temperature range of 100°C to 735°C (Adl-Zarrabi et al. 2006). The sensor acts as a heat source as well as detector (resistance thermometer) and reads input and output voltage signals. During test, the temperature in the sensor rises and heat flow starts in the specimen being tested. This increase in temperature affects the resistance of the specimen. The variation in resistance is then recorded as a voltage. The recorded voltage is utilized to determine thermal conductivity, specific heat and thermal diffusivity of the test specimen.

### 3.2.3 Test procedure

Thermal conductivity and specific heat of steel was measured during heating phase and cooling phase at nine temperature points - 20, 100, 200, 300, 400, 500, 600, 700 and 735°C.

Kapton sensor was used to undertake tests at 20°C. This Kapton sensor is sandwiched between two halves of test specimens and is connected to Hot Disk (TPS2500). The specimen and sensor assembly rests on a platform of room temperature specimen holder (Figure. 3.2 b). A small pressure is applied to ensure good contact between the specimen and the sensor. The equipment sends and receives signal through the sensor connected to the TPS2500S (Figure 3.2 a), and is monitored with a computer program.

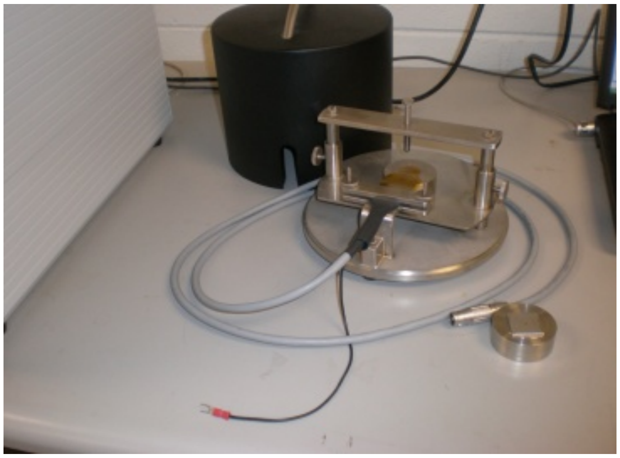
For high temperature tests, mica sensor was sandwiched between two specimens and the assembly was subjected to high temperatures in a furnace connected to the Hot Disk apparatus. The computer program controlled the temperature of specimen and furnace. In the heating phase the data was recorded in the temperature range of 100 to 735°C. Then the specimen was cooled down to room temperature to record the thermal conductivity and specific heat in the cooling phase. Up to 700°C the data was recorded at an interval of 100°C, and then an additional data value at 735°C. The various temperature points were designed to capture phase change in steel. This phase change that occurs in steel affects its thermal properties. ASCE and Eurocode guidelines show sudden change in specific heat value at approximately 735°C, which can be attributed to the phase transition process in steel. To capture this important phase change test was performed at 735°C. Heating rate of 5°C/min was determined depending on the rate required for steel to undergo physiochemical changes (Kodur and Harmathy, 2002). Higher heating rate causes the reaction to develop faster and shifts to higher temperature, which may overlook important phase change in steel. The test specimen must have uniform temperature throughout at the time of measurement.

In each test the specimen temperature was raised to the target exposure temperature and held for 30 minutes to achieve thermal equilibrium in the test specimen. At this stage the thermal conductivity and specific heat were recorded by data acquisition system. Then the temperature was increased or decreased to the next target temperature and this procedure was continued. The target temperature, sensor resistance and time of measurement were controlled by programmed test set up built into Hot Disk equipment.

Three tests were performed on each of the three steel types. Two specimens of each steel type were tested in both heating and cooling phase in 20 to 400°C temperature range, whereas the third specimen was tested in both phases in 20 to 735°C range and the measurements were recorded at an interval of 100°C.

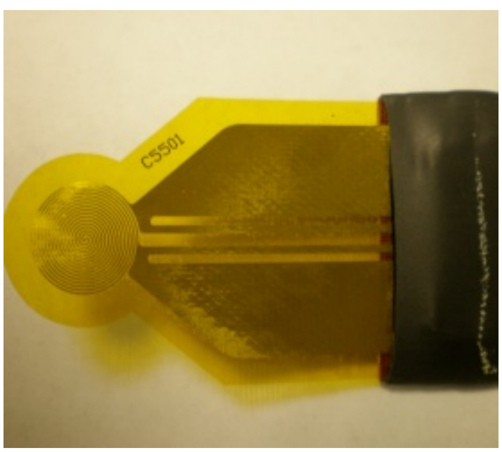
(a) Hot Disk (TPS 2500) thermal constant analyzer

Figure 3.2 – Test setup and apparatus for room temperature and high temperature thermal conductivity and specific heat tests



(b) Room temperature sample holder

(c) Electric furnace for high temperature tests




(d) Kapton sensor

(e) Mica sensor

Figure 3.2 (Cont'd) – Test setup and apparatus for room temperature and high temperature thermal conductivity and specific heat tests

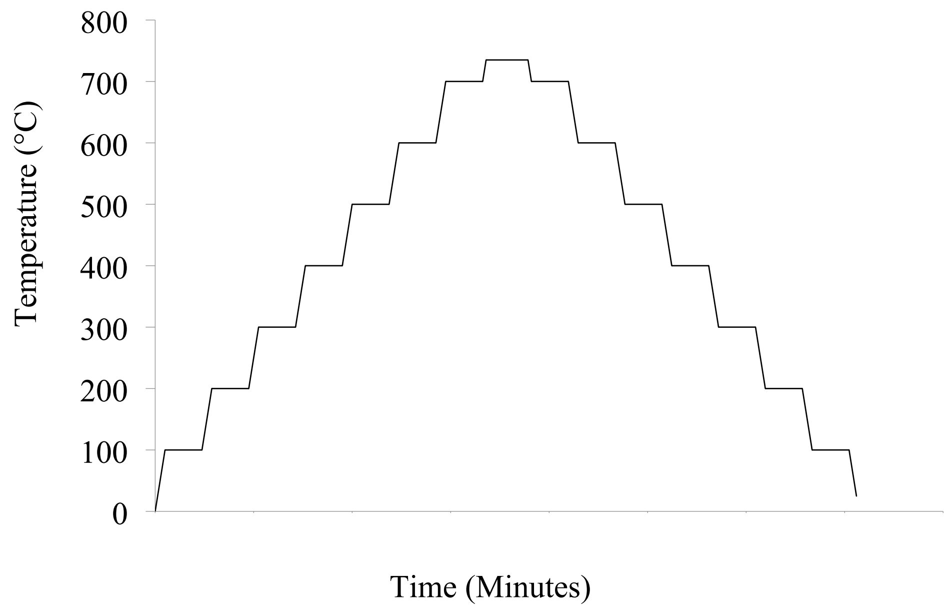



Figure 3.3 - Temperature ramp used for thermal conductivity and specific heat test

**3.2.4 Measured data - Thermal conductivity**

Thermal conductivity of different steel types that was obtained by averaging measured values is presented in Table 3.2. The test results show that the thermal conductivity is relatively high at room temperature and decreases gradually as the temperature increases. Room temperature values are observed to be ranging from 49 W/m°C to 47 W/m°C for three types tested steel. These values are in line with reported thermal conductivity of steel at room temperature, which was in the range of 46 to 65 W/mK (Kodur and Harmathy, 2007). As the temperature increases thermal conductivity decreases gradually. At 735°C, it is observed to be approximately 28 W/m°C for the three types of tested steel.

Figure 3.4 shows variation in thermal conductivity with temperature in heating and cooling phase. Figure 3.4 (a) shows that the thermal conductivity of three tested steel types vary slightly up to 600°C. However, this difference tends to decrease and is almost negligible at after 700°C.

Thermal conductivity of A490 steel is observed to be slightly less than that of A36 and A325 steel. This can be attributed to comparatively high carbon content in A490 steel. Thermal conductivity in highly influenced by the microstructure and chemical composition of the material. At room temperature it is highly sensitive to the carbon content of steel (Yafei, 2009). It decreases with increase in carbon content. However as the temperature increases, the effect of carbon content decreases as shown in Figure 3.5. As at these temperatures steel forms austenite in which carbon atoms dissolve, hence lesser free carbon is available for thermal conductance.

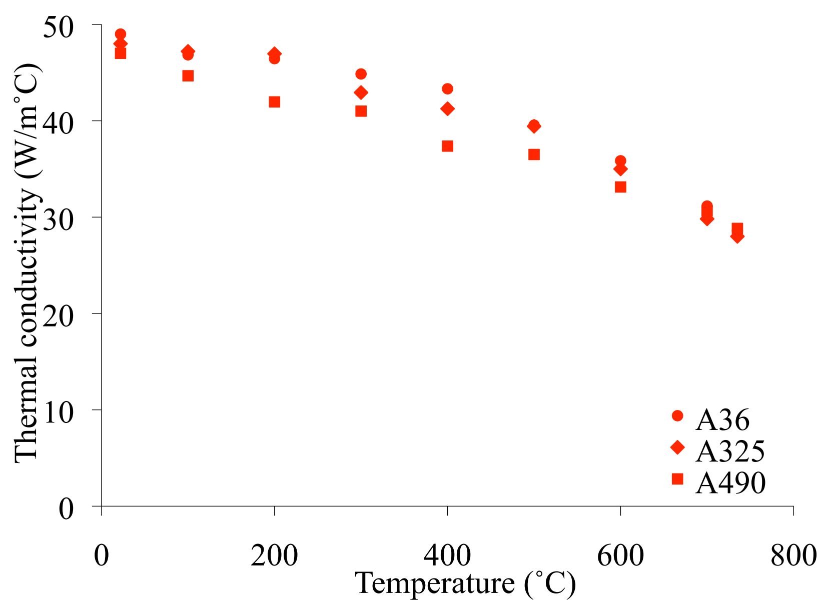
Figure 3.4 (c) shows that as steel cools down the thermal conductivity increases and restores its original value. Thus thermal conductivity measured in heating and cooling phase is in close proximity. Irrespective of the irreversible microstructure changes that occur due to temperature variation in steel, thermal conductivity in heating and cooling phase is very much similar and that was inferred by Kodur and Harmathy, (2007).

Table 3.2 (a) – Recorded thermal conductivity of A36, A325 and A490 steel in heating phase

| Temperature (°C) | Thermal conductivity (W/m°C) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A36 | | | | A325 | | | | A490 | | | |
| | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average |
| 25 | 50.1 | 48.9 | 49.0 | 49.3 | 48.3 | 49.5 | 47.3 | 48.3 | 48.6 | 44.8 | 46.8 | 46.7 |
| 100 | 47.3 | 44.5 | 47.4 | 46.4 | 47.3 | 48.8 | 46.9 | 47.6 | 45.0 | 46.5 | 44.0 | 45.1 |
| 200 | 45.9 | 47.4 | 46.3 | 46.5 | 47.8 | 46.0 | 45.7 | 46.5 | 41.7 | 41.8 | 42.7 | 42.1 |
| 300 | 44.2 | 44.6 | 45.2 | 44.7 | 43.3 | 42.2 | 42.9 | 42.8 | 41.2 | 39.5 | 39.4 | 40.0 |
| 400 | 42.9 | 43.8 | 43.1 | 43.3 | 41.0 | 40.8 | 41.7 | 41.2 | 36.8 | 38.2 | 37.6 | 37.5 |
| 500 | - | - | 39.5 | 39.5 | - | - | 39.4 | 39.4 | - | - | 36.5 | 36.5 |
| 600 | - | - | 35.8 | 35.8 | - | - | 35.0 | 35 | - | - | 33.1 | 33.1 |
| 700 | - | - | 31.1 | 31.1 | - | - | 29.8 | 29.8 | - | - | 30.6 | 30.6 |
| 735 | - | - | 28.6 | 28.6 | - | - | 28.0 | 28 | - | - | 28.8 | 28.8 |

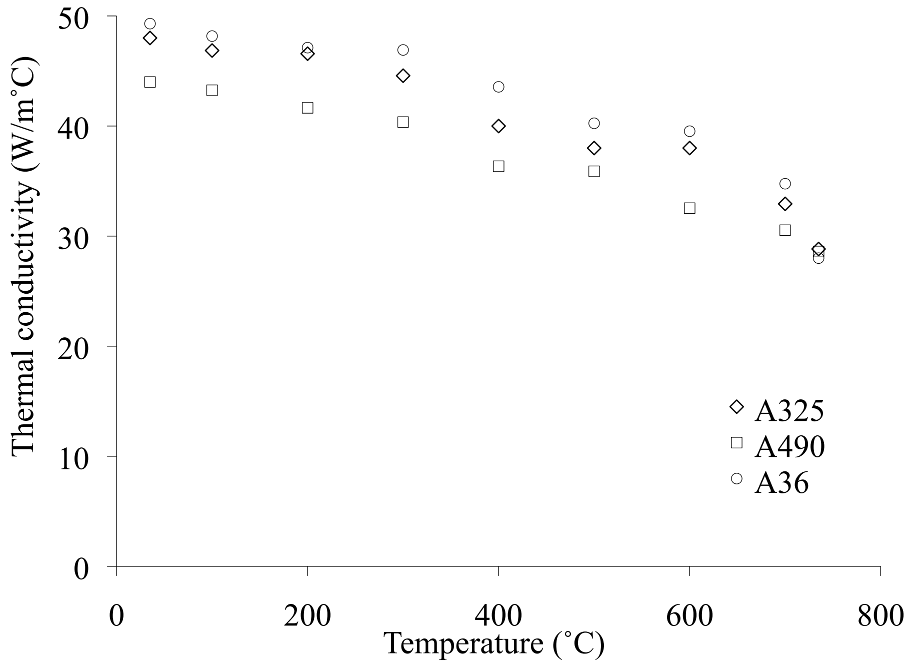
Table 3.2 (b) (Cont'd) – Recorded thermal conductivity of A36, A325 and A490 steel in cooling phase

| Temperature (˚C) | Thermal conductivity (W/m˚C) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A36 | | | | A325 | | | | A490 | | | |
| | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average |
| 25 | 48.7 | 48.6 | 49.3 | 49.3 | 49.0 | 47.0 | 48 | 48.0 | 45.9 | 47.1 | 46.0 | 46.3 |
| 100 | 48.3 | 47.9 | 48.1 | 48.1 | 47.2 | 45.9 | 46.8 | 46.6 | 44.3 | 42.6 | 43.2 | 43.4 |
| 200 | 46.0 | 48.9 | 47.1 | 47.1 | 45.8 | 45.5 | 46.5 | 45.9 | 42.3 | 40.0 | 41.6 | 41.3 |
| 300 | 47.9 | 45.8 | 46.9 | 46.9 | 45.6 | 43.8 | 44.5 | 44.6 | 40.4 | 40.8 | 40.3 | 40.5 |
| 400 | 42.7 | 44.6 | 43.6 | 43.5 | 39.9 | 41.0 | 40.0 | 40.3 | 36.3 | 37.7 | 35.9 | 36.6 |
| 500 | - | - | 40.2 | 40.2 | - | - | 38.0 | 38 | - | - | 35.8 | 35.8 |
| 600 | - | - | 39.5 | 39.5 | - | - | 38.0 | 38 | - | - | 32.5 | 32.5 |
| 700 | - | - | 34.7 | 34.7 | - | - | 32.9 | 32.9 | - | - | 30.5 | 30.5 |
| 735 | - | - | 28.0 | 28.0 | - | - | 28.8 | 28.8 | - | - | 28.6 | 28.6 |

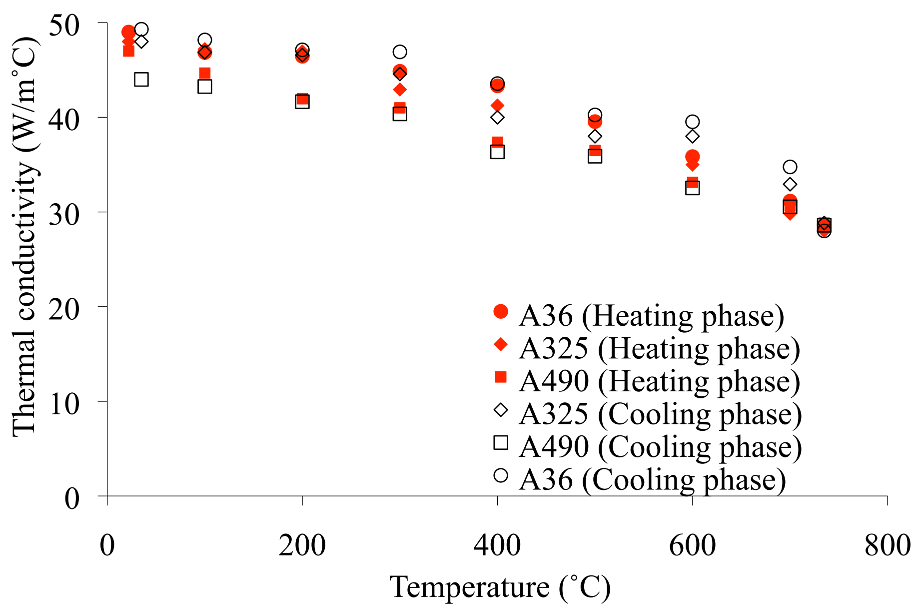



(a) Heating phase

Figure 3.4 – Measured average thermal conductivity of A36, A325 and A490 steel in heating and cooling phase




(b) Cooling phase




(c) Comparison of thermal conductivity in heating and cooling phase.
Figure 3.4(cont'd) – Measured average thermal conductivity of A36, A325 and A490 steel in heating and cooling phase

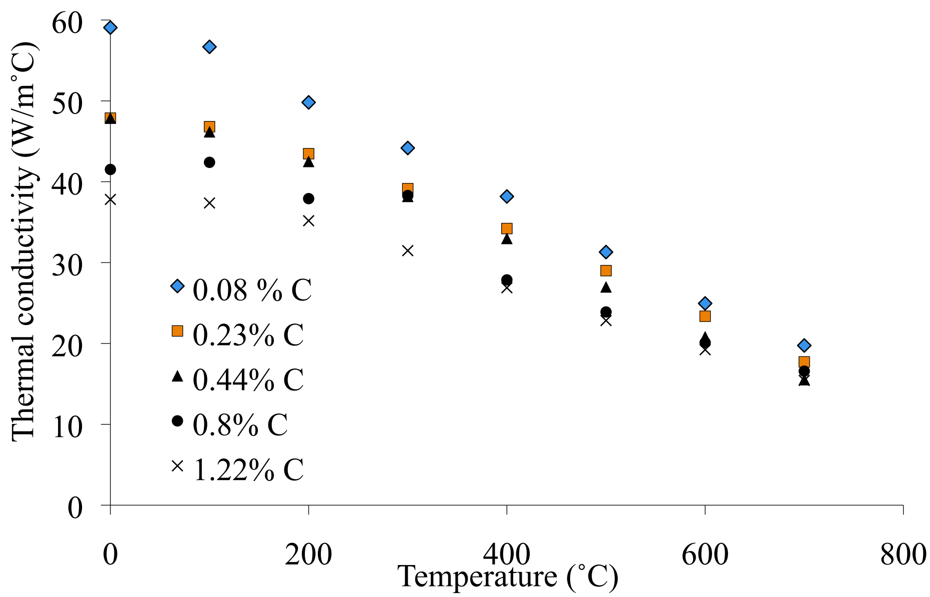



Figure 3.5 – Effect of carbon content (C) of steel on thermal conductivity (Yafei, 2009)

### 3.2.5 Measured data – Specific heat

Average specific heat for various types of steel measured duringtests is tabulated in Table 3.3. The result indicates that specific heat of the A36, A325 and A490 steel at room temperature is 490(J/kg°C), 506 (J/kg°C) and 493 (J/kg°C) respectively. These values increase slowly with rise in temperature up to 600°C. At 735°C, these values suddenly increase to 1538 (J/kg°C), 1487 (J/kg°C) and 1798 (J/kg°C). This increase can be attributed to contribution of latent heat required for the phase change that occurs in steel. Around 700°C, steel undergoes phase transition from body centered cubic crystal structure 'α ferrite' structure to α ferrite to α ferrite + γ austenite a face centered cubic crystal structure. This phase change in steel absorbs considerable energy (latent heat) and results in sudden increase in value of specific heat. It also can be seen that specific heat for A36, A325 and A490 at each test temperature is different. This difference is negligible till 400°C. After 400°C difference in specific heat values appears to be increasing and can be attributed to the variation in carbon content of steel. When temperature of specimen reaches 'Curie point', specific heat varies with variation in carbon content of steel (Yafei, 2009) as shown in Figure 3.7.

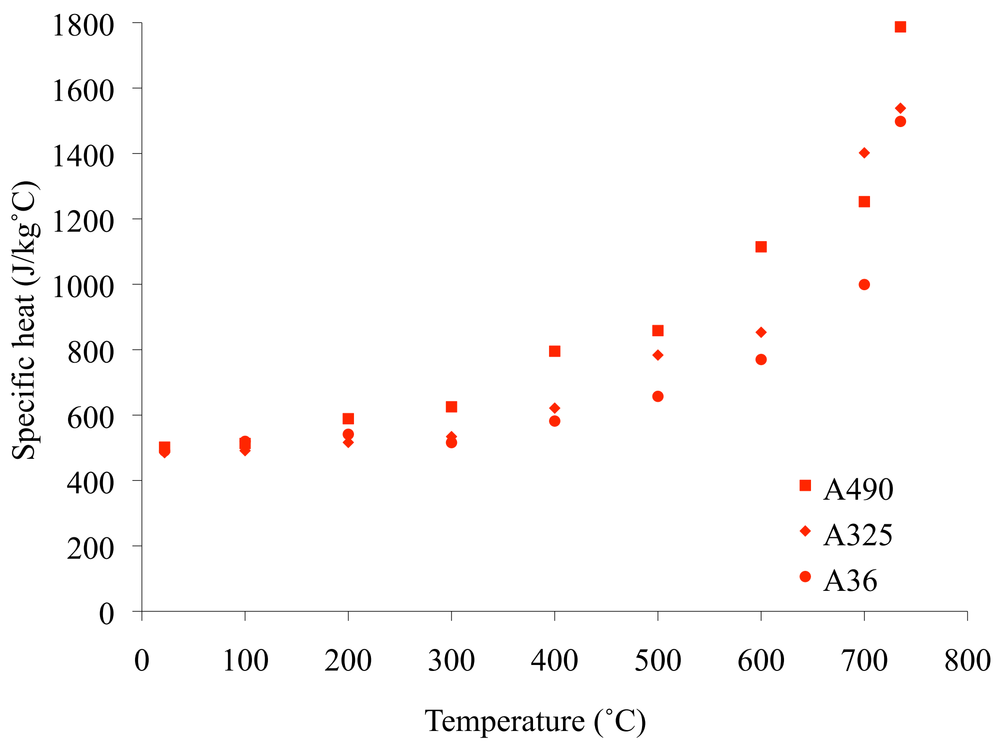
Table 3.3(a) – Recorded specific heat of A36, A325 and A490 steel in heating phase

| Temperature (°C) | Specific heat (J/kg°C) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A36 | | | | A325 | | | | A490 | | | |
| | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average |
| 25 | 500 | 495 | 475 | 490 | 508 | 499 | 513 | 506 | 495 | 498 | 485 | 493 |
| 100 | 495 | 499 | 488 | 494 | 521 | 515 | 515 | 516 | 506 | 535 | 520 | 520 |
| 200 | 505 | 515 | 520 | 514 | 710 | 712 | 702 | 708 | 544 | 553 | 536 | 544 |
| 300 | 528 | 533 | 540 | 534 | 770 | 776 | 765 | 770 | 503 | 525 | 506 | 512 |
| 400 | 635 | 629 | 616 | 627 | 824 | 815 | 817 | 818 | 583 | 576 | 593 | 584 |
| 500 | - | - | 784 | 784 | - | - | 858 | 858 | - | - | 657 | 657 |
| 600 | - | - | 853 | 853 | - | - | 1115 | 1115 | - | - | 770 | 770 |
| 700 | - | - | 1402 | 1402 | - | - | 1253 | 1253 | - | - | 999 | 999 |
| 735 | - | - | 1538 | 1538 | - | - | 1487 | 1487 | - | - | 1798 | 1798 |

Table 3.3(b) – Recorded specific heat of A36, A325 and A490 steel in cooling phase

| Temperature (°C) | Specific heat (J/kg°C) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | A36 | | | | A325 | | | | A490 | | | |
| | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average | Test 1 | Test 2 | Test 3 | Average |
| 25 | 635 | 612 | 626 | 624 | 465 | 475 | 456 | 466 | 635 | 625 | 619 | 626 |
| 100 | 780 | 791 | 786 | 785 | 557 | 576 | 564 | 566 | 609 | 607 | 599 | 605 |
| 200 | 835 | 845 | 830 | 837 | 626 | 645 | 636 | 636 | 646 | 647 | 644 | 646 |
| 300 | 796 | 786 | 797 | 793 | 726 | 731 | 726 | 727 | 626 | 631 | 636 | 631 |
| 400 | 798 | 812 | 809 | 806 | 793 | 789 | 785 | 789 | 652 | 646 | 660 | 653 |
| 500 | - | - | 924 | 924 | - | - | 994 | 994 | - | - | 706 | 706 |
| 600 | - | - | 1024 | 1024 | - | - | 1093 | 1093 | - | - | 783 | 783 |
| 700 | - | - | 1189 | 1189 | - | - | 1346 | 1346 | - | - | 1350 | 1350 |
| 735 | - | - | 1538 | 1538 | - | - | 1487 | 1487 | - | - | 1798 | 1798 |

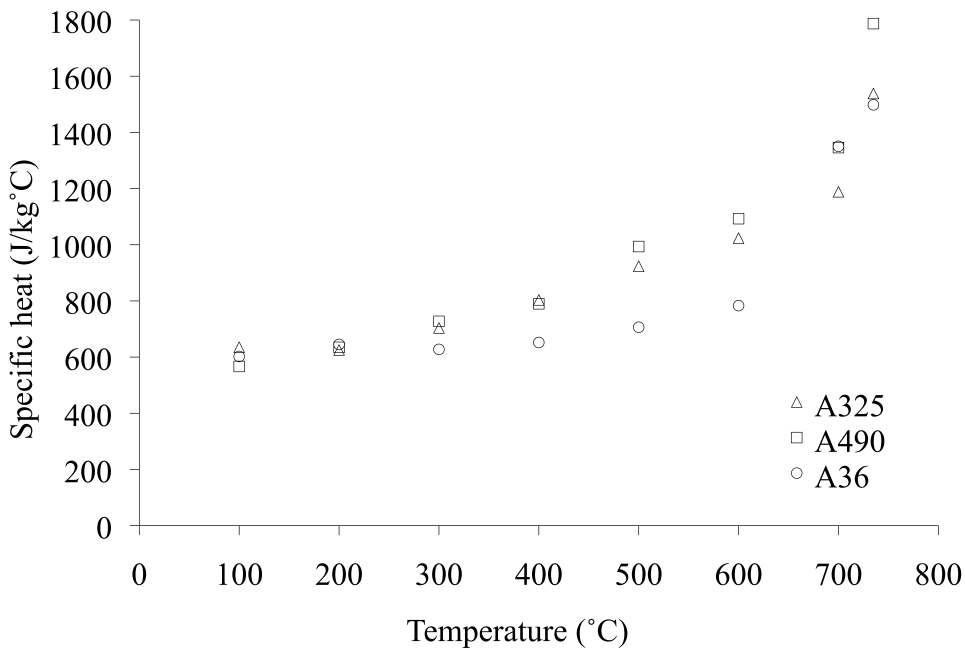
Figure 3.6 illustrates the specific heat of A36, A325 and A490 steel recorded in heating and cooling phase. Trends in Figure 3.6 (b) and Figure 3.6 (c) shows that, the specific heat starts to decrease with decrease in temperature during cooling of the steel and is observed to be in close proximity to the heating phase values. The specific heat accounts for the energy absorbed by the material during temperature variation. This energy absorbed by the particles is stored as minimal-sized deposits quanta. More the energy stored, higher the specific heat. During heating of the steel, more energy is supplied to the particles and thus specific heat increases with increase in temperature. When temperature starts to decrease energy stored in particles is limited. Thus as the temperature decreases the specific heat also reduces.



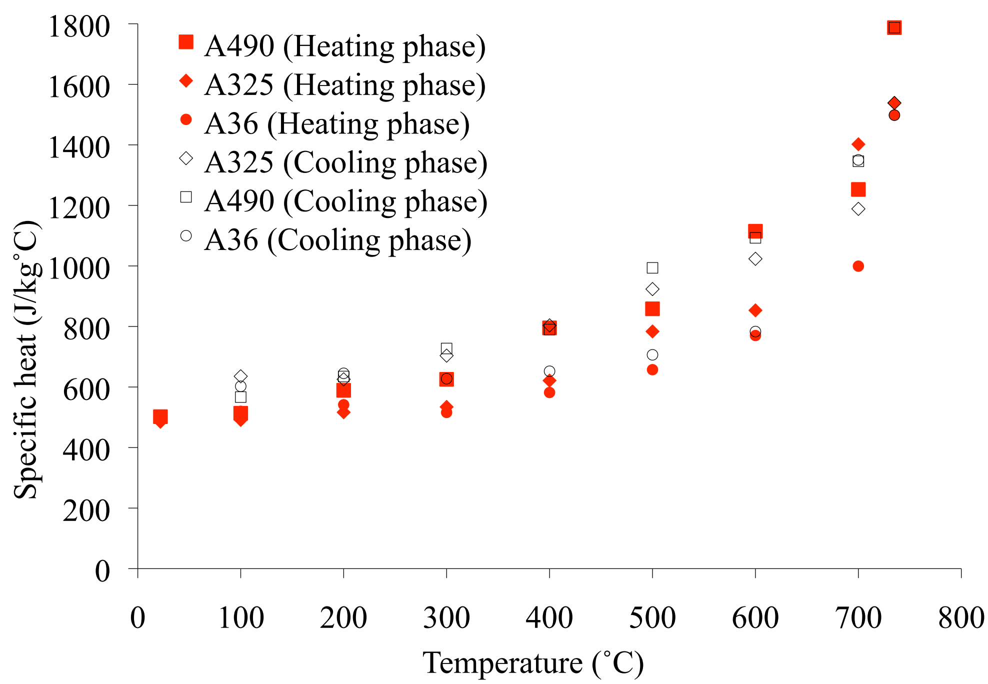

(a) Heating phase

Figure 3.6 – Measured average specific heat of A36, A325 and A490 steel in heating and cooling phase

(b)




(b) Cooling phase




(c) Comparison of recorded specific heat in heating and cooling phase

Figure 3.6(cont'd) – Measured average specific heat of A36, A325 and A490 steel in heating and cooling phase

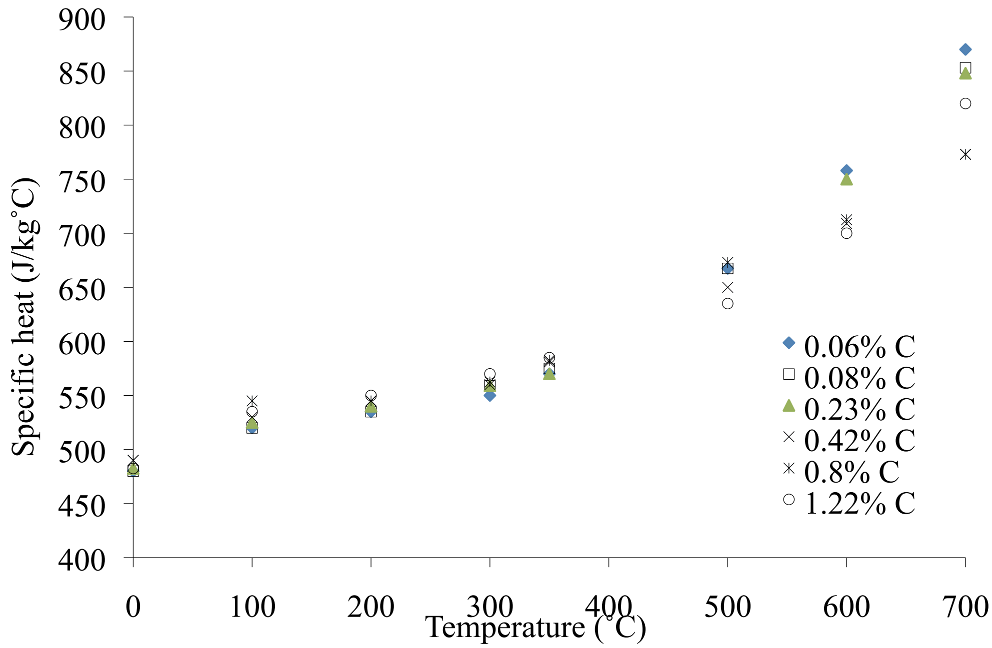



Figure 3.7 – Effect of carbon content (C) of steel on specific heat (Yafei, 2009)

### 3.2.6 Comparison of test data with published data

ASCE and Eurocode guidelines and published results from previous experimental studies on conventional structural and nonstructural steel was used to compare current measured test data for three types of steel. A comparison of the test results and published data is shown in Figure 3.8 and 3.9 to highlight the similarity and differences in the thermal property values of different steel types.

Figure 3.8 shows that the measured thermal conductivity falls within the range of values predicted by different models. ASCE and Eurocode models show that the thermal conductivity of steel decreases gradually with temperature. As per ASCE guidelines thermal conductivity decreases gradually till 900°C, but Eurocode takes into account the phase change that occurs in steel and hence predicts thermal conductivity to be constant after 750°C. However, different test programs are restricted to the temperatures up to 700˚C, due to the complexity of measurement of thermal properties during phase change in steel. Also the comparison of different code provisions indicates large variations in thermal conductivity at room temperature. This variation is observed to be decreasing with increase in temperature and the majority of models produce almost similar thermal conductivity at and after 700˚C. The thermal conductivity of material is very sensitive to the method and procedure used for conducting test. Hence large variations in the thermal conductivity can be observed in different test models. Also the comparison indicates that the chemical composition of steel has influence on its thermal conductivity. A490 steel with higher carbon content shows comparatively less thermal conductivity than A325 and conventional steel.

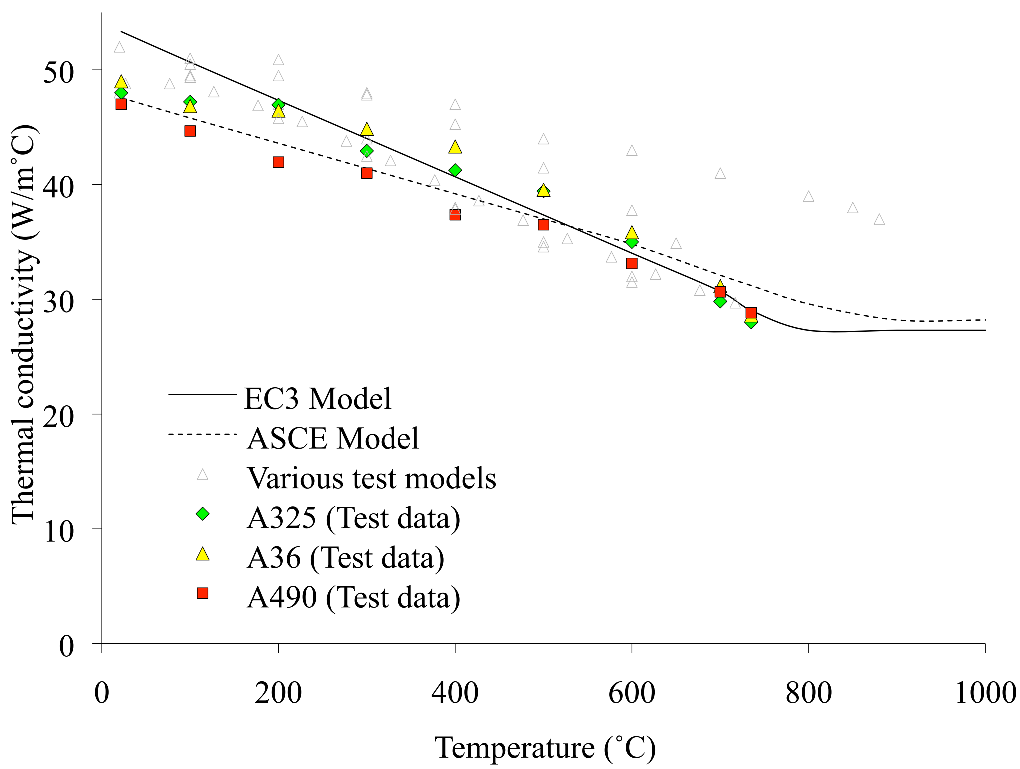



Figure 3.8 - Thermal conductivity of steel predicted by different models

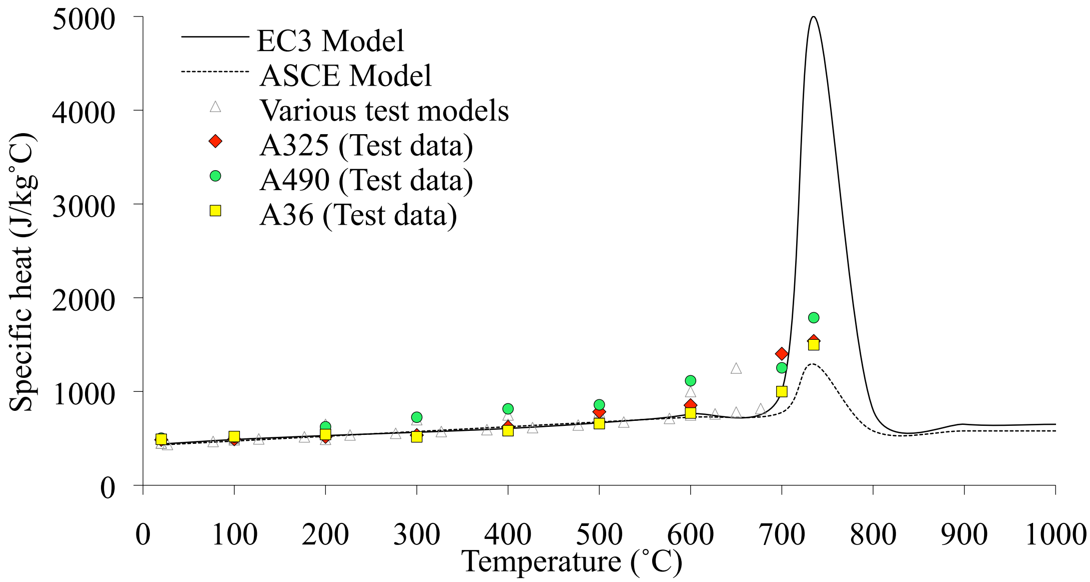



Figure 3.9 – Specific heat of steel predicted by different models

Trends in Figure 3.9 indicate that measured specific heat follow similar trend as that of ASCE and Eurocode provisions till 700°C. At 735°C specific heat predicted by Eurocode is 5000 J/kg°C, whereas as per ASCE it is 1200J/kg°C. The maximum temperature reached in the studies for other models was below 735°C, thus not capturing the full range of temperatures observed in fire conditions. The test results show the sudden increase in specific heat value at 735°C, however the maximum value observed is 1787 J/kg°C. A490 bolts show comparatively higher specific heat than A325 and conventional steel due to its higher carbon content.

### 3.2.7 Summary

Results for the thermal conductivity and specific heat tests on A36, A325 and A490 steel indicate that thermal properties vary with temperature. Also phase change that occurs in steel at high temperatures and chemical composition particularly on the carbon content of the steel has significant influence on its thermal properties.

### 3.3 Thermal expansion

The coefficient of thermal expansion is defined as the percentage change in length of the specimen per degree temperature rise. Thermal expansion of steel is generally affected by the phase change of steel. In order to evaluate thermal expansion of A36, A325 and A490 steels at elevated temperature, thermal expansion tests were conducted in 20 to 1000°C temperature range both in heating and cooling phase.

#### 3.3.1Test specimen

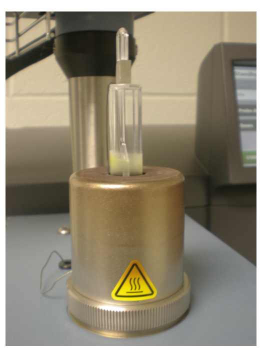
Test specimens of size 5mm x 5mm x 10mm were cut out from 40mm diameter A36 rods, and 40mm diameter A325 and A490 bolts, in the same as manner as that for thermal conductivity and specific heat tests to ensure material uniformity for all tests. The detailed chemical composition and mechanical properties of these steels are given in Table 3.1. Figure 3.10 (c) shows the test specimen used for thermal expansion test.

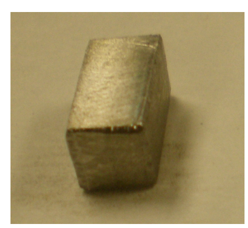
#### 3.3.2 Test apparatus

Q-400 EM, Thermomechanical Analyzer (TMA) shown in Figure 3.10 (a) was used to measure thermal expansion of A36, A325 and A490 steel at elevated temperature as per ASTM E831-06 standard (2006). A movable-core linear variable differential transducer (LVDT) was utilized in TMA. LVDT generates an output signal directly proportional to specimen dimension change. Test specimen rests on a pedestrian, placed in contact with a flat tip expansion probe shown in

Figure 3.10 (b), which reads any linear or volumetric changes in the dimension at the test temperature. The probe and platform of TMA is made with material that has least thermal expansion coefficient to ensure recording of pure steel thermal expansion. TMA can be used to test most varied sample geometries in 20 to 1000$^{\circ}$C temperature range. Temperature precision can be maintained within 1$^{\circ}$C of target temperature.

(a) Thermomechanical analyzer





(b) Expansion probe (c) Test specimen

Figure 3.10 - Thermomechanical analyzer apparatus and test specimen for measuring thermal expansion

### 3.3.3 Test Procedure.

For thermal expansion measurements, the steel specimen was placed on a pedestal in the moveable furnace of the Themomechanical Analyzer. A flat tip expansion probe was set on the specimen. A small force of 0.02N magnitude was applied by the probe on the specimen in order to maintain static contact between the specimen and the probe. However, due to the small magnitude of this force the thermal expansion was not affected due to the presence of this force.

The temperature increase for thermal expansion test depends on the temperature ramp (heating rate) set by the user for that particular test. Once the specimen was placed in a position inside the TMA furnace, the test was run and controlled by software that records dimensional change continuously with increasing temperature. The specimen was heated up to 1000°C at a rate of 5°C/min and then cooled down to room temperature. The machine records initial length, and temperature. Figure 3.11 illustrates the temperature ramp used in thermal expansion tests. To ensure reproducibility of the test results, 5 tests of each steel type were conducted during heating and cooling phase.

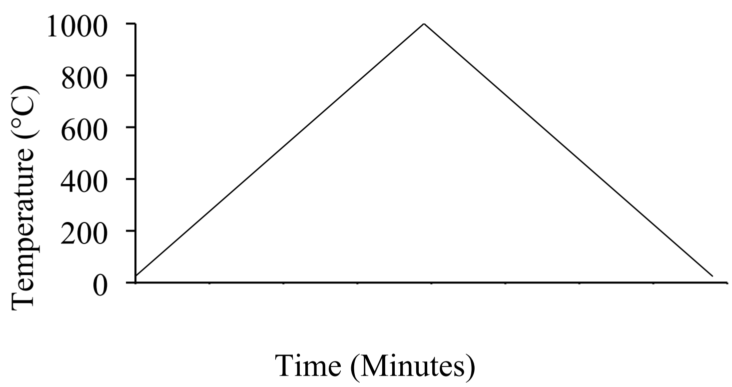



Figure 3.11 - Temperature ramp used for thermal expansion tests

### 3.3.4 Measured data – Thermal expansion

Thermal strain is obtained as percent change in length over original length of the specimen. Table 3.4 provides measured thermal strains of A36, A325 and A490 steel in the heating and cooling phase. Figure 3.12 shows the average measured thermal strain in heating and cooling phase. It is clearly seen from Figure 3.12 (a) that the thermal expansion of A490 steel increases linearly with increase in temperature up to 700°C. When heated beyond 700°C thermal strain of steel appears to be constant till 800°C and starts to increase again till 1000°C. At 723°C, A490 steel undergoes phase transition from ferrite to austenite. The temperature at which crystalline structure of steel changes to homogenous austenite depends on the amount of carbon present in steel. A typical phase change for 0.4% carbon content steel is shown in Figure 3.13, which illustrates the microstructural changes that steel undergoes when subjected to temperature variation. This phase change for A490 steel completes approximately at 800°C. During this phase change, energy absorbed by the steel is utilized for the microstructural changes in steel and hence the thermal expansion is observed to be constant. Once the crystalline structure of steel transforms completely into $\gamma$ austenite, the specimen starts expanding again. However test result for A36 and A325 steel do not show this trend. Figure 3.12 (b) illustrates average thermal strain recorded during cooling phase for A36, A325 and A490 steel. A small variation in thermal strain observed for the three-tested steel types can be attributed to the change in carbon content of these steels. It can be seen clearly from Figure 3.12 (c) that trend of thermal strain during cooling phase and heating phase do not coincide. This can be attributed to the irreversible changes that occur in microstructure of steel when it is subjected to heating and cooling cycle. If steel is held at 723°C, microstructure changes to homogeneous austenite. When it is slowly cooled down, pearlite is formed and entire structure changes to a lamellar structure with alternate layers of ferrite and cementite. These physiochemical changes take place at a finite rate, and as a result thermal expansion for the heating phase does not coincide with that of the cooling phase.

Table 3.4 – Recorded thermal expansion of A36, A325 and A490 steel in heating and cooling phase

(a) A36 steel

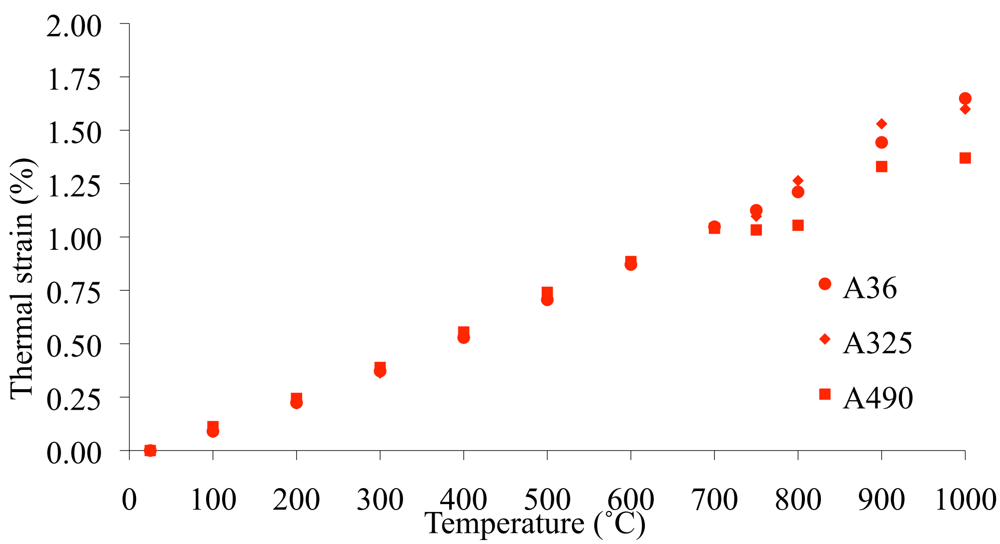
| Temperature (°C) | Thermal strain | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Heating phase | | | | | | Cooling phase | | | | | |
| | Test 1 | Test 2 | Test 3 | Test 4 | Test 5 | Average | Test 1 | Test 2 | Test 3 | Test 4 | Test 5 | Average |
| 25 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | - | 14.5 | 9.8 | 12.7 | 11.9 | - |
| 100 | 0.92 | 1.02 | 0.89 | 0.81 | 0.84 | 0.90 | 10.5 | 16.4 | 10. | 13.1 | 12.4 | 12.2 |
| 200 | 2.26 | 2.37 | 2.29 | 2.06 | 2.20 | 2.24 | 11.6 | 17.8 | 11.0 | 14.5 | 13.5 | 12.5 |
| 300 | 3.71 | 3.96 | 3.81 | 3.49 | 3.61 | 3.72 | 12.5 | 18.0 | 11.5 | 15.0 | 13.6 | 13.7 |
| 400 | 5.28 | 5.33 | 5.48 | 5.09 | 5.27 | 5.29 | 13.7 | 16.3 | 11.8 | 14.5 | 13.1 | 14.1 |
| 500 | 7.06 | 6.99 | 7.29 | 6.85 | 7.09 | 7.06 | 13.0 | 15.5 | 11.4 | 14.1 | 13.3 | 13.9 |
| 600 | 8.76 | 8.63 | 8.90 | 8.48 | 8.81 | 8.71 | 13.6 | 16.7 | 12.3 | 15.1 | 14.1 | 13.5 |
| 700 | 10.4 | 10.5 | 10.6 | 10.1 | 10.5 | 10.4 | 14.4 | 15.8 | 11.7 | 14.3 | 13.4 | 14.3 |
| 750 | 11.2 | 11.0 | 11.3 | 10. | 11.7 | 11.2 | 13.2 | 16.3 | 12.0 | 15.1 | 13.7 | 13.9 |
| 800 | 12.3 | 11.0 | 11.4 | 11.3 | 14.3 | 12.1 | 13.2 | 16.3 | 12.0 | 15.1 | 13.7 | 14.1 |
| 900 | 15.1 | 13.2 | 13.7 | 13.6 | 16.3 | 14.4 | 15.1 | 18.2 | 13.5 | 16.5 | 15.5 | 15.7 |
| 1000 | 17.5 | 15.3 | 14.8 | 17.7 | 16.9 | 16.4 | 17.5 | 15.3 | 14.8 | 17.7 | 16.9 | 16.4 |

(b) A325 steel

| Temperature (°C) | Thermal strain | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Heating phase | | | | | | Cooling phase | | | | | |
| | Test 1 | Test 2 | Test 3 | Test 4 | Test 5 | Average | Test 1 | Test 2 | Test 3 | Test 4 | Test 5 | Average |
| 25 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 13.5 | 11.3 | 13.7 | 18.7 | 13.0 | 12.7 |
| 100 | 0.81 | 0.74 | 0.80 | 0.84 | 1.08 | 1.12 | 14.2 | 10.7 | 13.1 | 18.0 | 14.0 | 13.1 |
| 200 | 2.34 | 1.96 | 2.02 | 2.21 | 2.44 | 2.44 | 14.9 | 10.0 | 13.8 | 17.5 | 14.5 | 13.3 |
| 300 | 3.78 | 3.33 | 3.43 | 3.71 | 3.92 | 3.89 | 15.3 | 8.84 | 14.3 | 16.5 | 14.5 | 13.6 |
| 400 | 5.41 | 4.73 | 5.11 | 5.35 | 5.72 | 5.55 | 14.3 | 7.84 | 14.9 | 16.1 | 15.0 | 13.8 |
| 500 | 7.26 | 6.44 | 6.94 | 7.23 | 7.56 | 7.41 | 13.6 | 7.65 | 14.3 | 15.2 | 14.7 | 13.1 |
| 600 | 9.03 | 8.02 | 8.68 | 9.05 | 9.18 | 8.85 | 12.8 | 7.44 | 15.3 | 13.8 | 14.3 | 12.2 |
| 700 | 10.7 | 9.62 | 10.2 | 10.9 | 10.5 | 10.4 | 12.1 | 7.34 | 16.4 | 13.1 | 14.1 | 11.0 |
| 750 | 11.2 | 9.54 | 11.4 | 11.6 | 10.8 | 10.3 | 12.6 | 8.05 | 16.2 | 13.4 | 14.4 | 10.7 |
| 800 | 12.2 | 9.78 | 15.0 | 13.6 | 12.4 | 10.5 | 13.4 | 8.74 | 16.9 | 14.0 | 14.5 | 11.0 |
| 900 | 16.7 | 11.6 | 15.8 | 16.5 | 15.6 | 13.3 | 15.2 | 10.1 | 17.0 | 15.7 | 15.8 | 12.1 |
| 1000 | 16.7 | 11.4 | 17.2 | 17.2 | 17.2 | 13.7 | 16.7 | 11.4 | 17.2 | 17.2 | 17.2 | 12.9 |

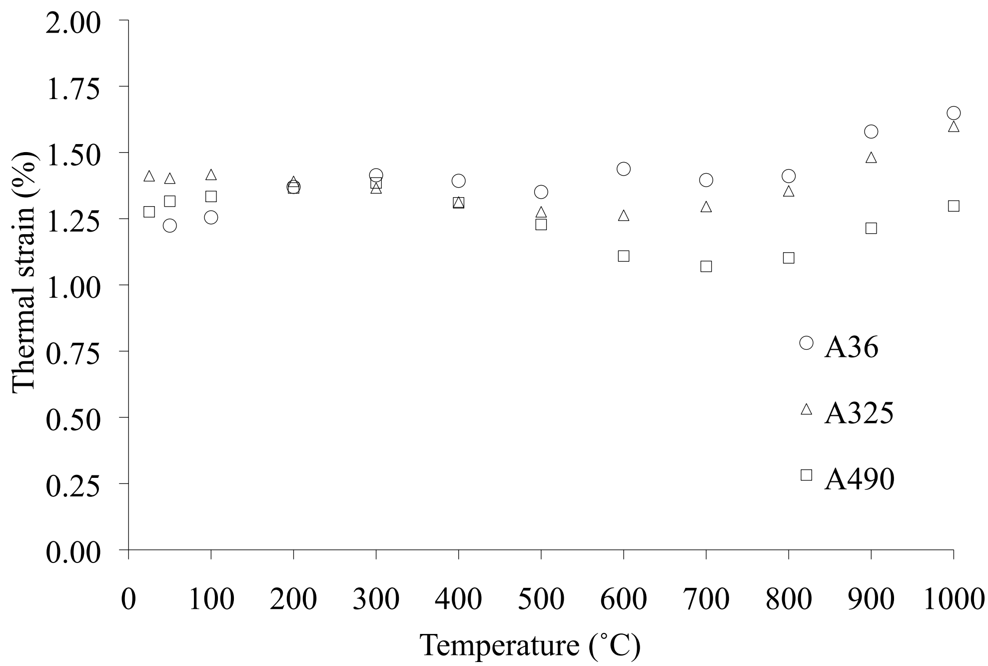
Table 3.4 (cont'd) – Recorded thermal expansion of A36, A325 and A490 steel in heating and cooling
(c) A490 steel

| Temperature (˚C) | Thermal strain | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Heating phase | | | | | | Cooling phase | | | | | |
| | Test 1 | Test 2 | Test 3 | Test 4 | Test 5 | Average | Test 1 | Test 2 | Test 3 | Test 4 | Test 5 | Average |
| 25 | 0.00 | 0.00 | 0.0 | 0.00 | 0.00 | 0.00 | 9.74 | 15.6 | 15.3 | 10.6 | 12.4 | 14.1 |
| 100 | 0.88 | 1.43 | 1.5 | 0.96 | 0.79 | 0.85 | 10.5 | 15.6 | 14.4 | 12.6 | 12.5 | 14.0 |
| 200 | 2.14 | 2.89 | 2.8 | 2.33 | 2.04 | 2.19 | 11.4 | 15.0 | 14.0 | 13.8 | 12.3 | 14.1 |
| 300 | 3.52 | 4.39 | 4.3 | 3.76 | 3.44 | 3.63 | 12.3 | 15.3 | 13.9 | 14.5 | 12.0 | 13.9 |
| 400 | 5.14 | 6.12 | 6.0 | 5.41 | 5.04 | 5.26 | 13.1 | 15.5 | 13.7 | 15.4 | 11.3 | 13.6 |
| 500 | 6.81 | 7.82 | 8.5 | 7.15 | 6.74 | 7.09 | 12.6 | 13.8 | 14.2 | 15.6 | 8.98 | 13.1 |
| 600 | 8.46 | 9.38 | 9.2 | 8.79 | 8.37 | 8.79 | 12.2 | 12.5 | 13.8 | 15.4 | 7.29 | 12.7 |
| 700 | 9.98 | 11.3 | 10.3 | 10.4 | 10.0 | 10.4 | 12.7 | 9.65 | 10.7 | 15.5 | 6.71 | 12.6 |
| 750 | 10.4 | 10.7 | 9.0 | 10.6 | 10.7 | 10.9 | 13.0 | 9.3 | 9.31 | 14.6 | 7.15 | 12.9 |
| 800 | 9.76 | 12.0 | 12.2 | 9.53 | 9.17 | 12.6 | 13.4 | 9.5 | 9.28 | 14.6 | 8.10 | 13.5 |
| 900 | 14.6 | 13.1 | 13.8 | 13.2 | 11.5 | 15.3 | 14.1 | 10.9 | 10.8 | 14.6 | 10.1 | 14.8 |
| 1000 | 15.7 | 12.7 | 12.6 | 15.7 | 11.6 | 15.9 | 15.7 | 12.7 | 12.6 | 15.7 | 11.6 | 15.9 |

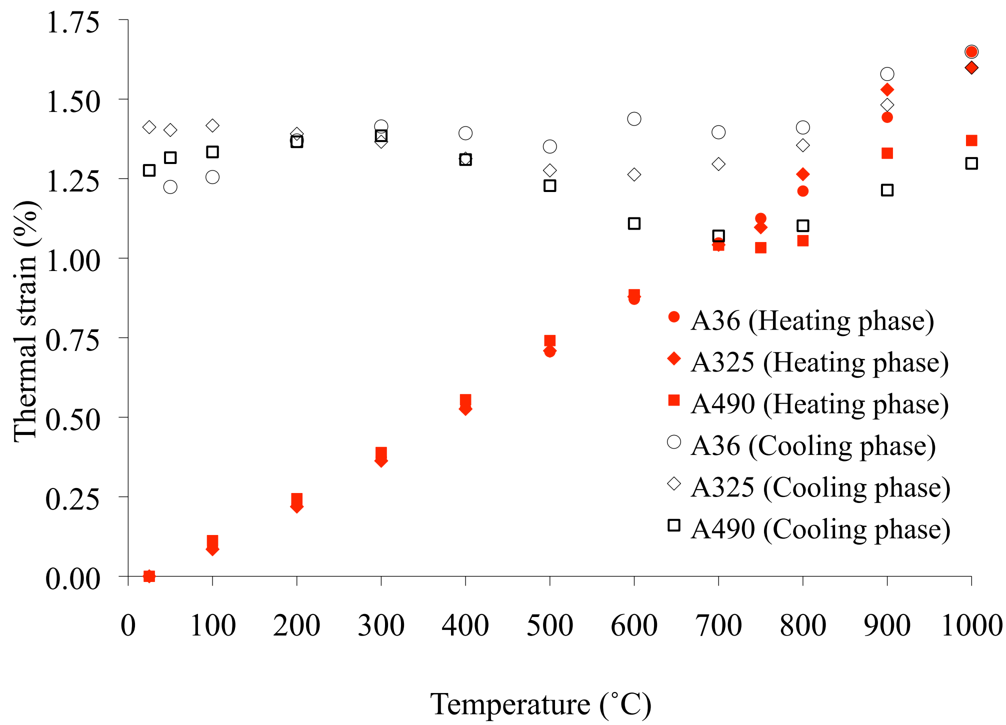



(a) Heating phase

Figure 3.12 – Measured thermal strain of A36, A325 and A490 steel in heating and cooling phases

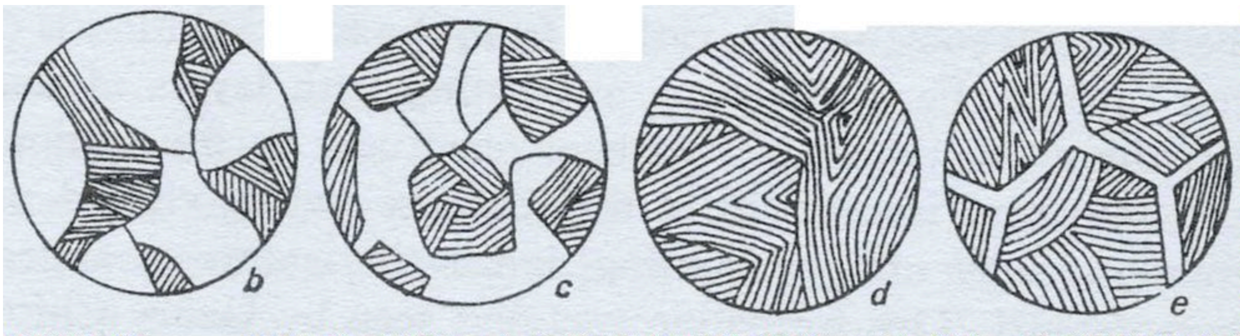



(b) Cooling phase




(c) Comparion in heating and cooling phases

Figure 3.12 (cont'd) – Measured thermal strain of A36, A325 and A490 steel in heating and cooling phases




(a) T<500°C (b) 500°C<T<700°C (c) 700°C<T<900°C (d) T>900°C

Figure 3.13 – Phase diagram of 0.4% carbon steel (effect of temperature on microstructure)

### 3.3.5 Comparison of test data with published data

Figure 3.14 shows comparison of measured thermal expansion with that from ASCE and Eurocode provisions and also from different published sources. It can be seen that measured thermal strains compare well with the ASCE and Eurocode provisions and also with other published sources. Minimal difference exists between the Eurocode and ASCE models for thermal strain of steel up to 700°C. However, in the temperature range of 750°C to 850°C, the ASCE model assumes a continuous increase in thermal strain, while the Eurocode model assumes a constant thermal strain followed by an increasing thermal strain up to 1000°C. This constant thermal expansion accounts for the phase change that occurs in steel when exposed to temperature change. Depending on the carbon content of steel, this temperature required for phase change varies. This temperature ranges from 900°C for 0% carbon steel to 723°C for 0.8% carbon steel. Structural steel falls within this specified range of carbon content. Hence Eurocode assumes constant value of thermal expansion for the range of 750°C to 900°C. Although A490 steel shows trends similar to Eurocode provisions, the constant thermal expansion for A490 steel is observed in the temperature range of 700-800°C.

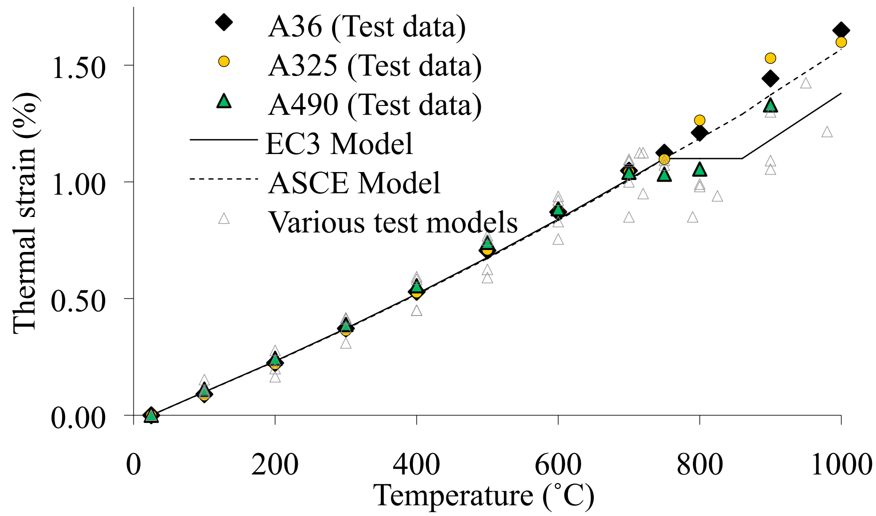



Figure 3.14 – Comparison of thermal strain with different models and published data

**3.4 High temperature thermal property relations**

Data generated from the thermal property measurements was utilized to develop thermal property relationships for A36, A325 and A490 steel. These properties are expressed in the form of empirical relationships over temperature range of 20-735°C for thermal conductivity and specific heat and 20-1000°C for thermal expansion. These empirical relationships were arrived at based on regression analysis. For the regression analysis, measured thermal properties were used as response parameter with temperature as their predictor parameter. The accuracy of the statistical model is indicated by coefficient of determination '$R^2$', that represents proportion of the sum of squares of deviations of the response values about their predictor (Mendenhall and

Sincich 2007). $R^2$ values for the proposed equation lie within range of 0.92 to 1. As $R^2$ value of 1 represents a perfect fit equation, based on the $R^2$ the proposed equations can be considered best fit for the data obtained from the test results.

Comparison of measured data with Eurocode or ASCE guidelines for thermal properties shows that ASCE and Eurocode provisions provide good representation for A36 steel. However, A325 and A490 bolt steel with different chemical composition shows different thermal properties than that of A36 steel. Hence following equations are proposed for A490 and A325 steel.

### 3.4.1 Thermal conductivity

Test result for A36, A325 and A490 steel tabulated in Table 3.2 shows linear variation of thermal conductivity with temperature, hence linear regression analysis was performed to find best fit curve for the test results. As seen from the test result that the thermal conductivity of steel measured during heating phase and cooling phase coincides, thus the proposed equation for A325 and A490 steel is applicable for both heating and cooling phase. The proposed equations for thermal conductivity of A325 and A490 steel are;

**A325 steel**

$$k_t = 50 - 0.027T \qquad 20^{\circ}C \leq T \leq 735^{\circ}C$$

**A490 steel**

$$k_t = 46 - 0.023T \qquad 20^{\circ}C \leq T \leq 735^{\circ}C$$

Thermal conductivity predicted by above equations is compared with the EC3 and ASCE 1992 provisions in Figure 3.15. Comparison of the two shows that A490 steel possesses lesser thermal conductivity than A36 and A325 steel because of higher carbon content.

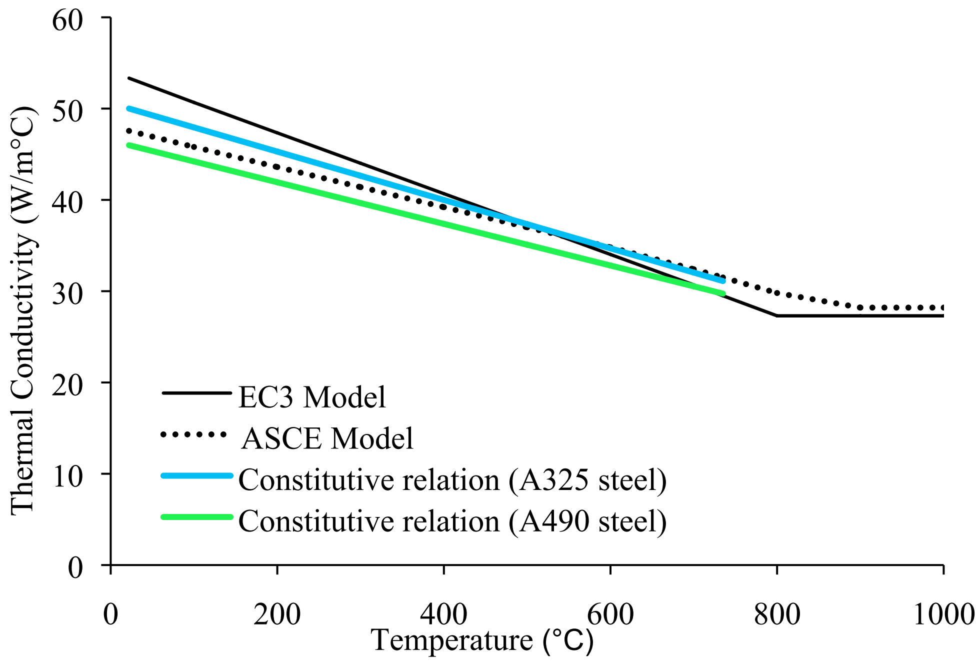



Figure 3.15 – Comparison of proposed thermal conductivity relation with ASCE and EC3 provisions for A325 and A490 steel

### 3.4.2 Specific heat

Test result for A36, A325 and A490 steel tabulated in Figure 3.6 shows that polynomial regression analysis was performed to find best-fit curve. As seen from the test result that the specific heat of steel measured during heating phase and cooling phase coincide, thus the proposed equation for A325 and A490 steel is applicable for both heating and cooling phase. For all three types of steel two relations are proposed for the temperature range of 25 to 600$^{\circ}$C and from 600 to 735$^{\circ}$C. The proposed equations for specific heat of A325 and A490 steel are;

**A325 steel**

$= 490 - 0.19T + 0.0014T^2$ $\quad 20^\circ C \leq T \leq 600^\circ C$

$= 20.79T - 0.0118\,T^2 - 7350$ $\quad 600^\circ C < T \leq 735^\circ C$

**A490 steel**

$= 0.001T^2 - 0.39T + 490$ $\quad 20^\circ C \leq T \leq 600^\circ C$

$= 15.274T - 9438.8$ $\quad 600^\circ C < T \leq 735^\circ C$

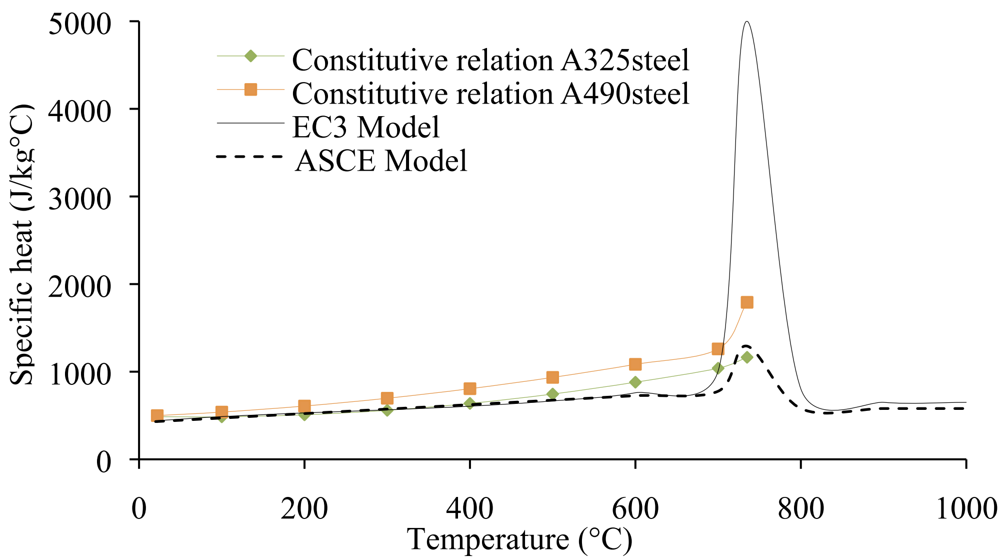



Figure 3.16 – Comparison of proposed specific heat equations for A36, A325 and A490 steel with ASCE and EC3 provisions

Figure 3.16 shows comparison of high temperature property relation proposed for A36, A325 and A490 steel with ASCE (1992) and EC3 (2005) guidelines. It can be seen that the specific heat of different steel and code guidelines are in great agreement till temperature up to 400°C (curie point). However after 400°C, effect of carbon content of different steels on the values of specific heat is visible. This effect is not considered in ASCE and Eurocde guidelines and hence these guidelines are not conservative. If used in thermal analysis these guidelines may produce result indicating higher temperature in steel.

### 3.4.3 Thermal strain

Test result for A36, A325 and A490 steel tabulated in Table 3.4 shows linear variation of thermal strain with temperature, hence linear regression analysis was performed to find best fit curve for the test results. As seen from the test result, the thermal strain of steel measured during heating phase and cooling phase does not coincide, thus the separate equations are proposed for heating and cooling phase. The proposed equations for thermal strain of A325 and A490 steel are;

**Heating phase**

**A325 steel**

$$\varepsilon_{th} = 0.00145T - 0.03825 \qquad 20^{\circ}C \le T \le 1000^{\circ}C$$

**A490 steel**

$$\varepsilon_{th} = 0.0016T - 0.052 \qquad 20^{\circ}C \le T \le 700^{\circ}C$$

$$\varepsilon_{th} = 1.05 \qquad 700^{\circ}C < T \le 800^{\circ}C$$

$$\varepsilon_{th} = 0.00135T - 0.17 \qquad 800^{\circ}C < T \le 1000^{\circ}C$$

Comparison of proposed equations and code provisions shows great agreement in the thermal strain value of different steels till 700$^{\circ}$C. However, as steel undergoes phase change when heated beyond 700$^{\circ}$C, thermal expansion of different steel is not consistent. A490 steel follows the Eurocode trend however the constant thermal strain due to phase change in A490 steel is observed to be appearing at earlier stage than that of Eurocode provision. Thermal strain for A36 and A325 steel falls in the range of ASCE and Eurocode guidelines.

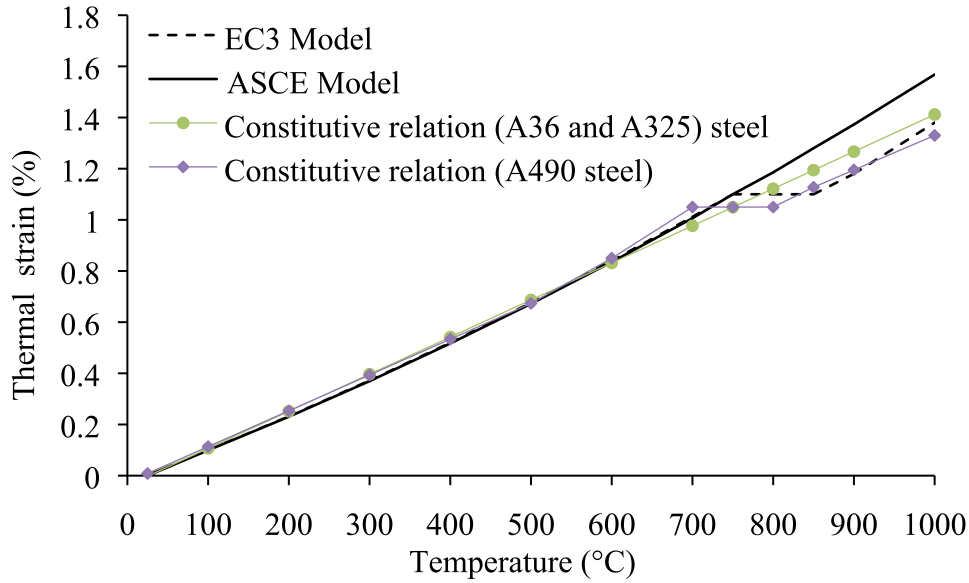



Figure 3.17 - Comparison of proposed thermal expansion equations for A325 and A490 steel with ASCE and EC3 provisions (heating phase)

Cooling phase test result indicate that for a particular test steel the values recorded at different temperature in the temperature range of 25 to 1000$^{\circ}$C, fall in the range of ±20% of the proposed value. Hence for ease of application in engineering use thermal strain is assumed to be a constant value for the cooling phase as shown in Figure 3.17.

**Cooling phase**

**A36**

$$\varepsilon_{th} = 1.45 \qquad 20^{\circ}C \leq T \leq 1000^{\circ}C$$

**A325 steel**

$$\varepsilon_{th} = 1.38 \qquad 20^{\circ}C \leq T \leq 1000^{\circ}C$$

**A490 steel**

$\varepsilon_{th} = 1.25$ $\qquad$ $20^{\circ}C \leq T \leq 1000^{\circ}C$

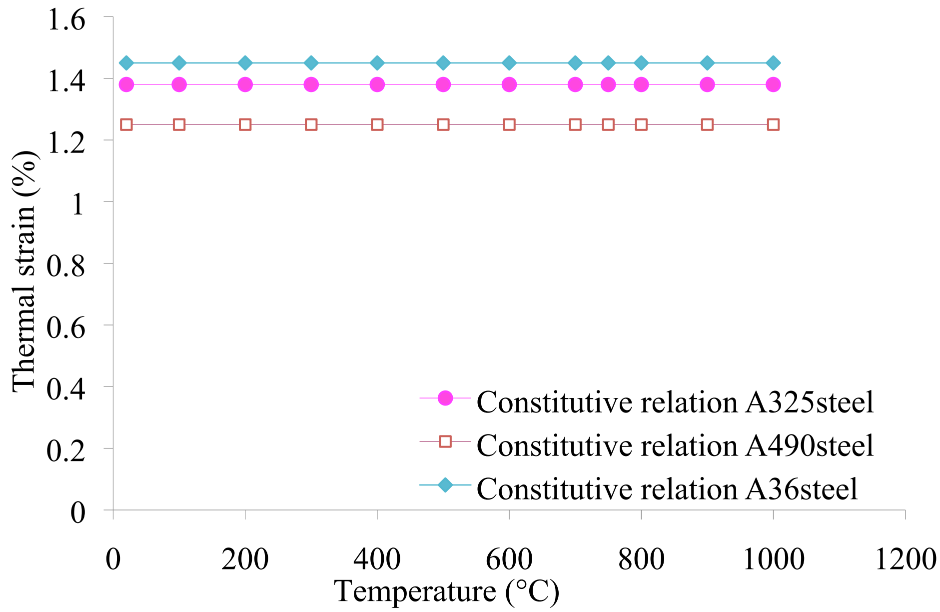



Figure 3.18 - Proposed thermal expansion equations for A36, A325 and A490 steel (cooling phase)

### 3.5 Summary

Tests were performed to measure thermal properties of steel at elevated temperature. Temperature dependence of thermal conductivity, specific heat and thermal expansion of different types of steel is determined. High temperature relations are proposed to reflect temperature dependence of thermal properties steel.

From the test results it can be said that thermal properties of steel at elevated temperature depends on the chemical constitution and phase changes that steel goes through when subjected to temperature variation. Thermal conductivity decreases with rise in temperature, whereas specific heat increases with increase in temperature. Thermal conductivity and specific heat of A36, A325 and A490 steel for both heating and cooling phase are observed to be same.

Generally, while performing thermal analysis thermal conductivity and specific heat of steel is assumed to be the same for heating and cooling phase. This assumption is justified by the test results. Thermal strain of steel increases with temperature. The test result indicates that the thermal strain for heating and cooling phase is different. Hence result produced in this study can be used as a reference for thermal strain while performing thermal analysis of structural members exposed to design fire with cooling phase. Also test results indicate the influence of composition, particularly % carbon content of steel on its thermal properties. Hence assuming thermal properties same for different steel types may lead to conservative or unrealistic temperatures in steel. Comparison of proposed high temperature property relationship with the code and standard provision indicates the conservative nature of the provisions.

# Chapter 4

## High temperature strength properties

### 4.1 General

The mechanical properties of steel that are of interest in the structural design are strength, stiffness, and toughness. Steel possesses excellent stiffness and toughness at ambient temperature. However, these properties in steel degrade at a faster rate at elevated temperatures as compared to concrete. Most previous studies were aimed at characterizing high temperature strength and stiffness properties of conventional structural steel and very little information is available on high strength steel used in bolts. The high strength properties of steel used in bolts are achieved through martensite crystalline structure by quenching and tempering of the steel with a designed chemical composition. Martensite is an unstable crystalline structure and is subjected to change with variation in temperature. Hence strength properties of these bolts can vary significantly depending on the temperatures achieved in the connections.

In order to evaluate the temperature dependence of mechanical properties of bolts an extensive experimental study was undertaken and the test procedures and results are discussed in detail in this chapter.

### 4.2 High temperature mechanical properties

The mechanical properties of steel can be gauged through stress-strain relations and typical stress-strain curve for conventional steel as shown in Figure 4.1. The maximum stress that steel can withstand before necking when subjected to the tensile load is the ultimate tensile strength of steel. The stress up to which the stress-strain curve is linear represents the yield strength of steel and the slope of this linear stress strain curve is known as elastic modulus. Shear strength of steel is the maximum stress steel can withstand when subjected to shear force. Effect of temperature on these four properties of steel is important for the design of bolted connections.

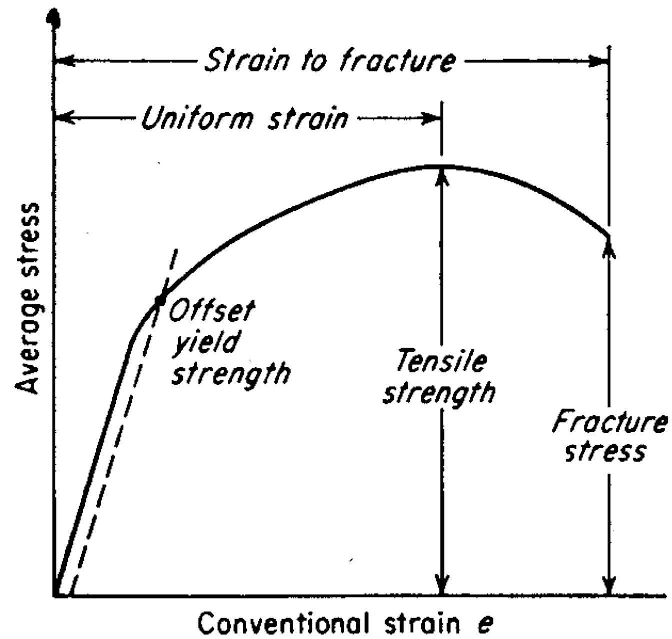



Figure 4.1 - Stress-strain curve for high strength steel

### 4.3 Test method

Past experimental work and review of literature describes that high temperature test procedure has great influence on test results. There are 2 types of test methods for evaluating high temperature strength properties of structural steel. In reality, when a structure is exposed to fire it is subjected to transient loading which depends on the applied temperature sequence. Hence transient test method may lead to realistic test results. In transient test, the specimen is subjected to constant tensile load determined for a selected stress level. Temperature is increased gradually to determine when failure strain in specimen will occur. Result from transient tests include mechanical, thermal and creep strain. To determine pure mechanical properties of material, thermal expansion can be easily separated however determining creep is a complex process. Generally it is required to perform additional creep tests in order to determine high temperature creep. Hence due to complex nature of transient test, steady state test is preferred over this test.

In steady state tests, the specimen is heated up to a specified temperature then strain-controlled load is applied on the specimen until failure while maintaining same temperature. Due to quick nature of the test, temperature induced creep does not significantly influence the results in these tests. Thus the recorded strain includes thermal and mechanical strain. In order to obtain pure mechanical strain, thermal strain can easily be deducted from the total strain.

Study of high temperature mechanical properties of bolts presented in this chapter are based on steady state tests performed on A325 and A490 bolts for temperature range of 25 to 800°C.

### 4.4 Development of test equipment

High temperature tension and single shear tests were performed to develop strength and stiffness data of Grade A325 and A490 steel bolts in 20 to 800°C temperature range. These tests required heating of the test specimen to a specified temperature followed by mechanical loading. Available commercial strength testing machines lack the heating facility and are suspect to damage at high temperature. Hence a special testing setup was designed and fabricated for undertaking high temperature strength tests. This test setup was designed so as to undertake tension, compression and shear tests on steel or any other construction material in 20 to 1000°C temperature range. Detailed operation and instrumentation of the test setup is discussed below.

### 4.4.1 Detailed operation of the test setup

The setup consists of two steel beams separated vertically at a distance 762mm by hydraulic jacks (schematic of the test setup shown in Figure 4.2). The beams were custom made W shaped 254mm wide and 254mm deep I sections with two webs of 12.7mm thickness each, A1090 steel beams. Bottom beam (B1) was fixed on strong floor of 609.59mm x 609.59mm grid with 660kN capacity tie-down rods. The top beam (B2) was supported by the hydraulic jacks and was free to move up and down. The hydraulic jacks separating these beams were connected to pneumatic pump, and were tied down to the bottom beam (B1) with the help of tension bolts. A calibrated threaded rod, which passes through the hydraulic jack, was connected to the top beam (B2). This calibrated rod controls the movement of top beam with the movement of the hydraulic jack. The specimen to be tested was connected to the loading frame with the help of anchorage plates. As the hydraulic jack pushes the top frame beam up with the help of the calibrated rod, displacement controlled loading was applied on the test specimen. This test specimen was surrounded by the electric furnace, which increases the temperature of the assembly to the specified test temperature.

Temperature of the furnace and the test specimen was monitored with the K type thermocouple. Load applied on the specimen and the deflection was recoded using the load cells and the strain pod mounted on the test setup.

During the test, temperature of the test specimen was increased to the specified test temperature Once the test temperature was reached the assembly was stabilized for 15 minutes to ensure uniform temperature throughout the test specimen. After stabilization, hydraulic jack applied displacement-controlled load on the specimen until its failure. Load applied on the specimen and its deflection was recorded. This recorded temperature, load and deflection of the specimen was monitored with the help of data acquisition system (ref Section 4.4.1.5) connected to the computer. Details of the instrumentation of the test setup are presented in the following section.

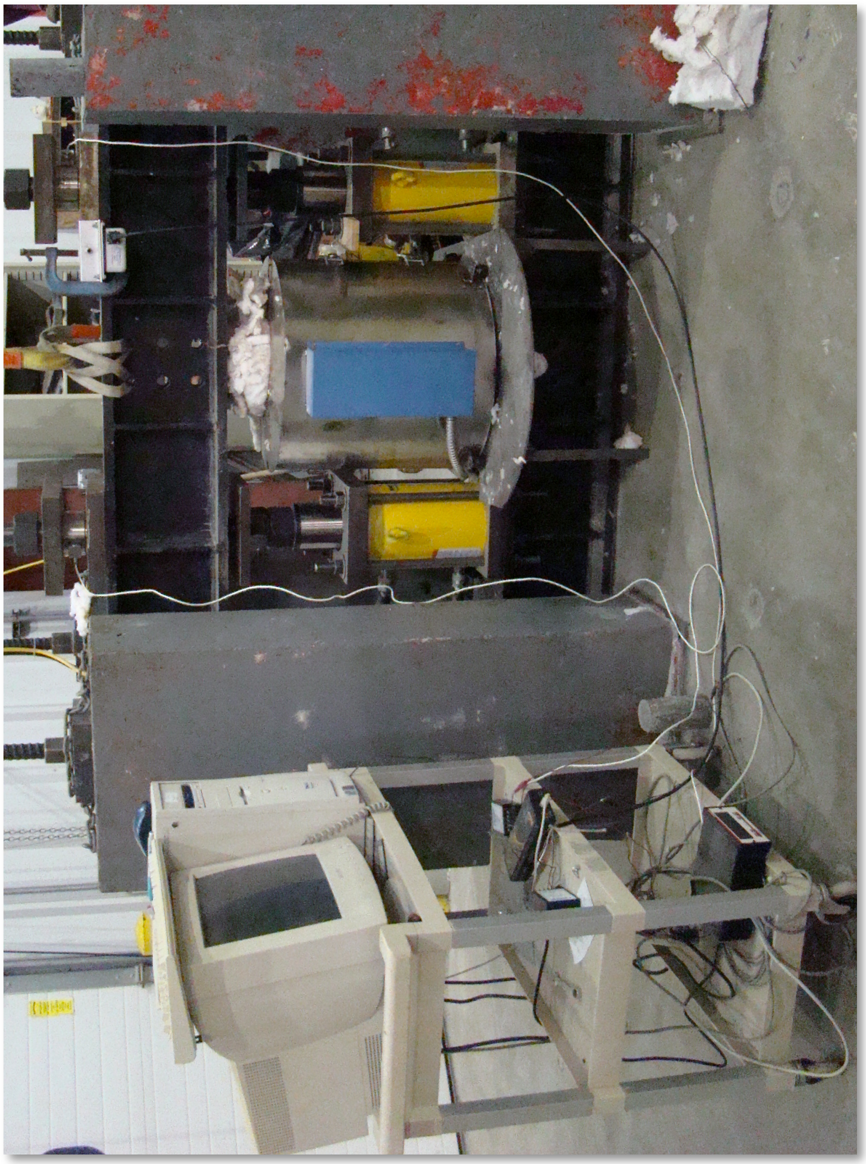


Figure 4.2 –Test setup to undertake tension and shear tests on bolts at elevated temperature

### 4.4.2 Electric furnace

Paragon KL 10DX9H FBR TUBE custom designed electric furnace (Figure 4.3) was acquired for generating high temperatures in test specimen. The cylindrical furnace has an outer diameter of 533.4mm and a height of 533.4mm with 203.2 openings at the top and the bottom. The heating chamber was 254mm in diameter and 254mm in height and was divided into three chambers of 76.19mm, 101.6mm and 76.19mm in height. The furnace can be used to generate a maximum temperature of 1000°C. The accuracy of the air temperature inside furnace was controlled to ±10°C of the larger temperature.

### 4.4.3 Hydraulic jack and pump

RRH1006 D/A Holl-O-cylinder hydraulic jack as shown in Figure 4.4 (a), with capacity of 100 ton was used. The hydraulic jacks were controlled by ZE3840MB electric hydraulic pump as shown in Figure 4.4 (b). Two hydraulic jacks of 152.4mm stroke were used to achieve the maximum loading capacity of 890kN. Threads provided on the collar of the jack provided a fixture for the calibrated rod that supports and connected top beam with the hydraulic jacks. Double acting operation of the hollow plunger of the jacks allow for both pull and push action, to perform tension and compression test with the same setup.

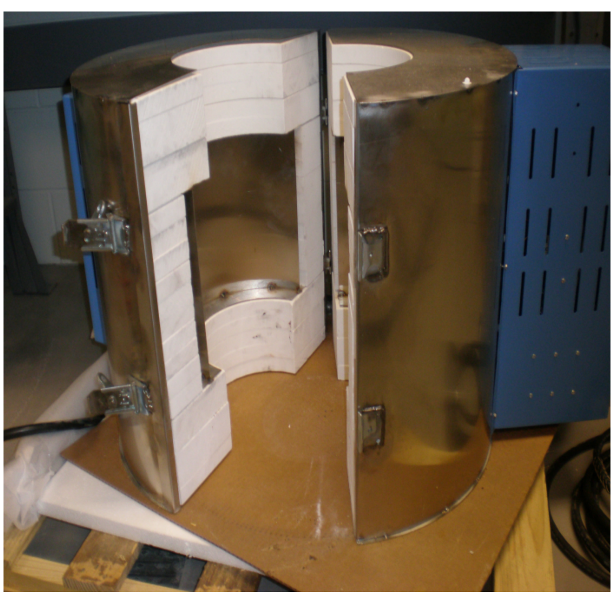
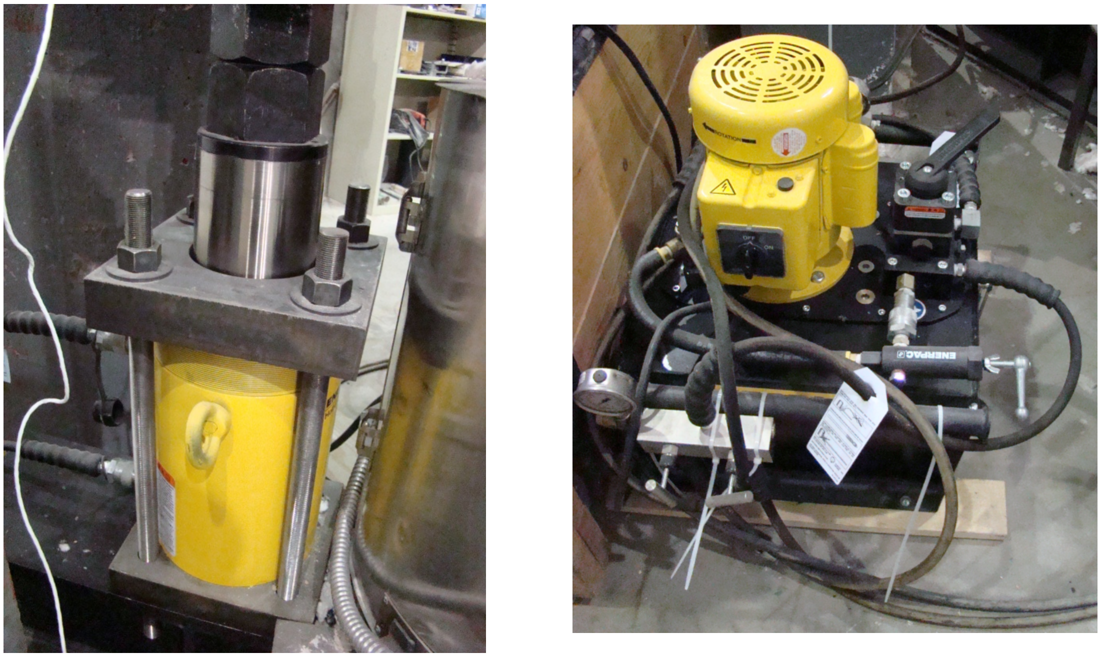


Figure 4.3 - Electric furnace for generating high temperatures in the test specimen



(a) Hydraulic Jack (b) Electronically operated hydraulic pump

Figure 4.4 – Hydraulic jack and electric pump utilized for applying strained controlled load on the test specimen

### 4.4.4 Anchorage plates and insulation

The beams of the loading frame were separated by a distance equal to 762mm to accommodate electric furnace and hydraulic jack. Hence three different anchorage plates were designed as per requirement of tension and shear tests to connect the test specimen to the loading frame. These anchorages were made up of S321. Use of carbon steel was avoided due to its high thermal conductivity and significant deterioration of material properties at elevated temperature. Also an insulation board of 8479K14 alumina silicate ceramic sheet, 12.7mm thick was placed between the anchorage plates. Low thermal conductivity of S321 steel and insulation was provided to maintain the temperature of the loading frame at a low level as compared to the test specimen. The temperature of the loading frame was observed to be less than $100^{\circ}$C during all the tests.

#### 4.4.4.1 Anchorage plate

Two stainless steel S321 plates of size 171.45mm x 508mm and 38.09mm thickness as shown in Figure-4.4 (a) project out from top and bottom beam of the loading frame. These plates were provided with 6 bolt holes of 30.15mm dia. to connect to the web of the beams. The projected portion of these plates was provided with two bolt holes of 36.5mm dia. to connect to tension or shear anchorage plates for tension or shear test.

#### 4.4.4.2 Tension anchorage

Tension anchorage as shown in Figure 4.4 (b) was made of two L-shaped S321 stainless steel plates of size 101.6mm x 260.35mm and 25.4mm thick were welded together to make a U shaped block of 101.6mm x 139.7mm x 260.35mm with central opening of 22.35mm diameter. During tension tests, tension anchorage was attached to the loading frame with the help of the anchorage plates (4.4.4.1) and test bolt runs through the central opening of these two tension anchorages. Head and nut of the test bolt was insulated so as to avoid direct contact of heated test specimen with tension anchorage in order to maintain minimum temperature in the anchorage plates.

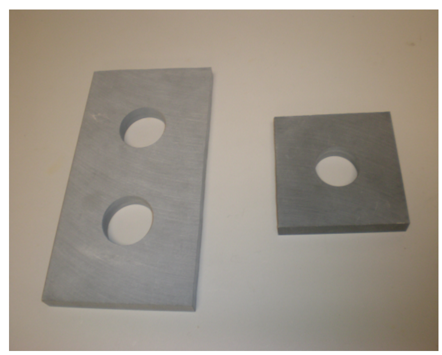
#### 4.4.4.3 Shear anchorage

"Y" shaped shear anchorage plates shown in Figure 4.5 (c) made of S321 stainless steel with a thickness of 57.15mm was used as anchorage plate. The thickness of the plate was designed so as to facilitate pure shear failure of the bolt and bearing of the plate was negligible. A bolt runs thought the bolt hole at the center of the leg of the two y-shaped shear anchorage plates connected to the loading frame and the shear force was applied on the shank of the bolts.

(a) Anchorage plate connected to the loading frame to facilitate the connection for shear and tension anchorage plates

(b) Tension anchorage plate to perform tension test

(c) Y-shaped shear anchorage plate to perform shear test



(d) Ceramic insulation board to insulate anchorage plates

Figure 4.5 – Various components of the mechanical testing apparatus

### 4.4.5 Data acquisition system

The data acquisition system consisted of personal Daq/3000 Series USB 1-MHz, a 16-Bit multifunction module (see figure 4.6). 8 differential or 16 single ended analog inputs were available for the thermocouple, load cell, and strain pod inputs. The thermocouple and load cell from the test setup were connected to the differential digital input, which reads the input signal in volts. The strain pod used to record deflection during test was connected to the analog input. This data acquisition system was connected to a computer that records and save the test data with the help of software.

Figure 4.6 - Data acquisition system ‘Personal Daq/3000 Series USB 1-MHz

**4.4.6 Measured response parameters**

K type thermocouple connected to the data acquisition monitored by computer software was used to record the temperature of the test specimen and the furnace. The position of thermocouple was determined as per the tension and shear test requirement.

Donut shaped compression type load cells (CTH930) manufactured by Stellar Technology were used. Capacity of the each load cell was 200kip. It has an outer diameter of 114.3mm, inner diameter of 76.19mm and height of 63.5mm. The load cells were located on the calibrated rods passing through the hydraulic jacks and were sandwiched between the top beam and bearing plate. With the movement of the top beam, the load cells were compressed and the load data was recorded. Load was recorded with accuracy of ±0.5% BFSL Nominal. The load cells are shown in Figure 4.7.

A strain pod was used to record the displacement during the mechanical tests. This strain pod (EP -10 L3M) was an EP-series strain pod made by Unimeasure. It has two channel outputs, which utilizes an incremental encoder as the sensor, and provides a quadrature square wave output. It measures displacement as a number of rotations the strain pod shaft. For each 25.4mm deformation strain reads 2000 counts. Strain pod has range of 254mm with linearity of 0.03% of the full scale. Strain pod located on the top beam of the loading frame that was connected to the data acquisition system.

Figure 4.7 – Donut shaped load cells to record load during the mechanical tests

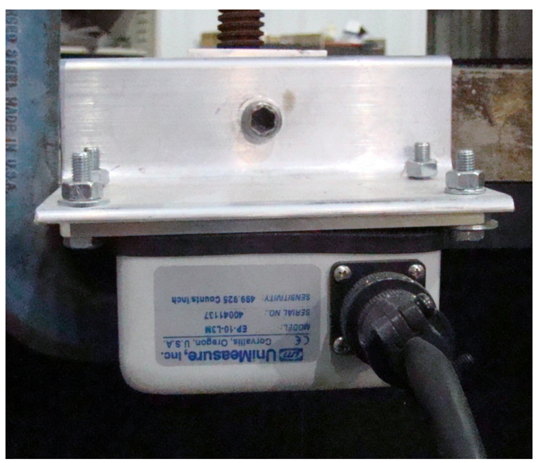


Figure 4.8 – Strain pod mounted to record deflection during the mechanical tests

### 4.5 Tension tests

A series of steady state tension tests were conducted on A325 and A490 bolts for the temperature range of 25 to 800$^\circ$C. A description of the test procedure and results are presented below.

### 4.5.1 Test specimens

For tension tests the specimens were round in shape and fabricated as per ASTM standard A370–05 specification from commercially available 22mm diameter 240mm long A325 and A490 bolts. In order to localize the zone of failure and ensure uniform stress over the cross section, 22mm diameter (D) bolts were machined to a reduced cross section of 15mm diameter (d) at mid height for gauge length (G) of 50mm. Total length of reduced cross section (A) was adapted to be 75mm as shown in Figure 4.1. It is desirable to ensure no stress-concentration along the length of the test specimen due to reduction in the cross section. This is provided for by the taper in the length of the test specimen as shown in Figure 4.9.

Figure 4.10 shows A325 and A490 bolts test specimen supplied as per the ASTM325/325M-04b and ASTM490/490M-04a standard specifications. Grade ASTM F436 washers and ASTM A563 nuts were used for the test. Mechanical properties, and chemical composition of test specimen are given in Table 4.1.

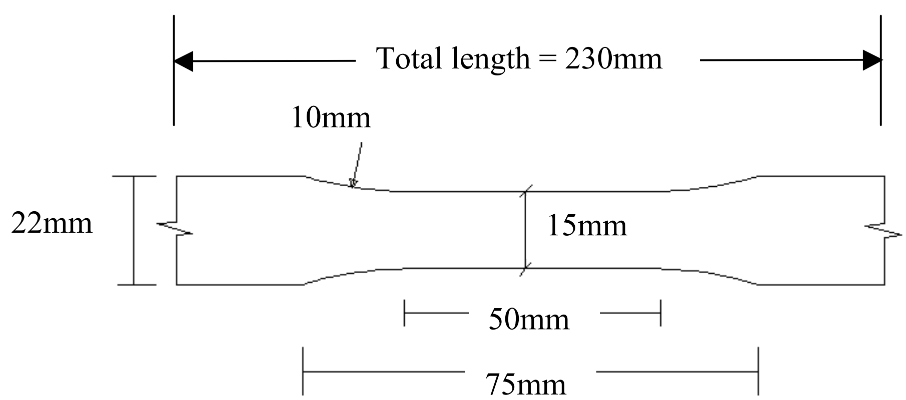



Figure 4.9 – Schematic of test specimen used for high temperature tension tests

Each test specimen was designated as per the grade and the test temperature. For example a test specimen designated as ‘A325T200’ represents A325 bolt subjected to tensile loading at 200°C.

(a) Grade A325 test specimen

(b) Grade A490 test specimen

Figure 4.10 – Illustration of Grade A325 and A490 bolt specimens used for tension tests

Table 4.1 - Chemical composition and mechanical properties of A325 and A490 bolts

(a) Grade A325 steel: ASTM A325/A325M -04(b) specification

| Chemical composition | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| C | Si | Mn | P | S | Cr | Mo | B | Cu |
| 0.25 | 0.17 | 0.6 | 0.02 | 0.05 | ---- | 0.025 | <0.0005 | ---- |

| Mechanical properties | | | | Geometry | | |
|---|---|---|---|---|---|---|
| Tensile strength (MPa) | Yield strength (MPa) | % elongation | % Reduction in area | Diameter (mm) | Length (mm) | Thread length (mm) |
| 830 | 92 | 630 | 35 | 22 | 150 | 40 (9 thread/25mm) |

(b) Grade A490 steel: ASTM A490/A490M -04(a) specification

| Chemical composition | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| C | Si | Mn | P | S | Cr | Mo | B | Cu |
| 0.38 | 0.27 | 0.8 | 0.04 | 0.04 | 0.12 | 0.2 | <0.0005 | 0.04 |

| Mechanical properties | | | | Geometry | | |
|---|---|---|---|---|---|---|
| Tensile strength (MPa) | Yield strength (MPa) | % elongation | Reduction in area | Diameter (in) | Length (in) | Thread length (in) |
| 1035 | 850 | 14 | 40 | 22 | 150 | 40 (9 thread/25mm) |

#### 4.5.2 Test procedure

Steady state tension tests were performed at seven temperature points; 20, 250, 400, 500, 600, 700 and 800°C on A325 and A490 test specimens. During each test, the specimen was held between the tension anchorage plates connected to top and bottom beams of the loading frame. As shown in Figure 4.11, only 4" test length of the bolt was subjected to heating and also direct contact of the specimen with tension anchorages was avoided by insulating the head and nut of specimen in order to maintain temperature in the loading frame to minimum. Tensile strength of steel at room temperature was evaluated by following ASTM E8 test procedure (ASTM standard E8 2009). In the absence of specific standardized test method at high temperature, ASTM room temperature test procedure was applied to evaluate high temperature tensile strength of steel. In these high temperature tests, the specimen was exposed to specific target temperatures of 250, 400, 500, 600, 700 and 800°C at a heating rate of 5°C per minute. Once the specimen was heated to test temperature, it was stabilized for 15 minutes to ensure steady state was attained (uniform temperature distribution) throughout the specimen. After achieving steady state conditions, deflection controlled quasistatic load was applied gradually on the specimen till failure. The load and deflection in the specimen was recorded at various time intervals with the help of load cell and strain pod mounted on the loading frame. Two K type thermocouples, one mounted on the test specimen and other inserted in the furnace recorded the specimen temperature and air temperature inside the furnace respectively. In most of the tests the fluctuation was recorded temperature during loading was observed to be in +5 to -5 °C range.

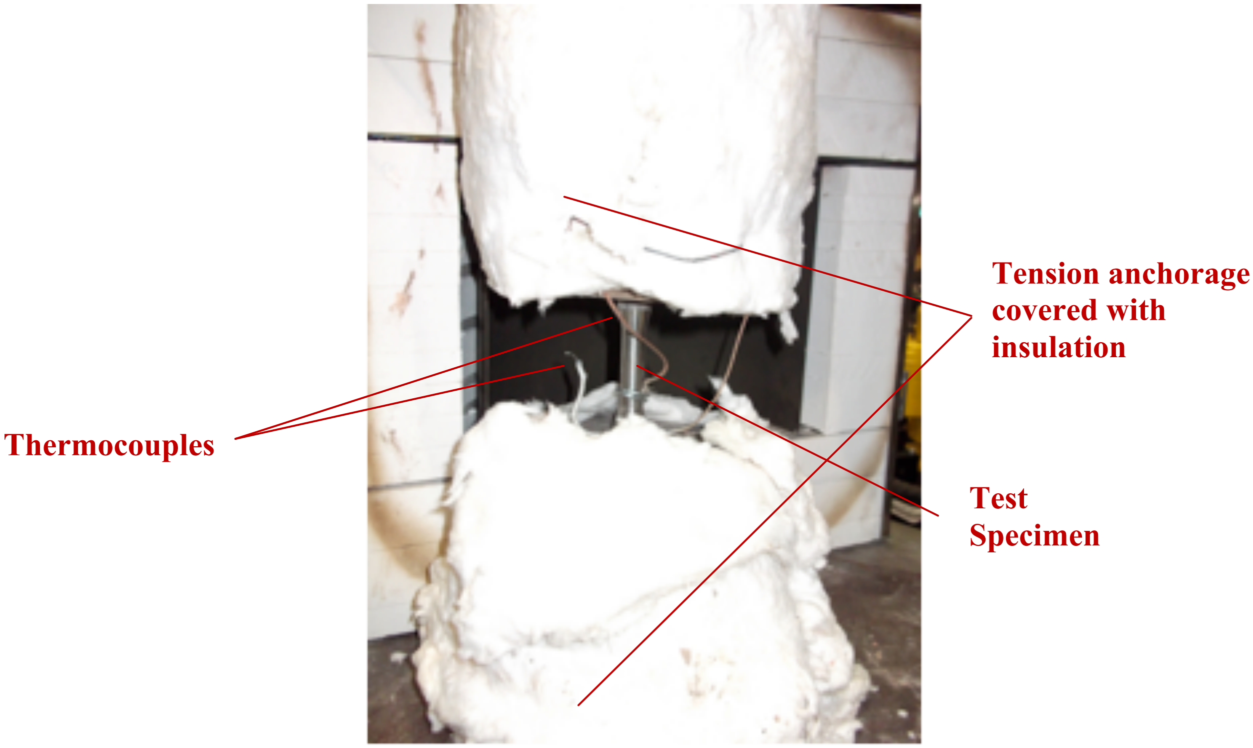



Figure 4.11 - Instrumentation on the tension test specimens

### 4.5.3. Measured data

The measured data include stress-strain points, ultimate strength, yield strength, and elastic modulus at various temperatures.

#### 4.5.3.1 Stress-strain curve

Load and deflection recorded during the tension tests was utilized to generate stress-strain curves at various temperatures.

Figure 4.12 shows the high temperature stress-strain points for A325 bolts. At each temperature the deflection was measured at least 15-16 points, with specific attention given to critical regions such as yield, rupture etc. Room temperature test data indicates clearly the linear-elastic behavior of bolts followed by nonlinear behavior. Once the ultimate stress is reached bolt was observed to undergo plastic deformation indicating unloading phase and failed at a failure strain of 5%. For 250°C temperature stress-strain curve follows similar trend as that of room temperature. However due to nature of high strength steel, the ultimate strength is slightly higher than that of room temperature and steel is comparatively more ductile than room temperature.

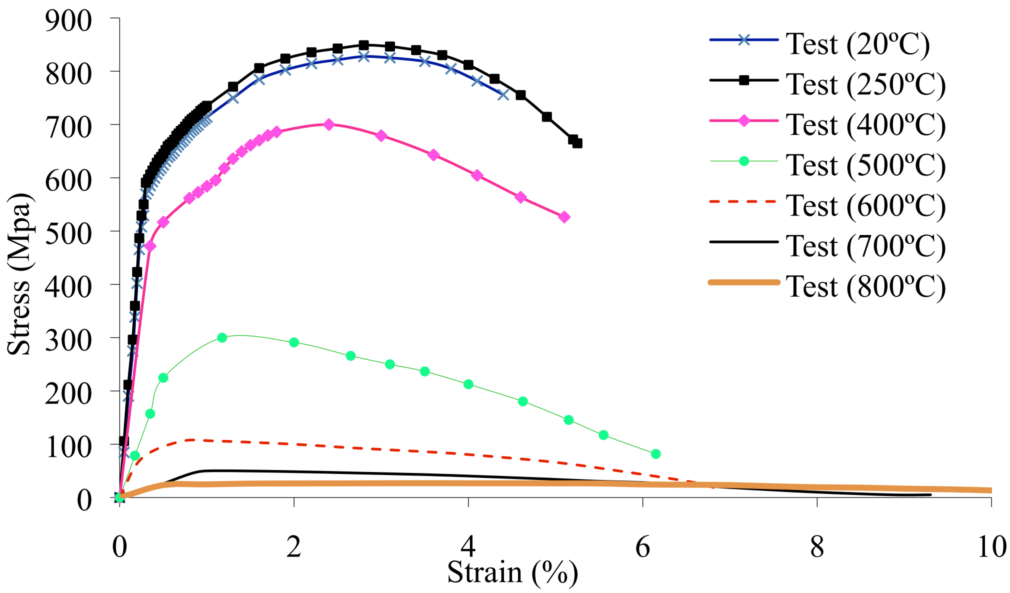



Figure 4.12 – High temperature stress-strain curve for A325 bolts

At 400°C, there is considerable loss in strength in A325 bolts, but the ductility of bolts gets slightly enhanced. This can be attributed to the softening of steel at elevated temperatures. With increasing temperature at 700 and 800°C, steel softens more and undergoes relatively higher deformation for smaller load increments. Hence linear elastic cutoff point at these temperatures cannot be clearly identified in the stress-strain curve. Failure strain increases with increase in temperature as the ductility of steel is considerably more at high temperature and reaches maximum value at 800°C. Thus at 800°C, failure strain is recorded as 10%, which is twice as that of room temperature.

Figure 4.13 shows the stress-strain curves obtained for the A490 bolts. Room temperature test data indicates clearly the linear-elastic and non-linear behavior followed by unloading phase due to necking of test specimen. Failure strain recorded was comparatively low at 4.75%, which indicates moderate ductile failure of A490 bolts. Stress-strain curve at 250 and 400°C follow the same trend as room temperature test result without considerable loss in strength and slight increase in ductility. At 500°C, the loss in strength is observed however elastic linear behavior is clearly visible and the ductility remains unchanged as indicated by the failure strain and the slope of unloading phase. When heated beyond 600°C, the elastic-linear relation of stress-strain appears to vanish as bolt undergoes considerably large deformation under small loads and considerable increase in failure strain is observed. At 700 and 800°C, the failure strain is recorded as9 and 9.75% as a result of increased ductility of steel.

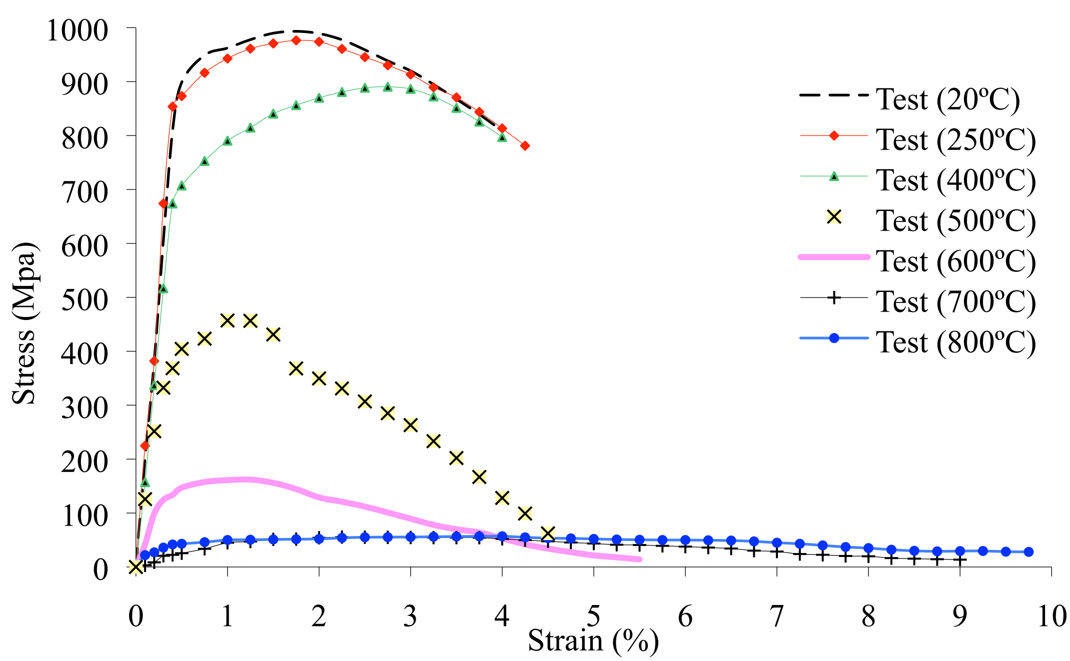



Figure 4.13 – High temperature stress-strain curves for A490 bolts

#### 4.5.3.2 Yield and ultimate strength

Maximum load applied on the test specimen during test at each test temperature was utilized to generate ultimate strength data. As stress-strain curve at high temperature does not show a sharp yield point, yield strength of bolts was evaluated as stress determined by drawing a line parallel to the linear portion of stress-strain curve at 0.2% strain as at elevated temperature.

Table 4.2 illustrates yield and ultimate strength of A325 bolts at elevated temperature. Ultimate tensile strength and yield strength of A325 bolt at room temperature was found to be 827MPa and 640Mpa respectively, which indicates that the mechanical properties of test specimen are in close agreement with ASTM A325-04b standard requirements. A slight increase of strength of specimen at 250°C can be attributed to the blue-brittle behavior of steel. At this temperature, steel becomes stronger as a result of oxidization. When heated beyond 400°C steel starts to lose strength. However the loss in strength was observed to be only 15% at 400°C. As per ASTM standard A325-04b, A325 bolts were produced by quenching followed by tempering at 427°C to achieve required strength level. Hence the influence of temperature over mechanical properties is negligible and not significant till 400°C. When heated beyond 400°C, the strength of test specimen decreases drastically.

Strength at elevated temperature normalized to the room temperature strength gives a better indicator of degradation of strength with temperature. Hence strength reduction factors are determined as illustrated in Table 4.2.

Table 4.2 - Summary of tension test results on A325 bolts

| No | Test specimen | Ultimate stress (MPa) | 0.2% Yield stress (MPa) | Reduction factor | |
|---|---|---|---|---|---|
| | | | | Ultimate stress | 0.2%Yield stress |
| 1 | 325T25 | 827 | 640 | 1.00 | 1.00 |
| 2 | 325T250 | 848 | 630 | 1.03 | 0.98 |
| 3 | 325T400 | 700 | 530 | 0.85 | 0.83 |
| 4 | 325T500 | 310 | 260 | 0.37 | 0.41 |
| 5 | 325T600 | 107 | 90 | 0.13 | 0.14 |
| 6 | 325T700 | 50 | 50 | 0.06 | 0.08 |
| 7 | 325T800 | 30 | 30 | 0.04 | 0.05 |

A325 bolts were medium carbon high strength steel bolts. High strength properties were achieved either by adding higher amount of carbon to steel or by heat treatment. During heat treatment, steel was heated to approximately 800°C, steel forms austenite crystalline structure. Sudden cooling of this austenite results into a fine grain, body centered tretragonal crystalline structure with considerable amount for carbon atoms trapped inside called martensite, which imparts high strength to steel. However the presence of carbon results in decrease in its ductility. Hence these bolts were tempered by reheating at or above 427°C to form tempered martensite and slowly cooled down to room temperature. Thus the ductility of steel increases. Hence it can be inferred that the heating of bolts above the tempering temperature changes its microstructure resulting in loss of strength. Also at high temperatures, steel molecules starts to move apart, decreasing the intermolecular bond. This ultimately results in reduction in strength of steel at 500, 600, 700 and 800°C temperature.

Strength of test specimen reduces to 37%, 13% and 6% at 500, 600 and 700°C respectively. However strength at 800°C was observed to be same as that of 700°C without any further loss as shown in Figure 4.14. When steel was heated above 700°C, microstructure of steel shows appearance of γ austenite, a high-density crystalline structure that results in decrease in steel volume. Due to this decrease in volume, the inter-atomic distance decreases and the chemical bond increases hence increasing the strength making it comparable to the strength at 700°C.

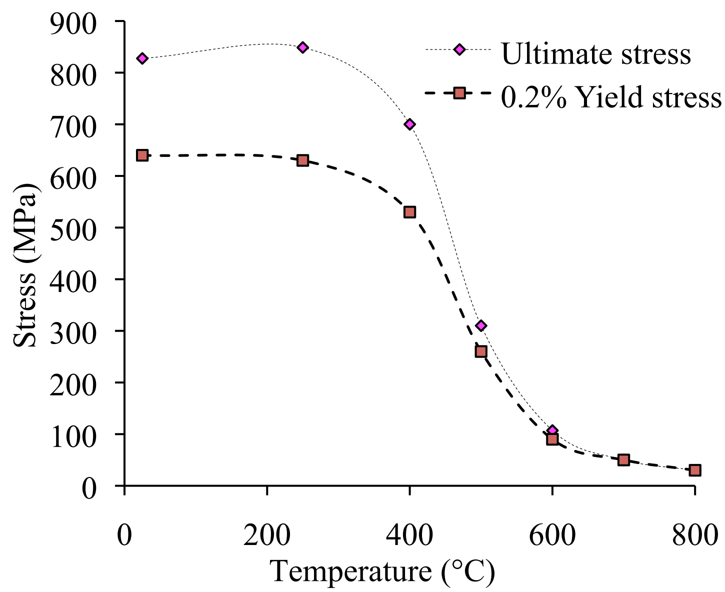



Figure 4.14 - Ultimate stress and 0.2% yield stress of A325 bolts at elevated temperature

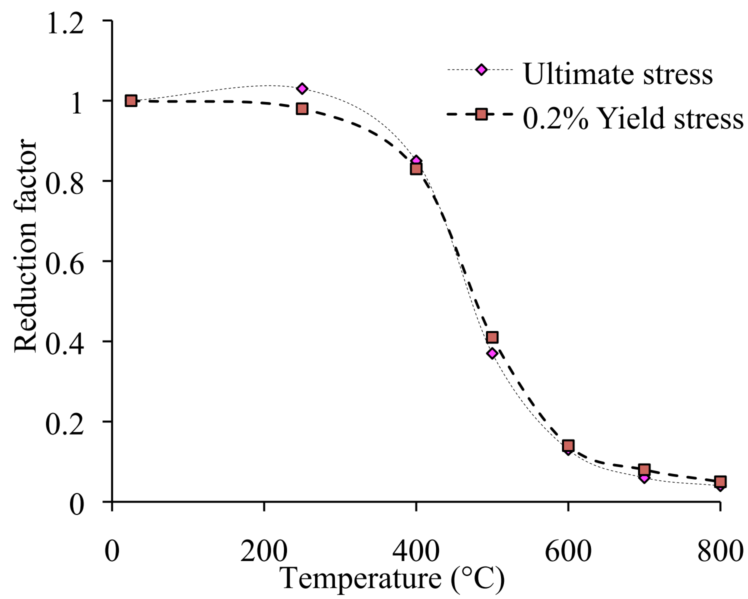



Figure 4.15 - Reduction factor for ultimate stress and 0.2% yield stress of A325 bolts at elevated temperature

Table 4.3 illustrates yield and ultimate strength of A490 bolts at elevated temperature. Ultimate strength of 993Mpa and 0.2% yield strength of 920MPa of A490 bolt observed at room temperature were in close agreement with the type I bolts requirement as per ASTM standard A490-04a. Fracture surface (Figure 4.23) shows fibrous pullout indicating plastic deformation for a small necking region followed by sudden and considerably brittle failure. At 250 and 400°C, the cup-cone fracture surface observed, indicates the ductile failure at this temperature. However no significant loss in strength was observed as a result of strength advantages imparted by tempering the A490 bolts at 450°C. At 500°C, the ultimate strength was only 50% of room temperature capacity and decreases further with increase in temperature up to 700°C (Figure 4.27). At 800°C, due to formation of the γ austenite, a slight increase in strength as compared to that at 700°C is observed. Summary of ultimate tensile strength, 0.2% yield strength, and strength reduction factors are given in Table 4.3.

Table 4.3- Summary of tension test results on A490 bolts

| No | Test specimen | Ultimate stress (MPa) | 0.2% Yield stress (MPa) | Reduction factor | |
|---|---|---|---|---|---|
| | | | | Ultimate stress | 0.2% Yield stress |
| 1 | 490T25 | 993 | 920 | 1.00 | 1.00 |
| 2 | 490T250 | 976 | 873 | 0.98 | 1.03 |
| 3 | 490T400 | 890 | 680 | 0.90 | 0.81 |
| 4 | 490T500 | 457 | 365 | 0.46 | 0.46 |
| 5 | 490T600 | 162 | 130 | 0.16 | 0.13 |
| 6 | 490T700 | 55 | 55 | 0.055 | 0.06 |
| 7 | 490T800 | 57 | 57 | 0.058 | 0.062 |

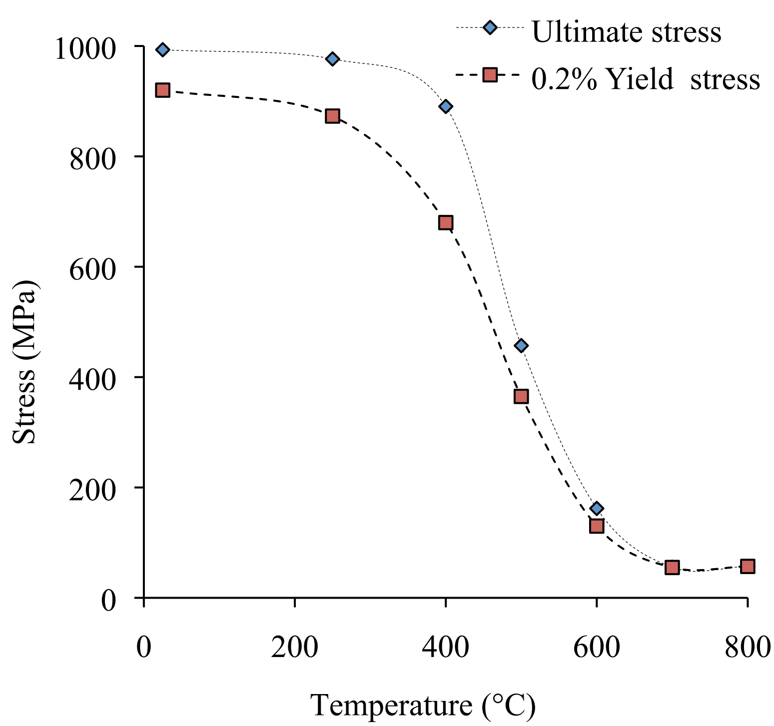



Figure 4.16 - Ultimate stress and 0.2% yield stress of A490 bolts at elevated temperature

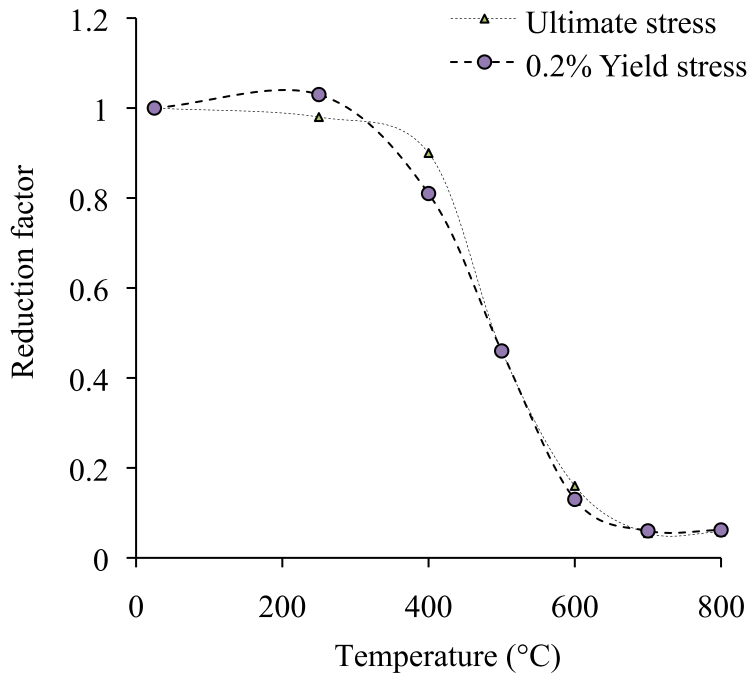



Figure 4.17 - Reduction factor for ultimate stress and 0.2% yield stress of A490 bolts at elevated temperature

#### 4.5.3.3 Elastic modulus

Slope of the linear portion of stress-strain curve defined, as elastic modulus is a measure of elastic behavior of steel subjected to tension or compression loads.

Stress-strain curves for A325 bolts as shown in Figure 4.12, indicate no reduction in strength and stiffness of bolt till 250°C. As temperature increases, steel softens and undergoes considerably large deformation under application of small loads. Thus the slope of elastic-linear portion of strain-strain curve reduces with increase in temperature. Like ultimate strength, modulus of elasticity of steel was not affected by temperature when exposed to temperatures below 300°C. However at temperatures beyond 400°C, drastic decrease in modulus of elasticity is observed. At temperature 700 and 800°C, bolts undergo considerably large deformation, which mimics the flow of viscous liquid under application of load. Hence the elastic loading zone at these temperatures is not clearly defined and this temperature range is called as 'flowaway temperature'.

Table 4.4 - Elastic modulus of A325 and A490 bolts at elevated temperature

| No | Test temperature | A325 | | A490 | |
|---|---|---|---|---|---|
| | | Elastic modulus (GPa) | Elastic modulus reduction factor | Elastic modulus (GPa) | Elastic modulus reduction factor |
| 1 | 325T25 | 217 | 1.00 | 202 | 1.00 |
| 2 | 325T250 | 217 | 1.00 | 224 | 1.00 |
| 3 | 325T400 | 140 | 0.64 | 180 | 0.89 |
| 4 | 325T500 | 45 | 0.21 | 120 | 0.59 |
| 5 | 325T600 | 29 | 0.13 | 42 | 0.21 |
| 6 | 325T700 | NA | NA | NA | NA |
| 7 | 325T800 | NA | NA | NA | NA |

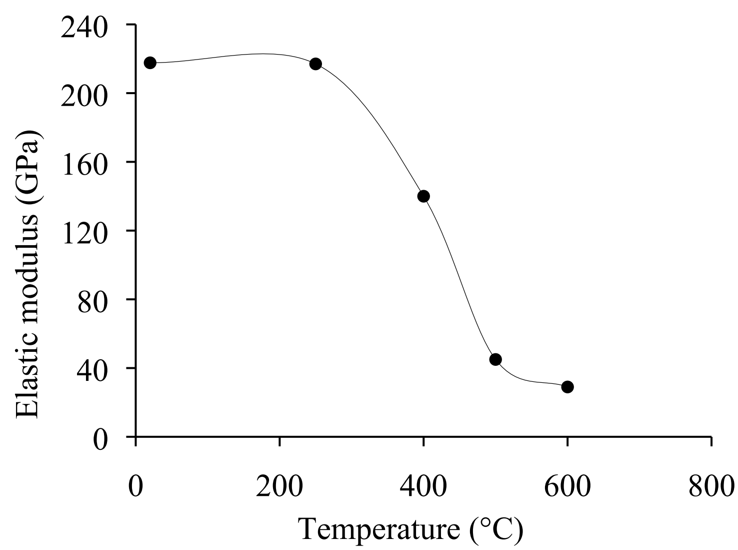



Figure 4.18 - Modulus of elasticity of A325 bolts at elevated temperature

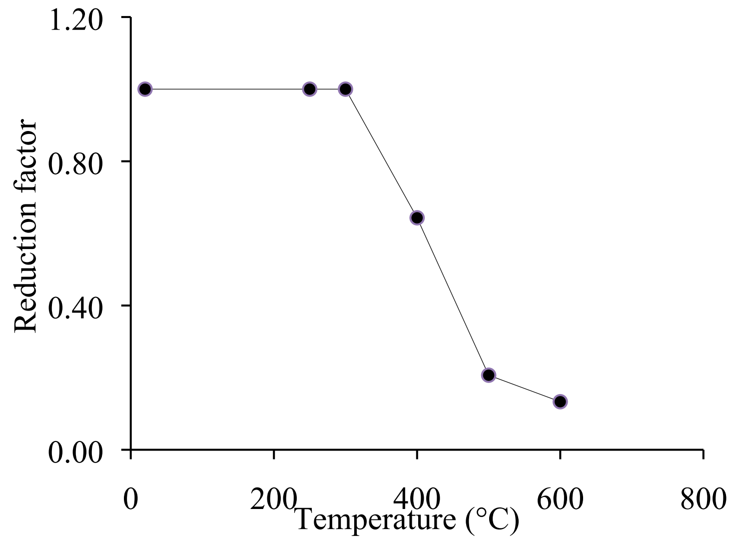


Figure 4.19 - Elastic modulus reduction factor for A325 bolts at elevated temperature

Table 4.4 illustrates the elastic modulus of A490 bolts at elevated temperatures. At room temperature the elastic modulus of A490 bolts was evaluated to be 202GPa. With increase in temperature elastic modulus of A490 bolts appears to decrease. This reduction in elastic modulus was drastic at and after 500°C. At 700 and 800 °C, stress-strain curve does not show clear elastic behavior under tensile load and hence elastic modulus of A490 bolts could not be evaluated. However it can be seen that A490 bolts perform better in keeping its elastic modulus at elevated temperatures than A325 bolts.

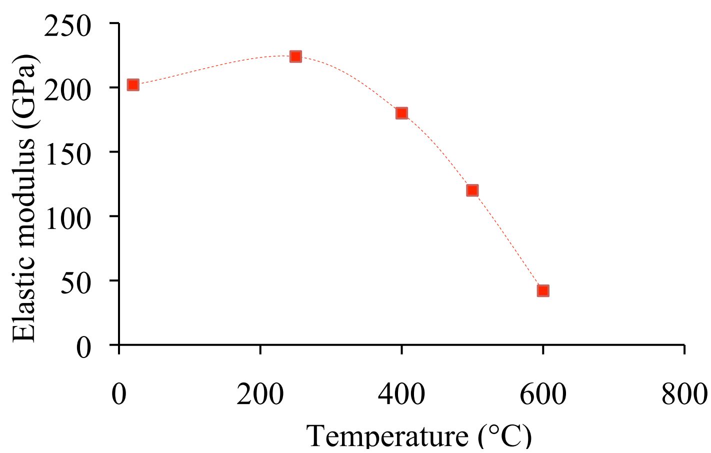


Figure 4.20 - Elastic modulus of A490 bolts at elevated temperature

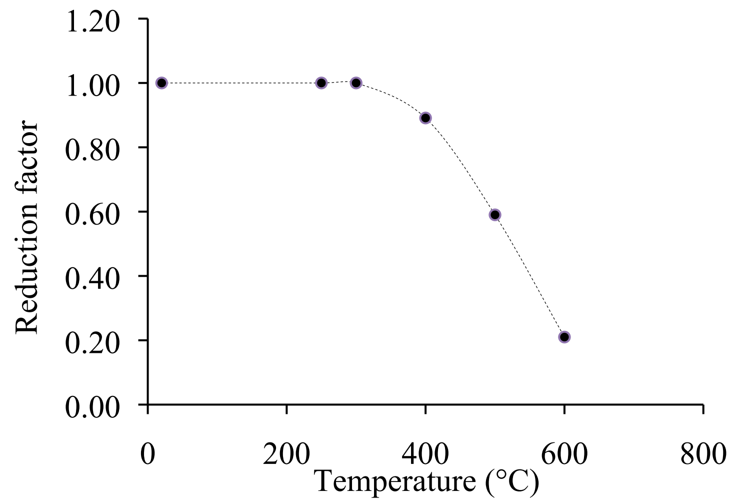



Figure 4.21 - Elastic modulus reduction factor for A490 bolts at elevated temperature

#### 4.5.3.4 Ductility

Ductility represents extent to which steel can undergo plastic deformation before failure. Failure strain or reduction in area of test specimen at fracture is used as indicators of ductility of steel. Reduction in area measured at the location of minimum size of the specimen neck is used as an indicator of ductility, which can be evaluated using;

$$\text{Percent reduction of area} = \frac{\text{Area of original cross section} - \text{Minimum cross section of fracture surface}}{\text{Area of original cross section}} \times 100$$

Minimum diameter of fracture surface and failure strain of test specimen for each test was recorded to gauge on the influence of temperature on the ductility of A325 and A490 bolts.

Figure 4.22 illustrates the fracture of test specimen during each test at a specified test temperature. Table 4.5 gives the percent reduction in cross section and failure strain of A325 bolts. As the tension test was performed by applying deflection-controlled load with the help of two hydraulic jacks, controlling the operation of both the hydraulic jacks was a challenge. Due to small variation in the load applied by both the jacks the fracture of test specimen was observed to be at different locations. However, all of the fracture surfaces lie within the gauge length of the round test specimen. Hence the test result was considered to be unaffected by the difference in motion of the two hydraulic jacks.

The cup-cone fracture surface of test specimen during room temperature test indicates the ductile behavior of A325 steel. From the Figure 4.22, the increase in length of necked region of test specimen with increase in temperature was clearly visible. As the temperature in steel increases the fracture surface appears to grow smaller. The failure strain of 5.75% at room temperature increases to 12.5% at 800°C. Whereas, the percent reduction of area increases from 75% to as much as 98% for temperature range of 25 to 800°C. This can be attributed to the increased ductility of steel at elevated temperature. For temperatures up to 500°C, ductility remains almost unchanged. With further increase in temperature, the ductility of steel reaches its maximum value at 800°C. At temperature range of 700 to 800°C, intergranular coagulation occurs during deformation for the applied load. During this coagulation, weak intergranular bond at these temperatures results in redistribution of deformation from boundaries into bulk of grains. As the intergranular boundaries vanish at these temperatures, no intergranular slip occurs at deformation hence the rate of cold-working reduces and ductility of steel increases. However this effect is visible only for the temperature range of 700 to ~850°C, when steel reaches homogenous crystalline microstructure after phase transformation.

Figure 4.23 and 4.24 illustrates fracture surface of A490 bolts at elevated temperatures. Fracture surface observed during room temperature test of A490 bolt clearly shows fibrous pullout indicating plastic deformation for a small necking region followed by sudden and considerably brittle failure as shown in Figure 4.23 and Figure 4.24. At 400°C, the cup-cone fractured surface observed during test indicates the increase in ductility. However, as indicated in Table 4.5, the percent reduction in fracture surface and failure strain increases slightly till 600°C as a result of only slight increase in ductility. At 700 and 800°C, like A325 bolts, A490 bolts also show considerable increase in ductility due to decrease in intergranular slip, avoiding cold work hardening and sudden brittle failure.

Table 4.5 - Percent reduction of area and failure strain of A325 and A490 bolts

| Temperature (°C) | A325 | | A490 | |
|---|---|---|---|---|
| | Percent reduction of area (%) | Failure strain (%) | Percent reduction of area (%) | Fracture strain (%) |
| 20 | 75.29 | 5.25 | 53.54 | 4 |
| 250 | 85.00 | 5.25 | 82.09 | 4.25 |
| 400 | 87.60 | 5.25 | 80.00 | 4 |
| 500 | 89.24 | 6.15 | 84.25 | 5.75 |
| 600 | 92.73 | 6.9 | 85.80 | 5.75 |
| 700 | 94.69 | 9.3 | 91.38 | 9 |
| 800 | 98.83 | 12.5 | 92.29 | 9.75 |


Figure 4.22 – Illustration of fracture in A325 bolts at various test temperature fractured test specimen after test

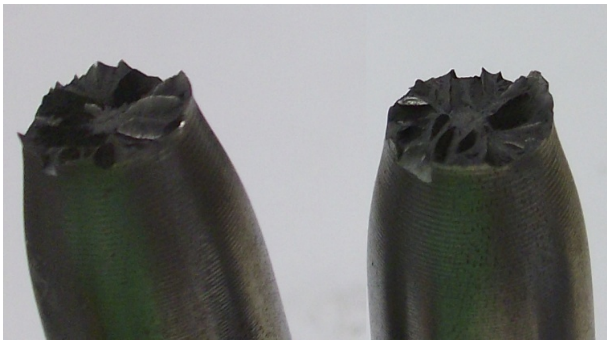


Figure 4.23 - Fractured surface with fibrous pullout of A490 bolts due to tensile loading at 20˚C


Figure 4.24 – Illustration of fracture surfaces in A490 bolts at various temperatures

## 4.6 Single shear tests

The load transfer through connections occurs through the shear mechanism. The failure in a bolted connection is mostly shear fracture and without prior warning. This can be fatal. Hence steel bolted connections have to be designed to possess adequate shear capacity at room temperature as well as at elevated temperature. A series of single shear tests was conducted to determine shear strength of A325 and A490 bolts at elevated temperatures. A description of the tests and results are presented in the following chapter.

### 4.6.1 Test specimen

A325 and A490 bolts were tested in single shear test at various temperatures. Test specimen of a diameter 22mm, 150mm long hex head bolt was supplied as per the ASTM325/325M-04b and ASTM490/490M-04a standard specification. ASTM F436 grade washer and nut was used for the test. Mechanical properties and chemical composition of steel used in bolts are given in Table 4.1.

The specimen for single shear tests was labeled by the bolt type and the test temperature. For example, A325S200 designated for A325 bolt specimen tested for single shear 200$^{\circ}$C.

### 4.6.2 Test procedure

Steady state single shear tests on A325 and A490 bolts were performed at eight temperature points - 20, 200, 300, 400, 500, 600, 700 and 800$^{\circ}$C. Each test assemble was made of two shear anchorage plates (Figure 4.6). A snug tightened test bolt runs through the center bolt hole of these anchorage plates, facilitating application of shear force on the shank of the bolt. Thickness of shear anchorage plates, which was 63mm, was designed to facilitate only shear failure in bolts. The joint assembly was connected to the loading frame and surrounded by the electric furnace. Two thermocouple wires were put under washer and bolt head separately and affixed by snug tightening the specimen bolt to read temperature of test specimen. The average reading of the two thermocouples was taken as the temperature in the specimen. A third thermocouple was inserted in the furnace next to the bolt assembly to record temperature of the air in the furnace.

A heating rate of 5°C per minute was used to heat the specimen to the required test temperature.

Once heated to test temperature, the test specimen was allowed to stabilized for 15 minutes so as to ensure uniform temperature distribution throughout the specimen. In most of the tests the fluctuation in temperature during loading was observed to be within 5 °C.

Followed by temperature stabilization, deflection controlled quasistatic load was applied on the shear anchorage plates, which was transferred to the test specimen through shear mechanism. Two load cells attached to each hydraulic jack recorded the test load. These loads were added to find total load applied on the specimen. Deflection was recorded with the help of the strain pod.

### 4.6.3Measured shear strength

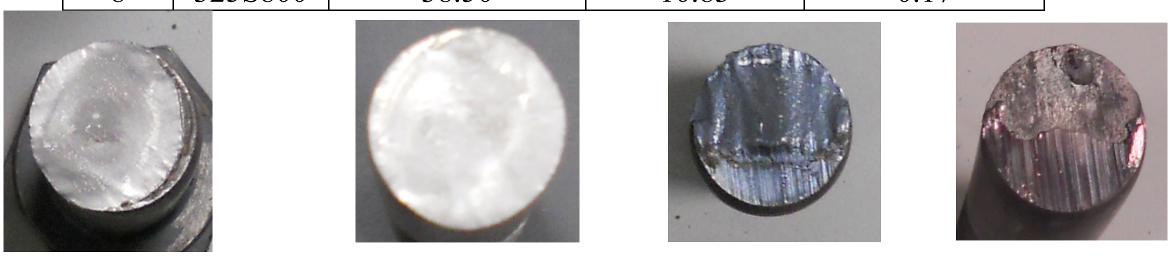
The maximum load recorded during the each shear test was considered to be the shear strength of the bolt. Shear strength of a bolt is typically considered to be 0.6 times its ultimate tensile strength. Test result shows that the shear to ultimate tensile strength ratio of A325 and A490 bolt was observed to be 0.68 and 0.62 respectively.

Table 4.6 illustrated the test results for single shear test performed on A325 bolts at various temperatures. Figure 4.27 illustrates the load-displacement curves of single shear tests on A325 bolts. Shear strength of A325 bolt at room temperature was observed to be 565MPa. The failure surface as shown in Figure 4.25 (a) is bright and shiny grey and shows no sign of ductility. From load-deflection curve indicate that the failure of test specimen was the brittle at room temperature with no unloading zone. When the bolt was heated to 200°C, loss in shear strength was observed to be about 6%. Failure surface appeared to be plane and shiny at 200°C. At 300°C, the shear strength appears to increase to its room temperature capacity. This strength in capacity can be attributed to the oxidation of steel at this temperature which makes steel stronger. This temperature is called blue brittle range of steel. As shown in Figure 4.25 (c) the fracture surface appears shiny blue due to oxidation of steel with no evidence of ductility. No loss in shear strength is observed till 300°C. At 400°C, the shear strength of the bolt drops to about 78% of its room temperature capacity. Fracture surface at this temperature appears party rough and party smooth. At this temperature, color of steel changes from metallic grey to shiny purple. However, failure was brittle. For temperature range of 25 to 400°C, flying of a part of bolt out of assembly with high velocity known as prying action, was observed due to sudden failure of bolts. Extra precaution was taken to avoid damage of furnace due to this prying action.

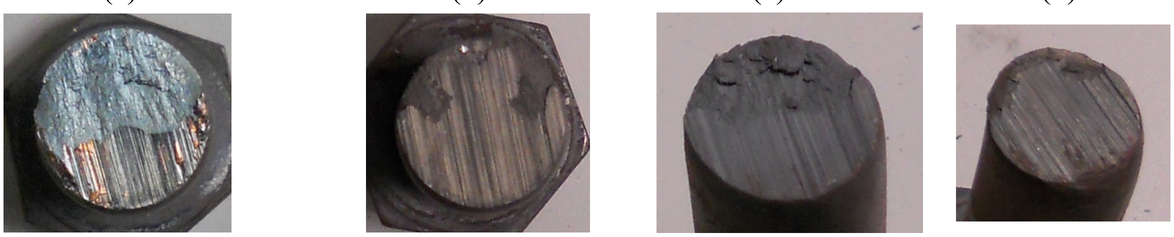
With increase in temperature, the bolts were observed to become soft, resulting in increase in ductility. At temperatures 500, 600, 700 and 800°C the failure surface observed is smooth with visible abrasion marks. This indicates tearing failure of steel at these temperatures. The color of steel changed from shiny metallic to grey. At these temperatures the shear capacity was observed to be 57%, 43%, 20% and 17% of the room temperature capacity respectively. It can be seen that the rate of strength loss reduces from 700 to 800°C. This can be attributed to phase change of steel that leads to formation of $\gamma$-austenite.

Table 4.6 - Summary of single shear test results on A325 bolts

| Sr. No | Test specimen | Maximum shear load (kN) | Shear strength (MPa) | Strength reduction factor |
|---|---|---|---|---|
| 1 | 325S25 | 218.76 | 564.93 | 1 |
| 2 | 325S200 | 206.52 | 533.32 | 0.94 |
| 3 | 325S300 | 217.10 | 560.65 | 0.99 |
| 4 | 325S400 | 212.92 | 549.85 | 0.78 |
| 5 | 325S500 | 152.71 | 394.38 | 0.57 |
| 6 | 325S600 | 95.20 | 34.61 | 0.43 |
| 7 | 325S700 | 44.95 | 14.79 | 0.20 |
| 8 | 325S800 | 38.50 | 10.83 | 0.17 |



(a) 25°C (b) 200°C (c) 300°C (d) 400°C



(e) 500°C (f) 600°C (g) 700°C (h) 800°C

Figure 4.25 - Failure sections of A325 bolts at different temperatures

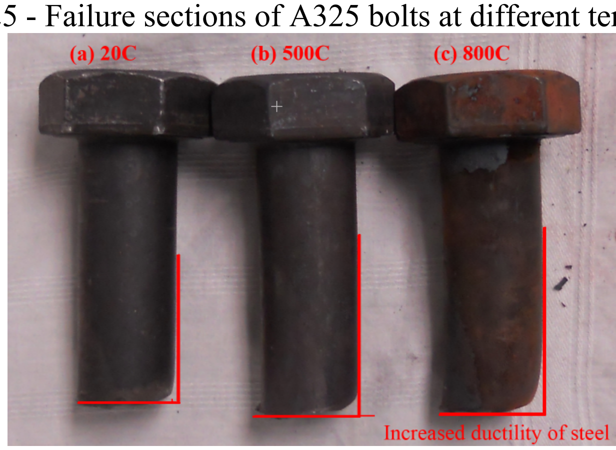



Figure 4.26 - Longitudinal fracture surface of A325 bolts indicating increased ductility with temperature

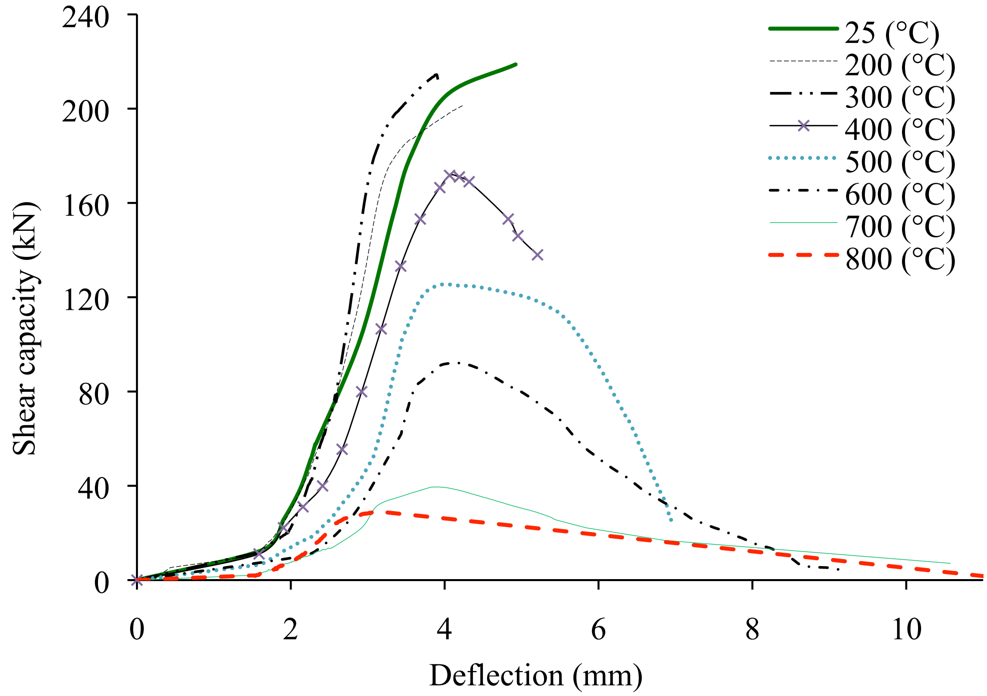



Figure 4.27 – Load-deflection curve for single shear test A325 bolts at elevated temperature

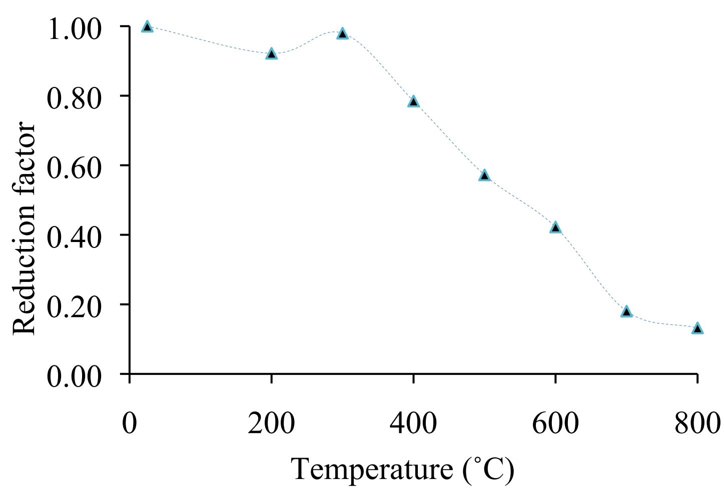



Figure 4.28 - Shear strength reduction factor for A325 bolts.

Single shear strength of A490 bolts at elevated temperature is given in Table 4.7. Similar to A325 bolts, A490 bolts also shows no significant loss in strength up to 300˚C. Strength of the bolt starts to depreciate with increase in temperature.

At room temperature the bolt shows shear capacity of 643MPa, which is 0.62 times the tensile strength of bolt. Slight reduction in strength was observed at 200°C and the fracture surface observed is shiny and smooth. However at 300°C, due to blue brittle effect the strength appears to increase and was almost same as its room temperature capacity. At this temperature due to oxidation, fracture surface appears shiny blue. Strength at 400°C is observed as 85% of room temperature capacity. Fracture surface is shiny purple in color.

As temperature increases strength reduces to 66% and 48% of original capacity at temperatures 500 and 600°C. Failure of bolt for temperature range of 25 to 500°C was observed to be brittle with considerable prying action as can be seen in Figure 4.32. The ductility appears to increase after 600°C (Figure 4.30). At 700°C, the measured strength of bolt was 14% of its room temperature capacity and it remains almost constant at 800°C. Fracture surface at 600, 700 and 800°C starts to show abrasion marks as an indication of increased ductility.

Table 4.7 - Summary of single shear test results on A490 bolts

| Sr. No. | Test | Test temperature (°C) | Maximum load (kN) | Shear strength (MPa) | Stress reduction factor |
|---|---|---|---|---|---|
| 1 | 490S25 | 25 | 249.18 | 643.51 | 1.00 |
| 2 | 490S200 | 200 | 229.26 | 592.05 | 0.92 |
| 3 | 490S300 | 300 | 240.11 | 620.08 | 0.96 |
| 4 | 490S400 | 400 | 212.92 | 549.85 | 0.85 |
| 5 | 490S500 | 500 | 165.20 | 426.62 | 0.66 |
| 6 | 490S600 | 600 | 118.36 | 305.66 | 0.48 |
| 7 | 490S700 | 700 | 51.65 | 133.38 | 0.19 |
| 8 | 490S800 | 800 | 37.42 | 96.64 | 0.14 |

(a) At 25˚C (b) At 200˚C (c) At 300˚C (d) At 400˚C

(e) At 500˚C (f) At 600˚C (g) At 700˚C (h) At 800˚C

Figure 4.29 – Fracture surface of A490 bolts due to shear at different temperature levels

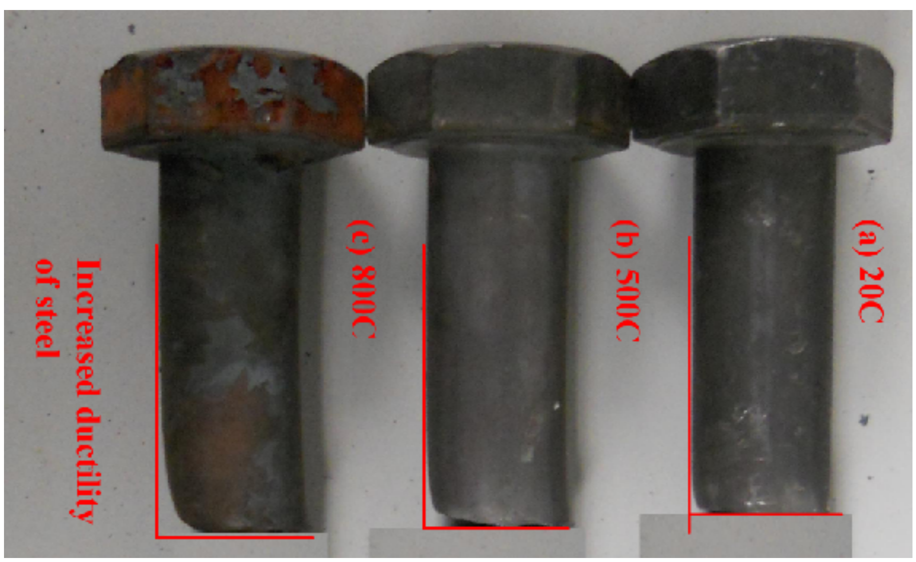



Figure 4.30 - Longitudinal fracture surface of A490 bolts indicating increased ductility with temperature

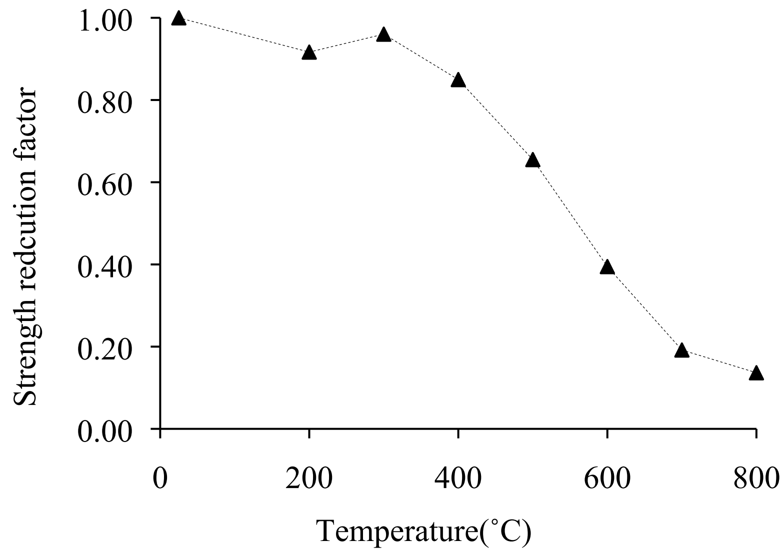


Figure 4.31 – Temperature induced shear strength reduction factor for A490 bolts.

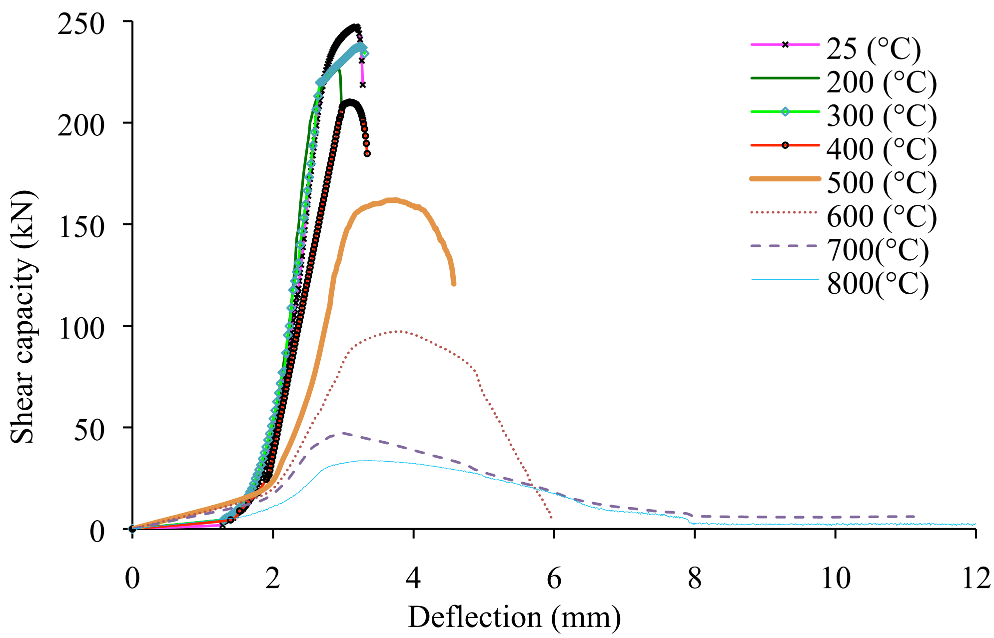


Figure 4.32 – Load-deflection curve for single shear test on A490 bolts

### 4.7 Comparative performance of strength of A325 and A490 bolts

The stress-strain curves of A325 and A490 bolts at three temperatures namely 25, 500 and 800˚C are compared in Figure 4.33 to illustrate difference in behavior. Room temperature stress-strain curve indicate relatively brittle failure of A490 bolts as compared to A325 bolts. As temperature increases considerable increase in ductility of both bolt type can be observed, however A490 bolt is still less ductile than A325 bolts. This can be attributed to higher carbon content and ambient temperature properties of A490 bolts.

Ultimate strength and shear strength reduction factors for A325 and A490 bolts are compared in Figure 4.34 and 4.35. The comparison indicates that reduction in both the ultimate and shear strength for the both bolts types is negligible at temperatures up to 400°C. Thus is can be concluded that the strength of bolts is not affected by the temperature up to the tempering temperature. At 500°C and 600°C the A490 bolts show considerably higher strength than A325 bolts. This can be attributed to the higher carbon, chromium and molybdenum content of A490 bolts. As carbon imparts strength to steel, higher carbon content results in higher strength and reduced ductility. Molybdenum and chromium increases high temperature strength of steel. Hence it can be seen that not only strength but also elastic modulus of A490 bolts at elevated temperature is superior to A325 bolts till 700°C. For the same reason, during single shear tests, failure of A325 bolts was observed to be sudden with considerable prying action for temperatures up to 400°C, whereas A490 bolts showed sudden failure till 500°C. The strength advantage of A490 bolts over A325 bolts gained due to heat treatment and chemical composition appears to depreciate above 700°C, where both bolts show almost similar strength. At temperatures above 700°C, all types of steel form the same γ-austenite crystalline structure, which governs the strength of steel at this temperature.

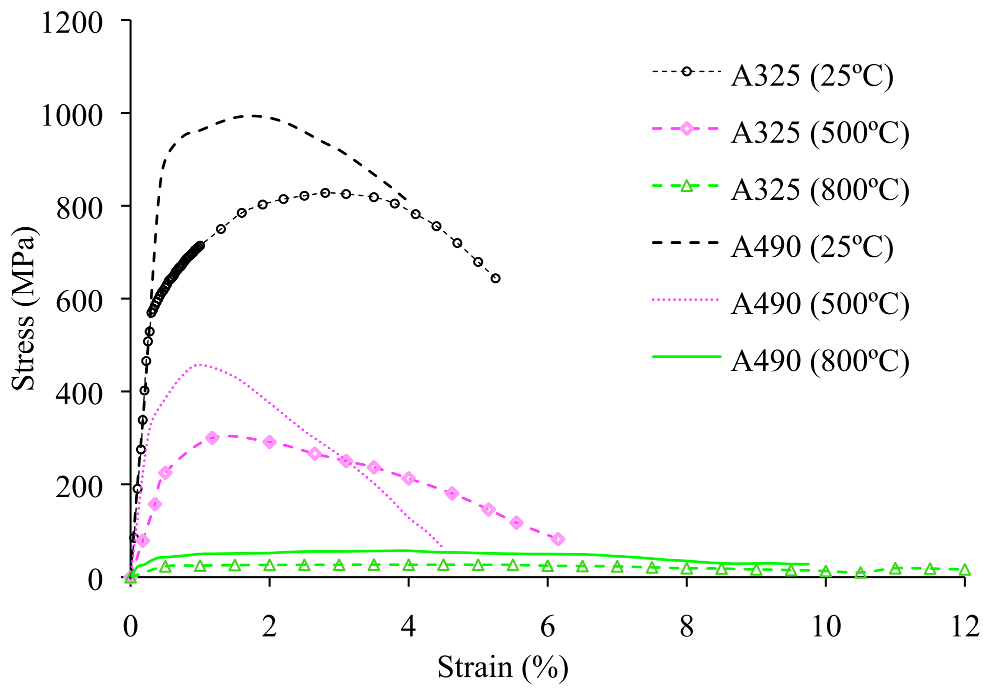



Figure 4.33-Comparison of stress-strain relation of A325 and A490 bolts at elevated temperature

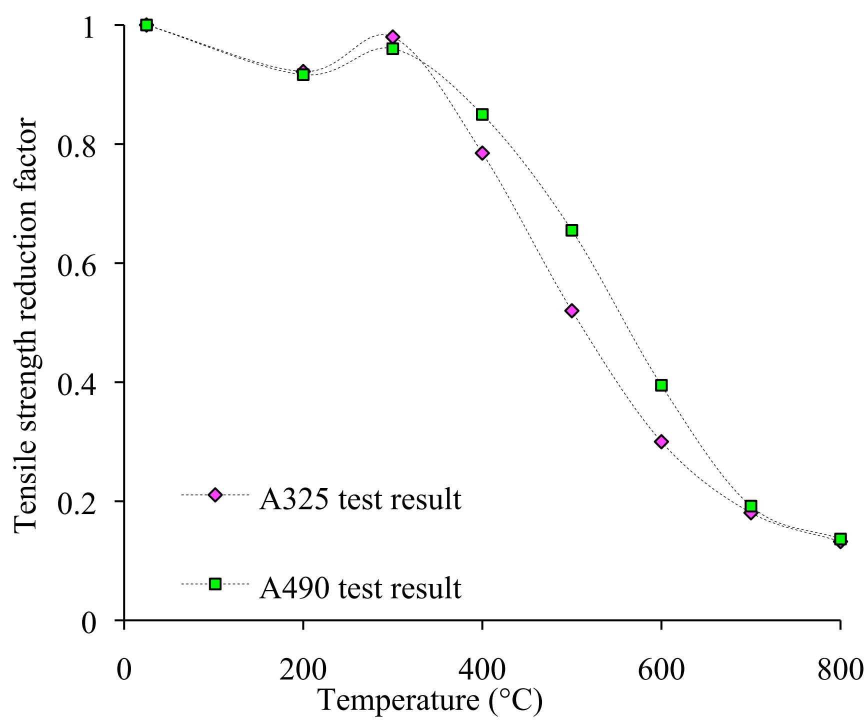



Figure 4.34 - Comparison of high temperature tensile strength capacity of A325 and A490 bolts

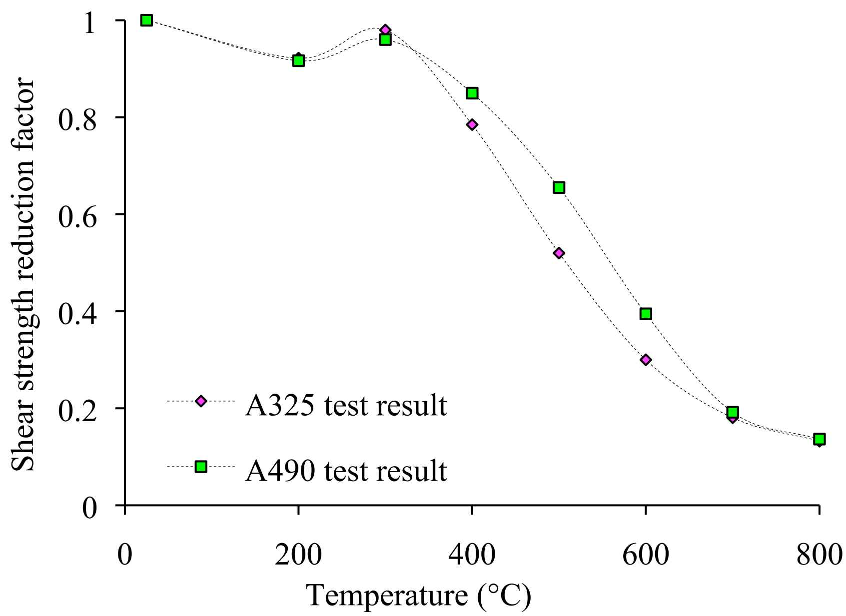



Figure 4.35 - Comparison of high temperature shear strength capacity of A325 and A490 bolts

### 4.8 Comparison of test data with published test results

Kirby (1995), Yu (2006) and Gonzalez (2009) performed high temperature shear and tension tests on A325 and A490 bolts. Following section compares the present test data with these published test models as well as with the Eurocode EC3, and British code BS5950 design guidelines.

#### 4.8.1 Comparison of ultimate tensile strength of A325 and A490 bolts

Kirby (1995) conducted tension tests on M20 grade 8.8 bolts at elevated temperature. Two different types of bolts one produced by hot forging (set A) and the other produced by cold forging (set C) were tested in the temperature range of 25 to 800°C at an interval of 100°C. Chemical composition of these bolts is given in Table 4.8. Gonzalez conducted steady state test and transient test on A490 bolts at elevated temperature for the range of 25 to 800°C.

Table 4.8 - Chemical composition of grade 8.8, A325 and A490 bolts tested by Kirby (1995) and Yu (2006)

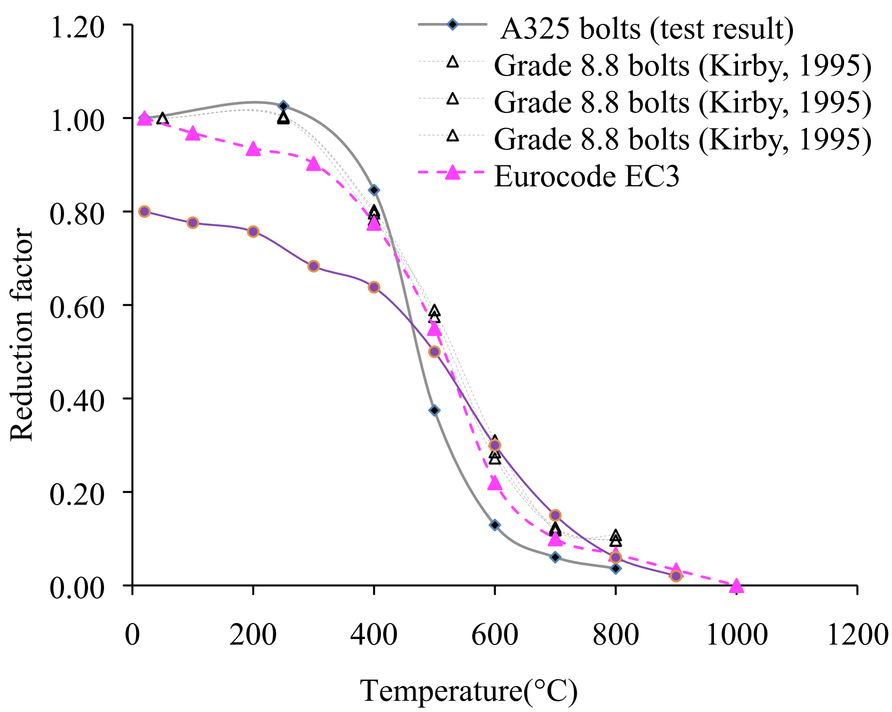
| Test series | Bolt type | Chemical composition | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | C | Si | Mn | P | S | Cr | Mo | Ni | B | Cu | N |
| Kirby (1995) | Gr. 8.8 Set A | 0.19 | 0.21 | 1.16 | 0.020 | 0.017 | 0.19 | 0.027 | 0.14 | 0.0051 | 0.22 | 0.0080 |
| | Gr. 8.8 Set C | 0.41 | 0.16 | 1.61 | 0.021 | 0.038 | 0.13 | 0.130 | 0.12 | <0.0005 | 0.23 | 0.013 |
| Yu (2006) | A325 | 0.29 | 0.27 | 0.76 | 0.006 | 0.010 | 0.05 | 0.010 | 0.06 | 0.0009 | 0.12 | 0.026 |
| | A490 | 0.36 | 0.24 | 0.76 | 0.015 | 0.009 | 1.13 | 0.180 | 0.04 | <0.0005 | 0.03 | 0.023 |

Strength reduction factors obtained from the tests performed on A325 and A490 bolts at elevated temperature are compared with the published data by Kirby (1995), Gonzalez (2009), and constitutive relations provided as per Eurocode and British code as shown in Figure 4.36 (a) and (b) respectively.

Constitutive relation provided by Eurocode EC3 and British code BS 5950, is not for any specific bolt type and is considered applicable for all bolts irrespective of its grade, chemical composition or manufacturing process. Comparing the test data of A325 and A490 bolts with these guidelines indicate that Eurocode is conservative for temperature upto 450$^{\circ}$C. However bolt strength predicted by Eurocode for temperature above 450$^{\circ}$C is considerably higher than the bolt capacity. Similar can be said for the British code provision. As these strength reduction factors provided by BS5950 are derived as 80% of strength of structural steel, it does represent realistic behavior of bolts at elevated temperature.

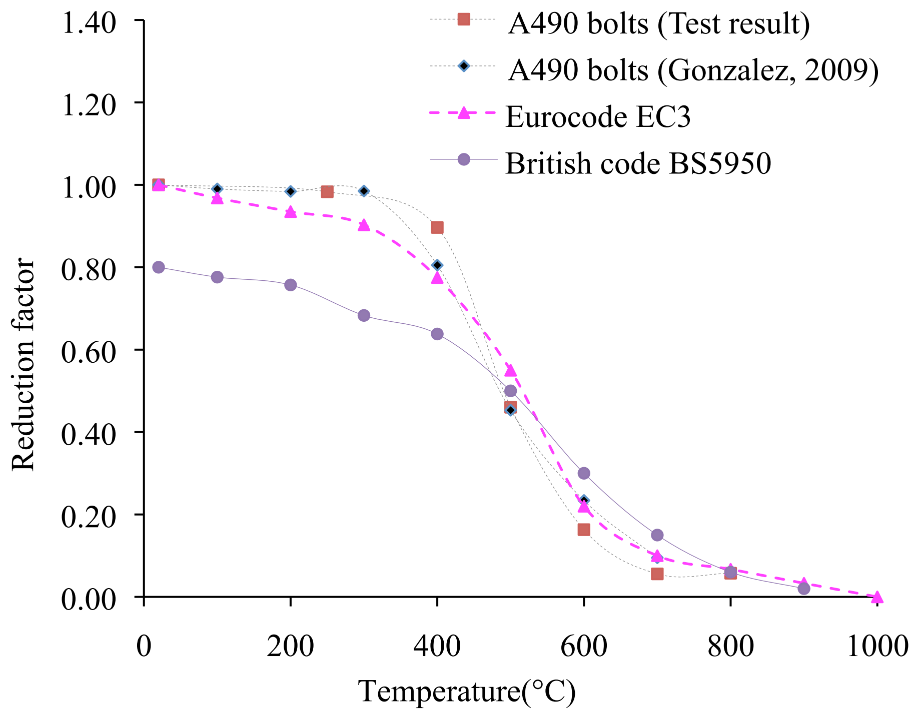
Comparison of A325 bolt test results with published data of Grade 8.8 bolt properties by Kirby (1995) indicate the Grade 8.8 bolts comparatively higher strength at temperatures more than 500˚C. This higher strength can be attributed to variation in chromium content of Grade 8.8 and A325 bolts. Chromium improves high temperature strength properties and also hardenability of steel. Presence of chromium also improves response of the steel to the heat treatment. Grade 8.8 bolts tested by Kirby(1995) has higher chromium content, hence it shows better strength capacity than A325 bolts at higher temperature.

Test results of A490 bolts when compared to published data by Gonzalez (2009) indicate slight variation in the strength of bolts with increase in temperature. As the tension test procedure, heating rate and strain rate has significant effect on the test result; this variation can be attributed to the slight differences in test procedure.




(a) Ultimate strength reduction factors of A325 bolts

Figure 4.36 - Comparison of ultimate strength reduction factors in bolts as predicted by different test programs and constitutive relations in EC3 and BS5950




(b) Ultimate strength reduction factors of A490 bolts

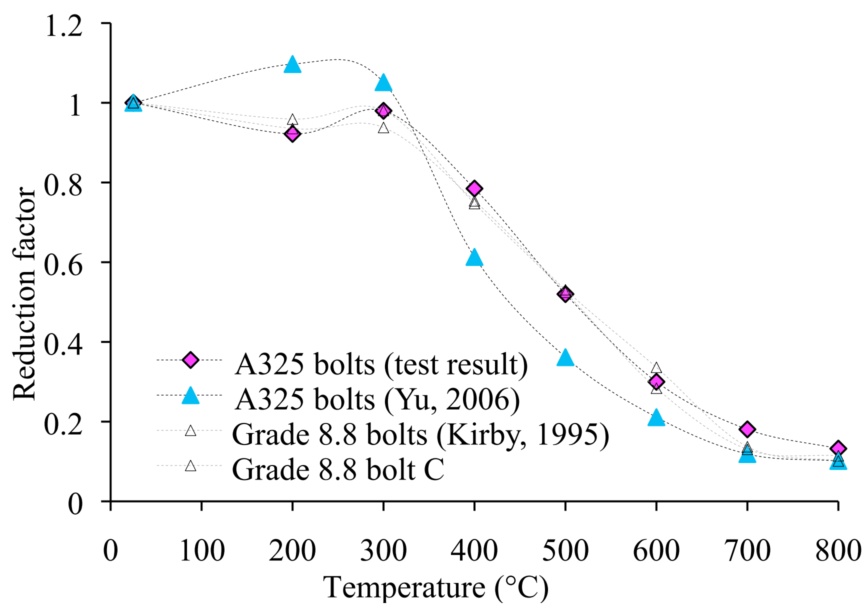
Figure 4.36(cont'd) - Comparison of ultimate strength reduction factors in bolts as predicted by different test programs and constitutive relations in EC3 and BS5950

**4.8.2 Comparison of shear strength of A325 and A490 bolts**

During a test program, Kirby (1995) conducted double shear tests on M20 Grade 8.8 bolts at elevated temperature. Yu (2006) conducted double shear test on A325 and A490 grade bolts. Chemical composition of these bolts is given in the Table 4.4.

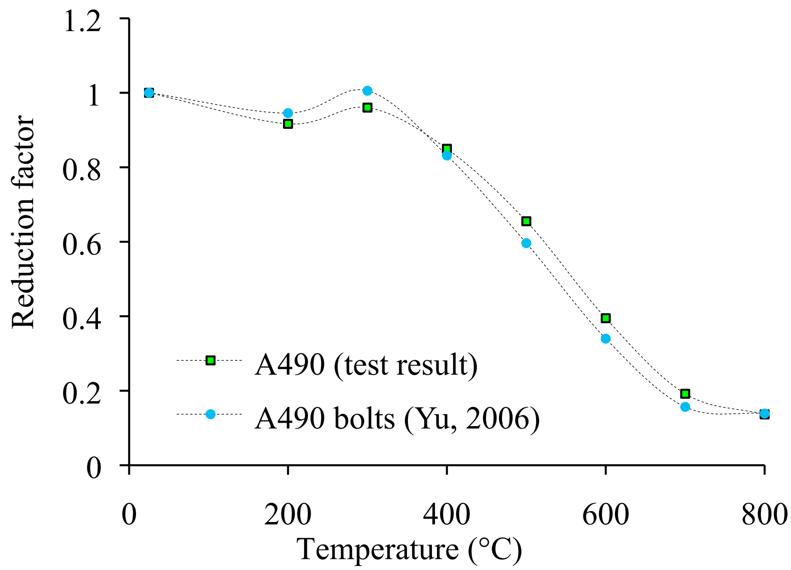
Shear strength reduction factors from different test models are compared as shown in Figure 4.37 (a) and (b). From comparison it can be seen that the data from the current tests on A325 bolt follows the trend shown by Grade 8.8 bolts set A and C tested by Kirby (1995). However, depreciation in strength of A325 bolts tested by Yu (2006) is comparatively more.

The difference in strength capacity of three different test programs can be attributed to the chemical composition of the bolts. From Table 4.1 and 4.9 it can be seen that the percent molybdenum content in A325 bolts tested by Yu (2006) is less as compared to other two tested bolts. Molybdenum is an important alloy that improves strength, creep resistance and hardness of steel at elevated temperature by introducing carbide. Strength of bolts (Figure 4.20(a)) from high to low in the temperature range of 300°C to 800°C is linearly dependent on the molybdenum content. Percent molybdenum for steel can vary up to 3%. However A325 and A490 bolts generally contain very limited molybdenum, less than 0.2%, which can be the reason for drastic strength loss at elevated temperature (Yu, 2006). The tested A490 bolts follow the trend of A490 bolts tested by Yu (2006). The little variation observed can be attributed to the variation in test procedure.




(a) Shear strength reduction factors for A325 bolts

Figure 4.37 -Comparison shear strength of bolts as predicted by different test programs and constitutive relations in EC3 and BS5950




(b) Shear strength reduction factors for A490 bolts
Figure 4.37 (Cont'd) -Comparison shear strength of bolts as predicted by different test programs and constitutive relations in EC3 and BS5950

**4.9 Comparison of high temperature properties of A325 and A490 bolts with conventional steel**

Following section compares high temperature strength properties of A325 and A490 bolts to that of conventional steel. Review of literature indicates that considerable research has been done for past few decades to study behavior of conventional structural steel. Based on this research, ASCE (1992) and Eurocode EC3: Part 1 and 2, provides constitutive relation to determine high temperature mechanical properties of conventional steel at elevated temperature. Yield strength and elastic modulus reduction factors of conventional steel as function of temperature obtained from Eurocode, ASCE code and different test models are compared with the yield strength of A325 and A490 bolts.

Figure 4.38 shows comparison of yield strength reduction factor of conventional steel and high strength bolt A325 and A490 steel. This comparison clearly shows that strength of high strength steel bolts is more sensitive to temperature as compared to that of conventional steel. Also, comparison of different models for high temperature properties of conventional steel indicates large variation in the Eurocode and ASCE guideline and different test models. This variation can be attributed to the influence of test procedure, heating rate, and strain rate on resulting tensile stresses. For example lower strain rate, results in longer duration of loading of steel member subjected to fire exposure, which develops temperature creep into steel reducing its load carrying capacity. At elevated temperature, microstructural changes in steel govern the mechanical and physical properties of steel. Higher heating rate of steel resulting in faster physiochemical changes may overlook important phase changes that govern the strength properties of steel. Hence the variation in the different test program results can be attributed to many factors, primarily variable heating and strain/load rates during the test. Further this variation in test data resulted in different test measurements, thereby leading to different in constitutive relations provided in ASCE and Eurocode provisions.

A comparison of yield strength of conventional steel as predicted by ASCE model predicts that strength of conventional steel starts to decrease immediately at 100°C. Whereas high strength steel bolts retain its strength up to 400°C. On the other hand Eurocode model shows great agreement with the bolt test result till 300°C, as it assumes no strength reduction in conventional steel up to 400°C. As temperature increases beyond 400°C, rate of strength reduction of A325 and A490 bolts increases drastically as compared to conventional steel. At temperatures beyond

800°C, high strength steel bolts loses 95% of its strength, however this reduction in conventional steel is observed at 1000°C.

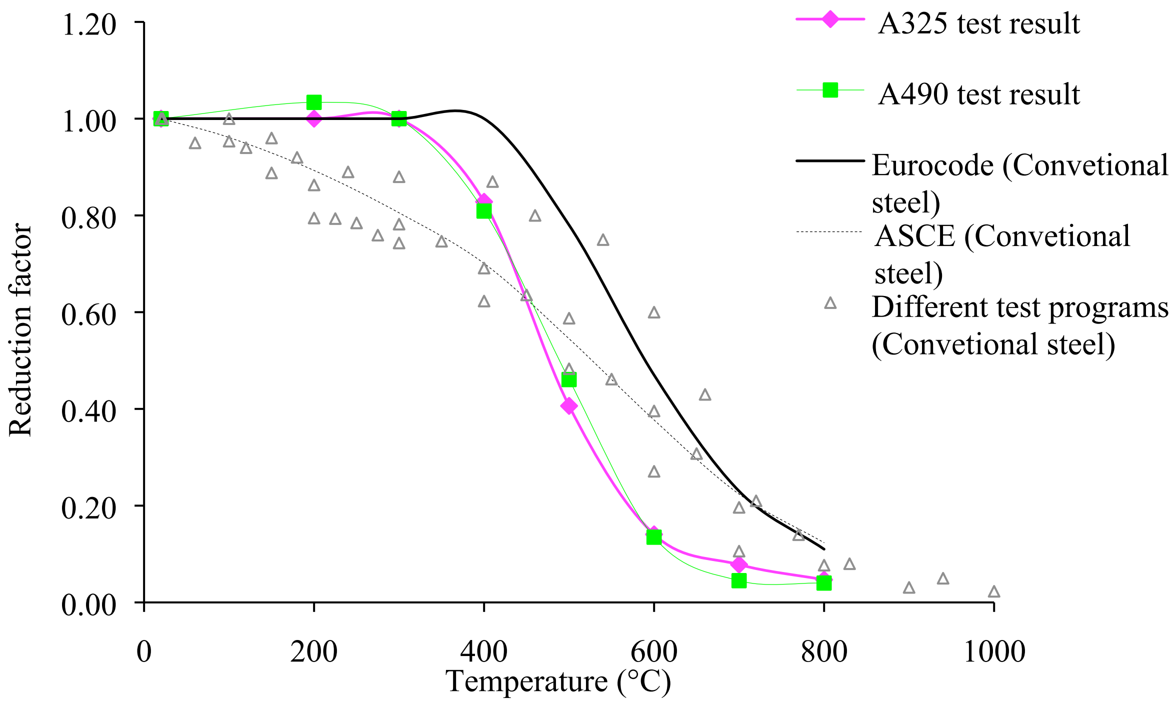



Figure 4.38 - Comparison of high temperature yield strength of A325 and A490 bolt steel to that of conventional steel

Most grades of conventional steels used for construction are low carbon steel manufactured by hot or cold forging method. Commonly used conventional steel grades are A36 and A992. The chemical composition and heat treatment of these steel grades are designed to achieve yield strength of 250MPa and 350MPa respectively. Whereas high strength A325 and A490 bolt steel is medium carbon steel produced by quenching and tempering. The chemical composition and heat treatment of these bolts steels are designed to achieve yield strength of 820MPa and 950Mpa respectively. After heat treatment crystalline structure produced in conventional steel is body centered cuboid of ferrite-cemenentite. This microstructure is dependent on the carbon content of steel and governs the strength properties of steel. High strength bolt steel are quenched to produce mertensite, a fine grain crystalline structure and tempered to increase ductility. The controlled tempering of bolts at designed temperature changes microstructure of steel to achieve required strength properties.

When exposed to higher temperature conventional steel loses its strength due to softening. At higher temperature the molecules of steel moves apart and intermolecular chemical bond in steel decreases resulting in softening and loss in strength. When high strength bolt steel is exposed to high temperature not only the steel softens but the microstructure of steel also changes from tempered martensite a fine grain structure to pearlite, a course grain structure which posses comparatively lesser load carrying capacity. Hence when exposed to temperature higher than tempering temperature these high strength bolt steel tend to lose strength much faster as compared to conventional steel. Thus it can be seen from the Figure 4.40 that the conventional steel tends to retain more strength than bolts when exposed to higher temperature. Comparison of high temperature strength properties of conventional steel and bolt steel indicates that properties of bolts produced by a quenching and tempering process are far more temperature-sensitive than conventional steels.

## 4.10 High temperature strength relations

Data generated from the mechanical property tests is utilized to develop mechanical property relationships for A325 and A490 steel. These properties are expressed in the form of empirical relationships over temperature range of 20-800$^{\circ}$C for ultimate tensile strength, 0.2% yield strength and elastic modulus. These empirical relationships were arrived at based on regression analysis. For the regression analysis, measured ultimate tensile strength, 0.2% yield strength and elastic modulus were used as response parameter with temperature as their predictor parameter. The accuracy of the statistical model is represented by coefficient of determination '$R^2$', that represents proportion of the sum of squares of deviations of the response values about their predictor (Mendenhall and Sincich 2007). $R^2$ values for the proposed equation lie within range of 0.95 to 1. As $R^2$ =1 represents a perfect fit equation, based on the $R^2$ the proposed equations can be considered best fit for the data obtained from the test results.

### 4.10.1 Ultimate strength and 0.2% yield strength

Test result for A325 and A490 steel given in Table 4.2 and 4.3 shows that ultimate strength and 0.2% yield strength of bolts is not affected by the temperature up to 300°C. The trend observed for reduction in ultimate strength and 0.2% yield strength is similar hence the following proposed equation can be used for determining temperature dependence of bolt ultimate and yield strength. At temperatures above 300°C, the strength reduction is drastic up to 700°C and rate of strength reduction decreases beyond 700°C. Hence to accommodate this variation of behavior of bolts strength with temperature three equations are proposed for each bolt type as follows;

**A325 steel**

For $20^\circ C \leq T \leq 300^\circ C$

$$F_{y,T} / F_{y,(20^\circ C)} = F_{u,T} / F_{u,(20^\circ C)} = 1$$

For $300^\circ C < T \leq 700^\circ C$

$$F_{y,T} / F_{y,(20^\circ C)} = F_{u,T} / F_{u,(20^\circ C)} = 0.00001T^2 - 0.0137T + 4.7007$$

For $700^\circ C < T \leq 800^\circ C$

$$F_{y,T} / F_{y,(20^\circ C)} = F_{u,T} / F_{u,(20^\circ C)} = 0.00001T^2 - 0.0137T + 4.75$$

**A490 steel**

For $20^\circ C \leq T \leq 300^\circ C$

$$F_{y,T} / F_{y,(20^\circ C)} = F_{u,T} / F_{u,(20^\circ C)} = 1$$

For $300^\circ C < T \leq 700^\circ C$

$$F_{y,T} / F_{y,(20^\circ C)} = F_{u,T} / F_{u,(20^\circ C)} = 0.000006T^2 - 0.0094T + 3.65$$

For $700^\circ C < T \leq 800^\circ C$

$$F_{y,T} / F_{y,(20^\circ C)} = F_{u,T} / F_{u,(20^\circ C)} = 0.000006*T^2 - 0.0094T + 3.72$$

Where,

$F_{u,T} / F_{y,T}$ is ultimate and yield strength at elevated temperature T

$F_{u,(20^\circ C)} / F_{y,(20^\circ C)}$ is ultimate and yield strength at $20^\circ C$

In Figure 4.39 and Figure 4.40 the strength reduction factors obtained from the proposed constitutive relationship for A325 and A490 bolts are compared to the test data and code and standard guideline. The proposed constitutive relationship appears to provide strength reduction factors that agree very well with test data. Also these relations take into account the strength advantage A325 and A490 bolts show due to chemical composition and heat treatment up to $400^\circ C$ which is neglected in Eurocode.

### 4.10.2 Elastic modulus

Like ultimate strength and 0.2% yield strength the elastic modulus of steel also reduces with increase on temperature. Test results indicate that elastic modulus is not affected by temperature up to 300˚C.

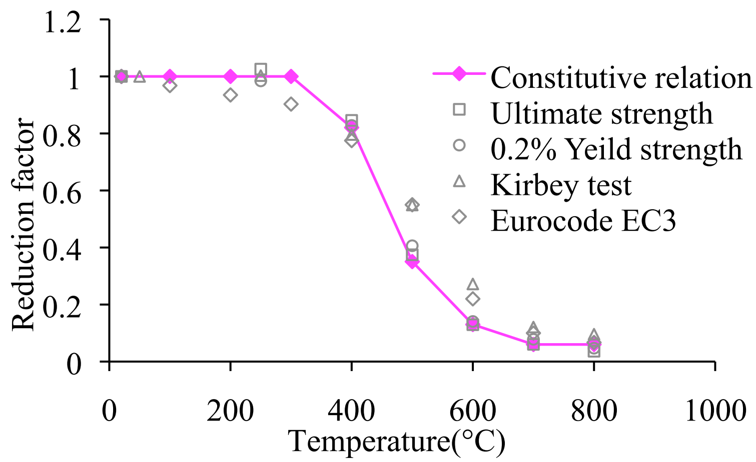



Figure 4.39 – Comparison of strength temperature relation for A325 bolts

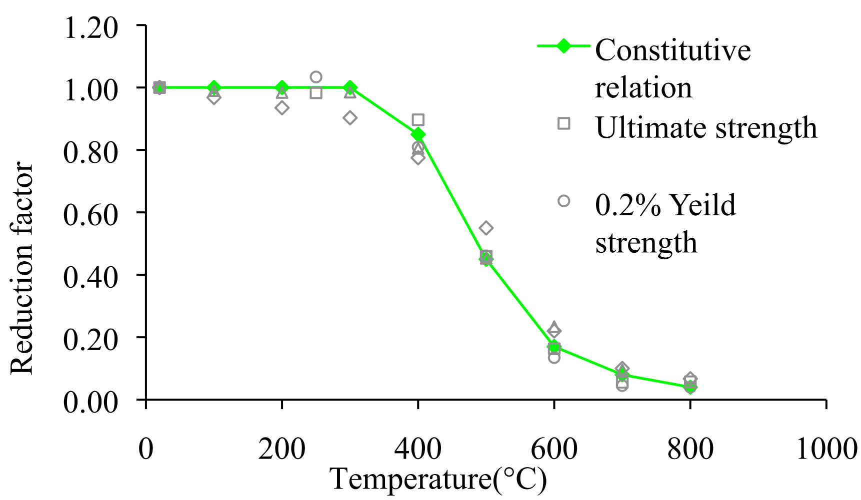



Figure 4.40 – Comparison of strength temperature relation for A490 bolts

Hence proposed relationship doesn't consider any reduction in elastic modulus value of A325 and A490 bolts up to 300°C. For the temperature range of 300 to 600°C, the elastic modulus decreases gradually with temperature. Hence to accommodate this variation of elastic modulus as a function of temperature two separate equations are proposed for temperature range of 20 to 300°C and 300 to 600°C as shown below.

**A325 steel**

For $20^\circ C \leq T \leq 300^\circ C$

$$E_T / E_{20^\circ C} = 1$$

For $300^\circ C < T \leq 600^\circ C$

$$E_T / E_{20^\circ C} = 1.95 - 0.0034T$$

**A490 steel**

For $20^\circ C \leq T \leq 300^\circ C$

$$E_T / E_{20^\circ C} = 1$$

For $300^\circ C < T \leq 600^\circ C$

$$E_T / E_{20^\circ C} = 1.6027 - 0.0025T$$

Where

$E_T$ is elastic modulus at elevated temperature T

$E_{20^\circ C}$ is elastic modulus at 20$^\circ$C

Comparison of the elastic modulus reduction factor proposed by these constitutive relations with the test data is illustrated in Figure 4.41 and Figure 4.42.

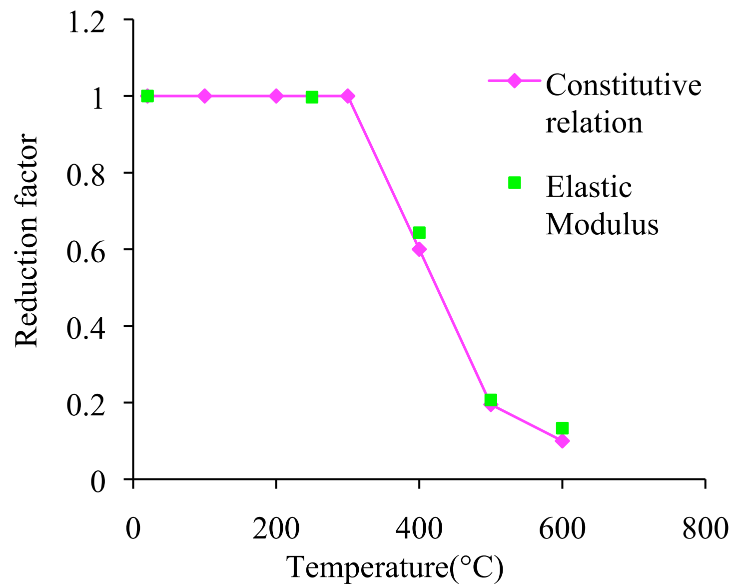



Figure 4.41 – Comparison of elastic modulus temperature relation with test data for A325 bolts

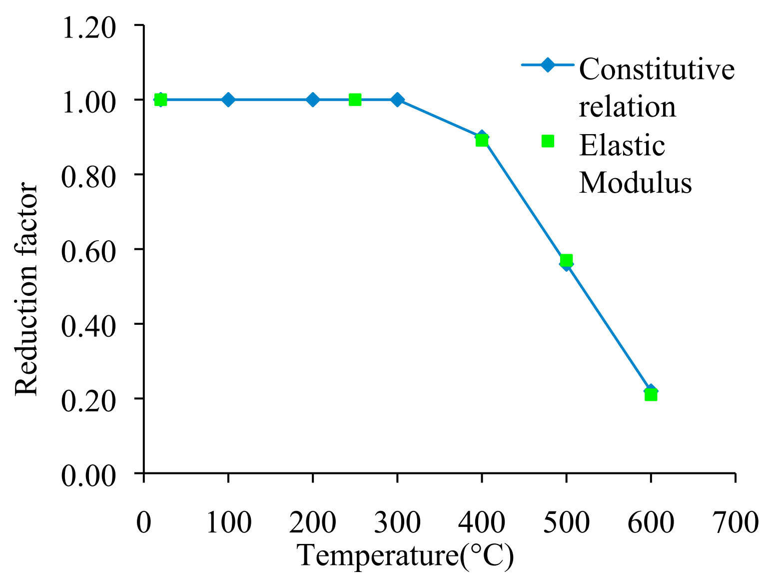



Figure 4.42 – Comparison of elastic modulus temperature relation with the data for A490bolts

### 4.11 Summary

The variation of strength properties of high strength steel used in A325 and A490 bolts is predicted. The strength tests were performed to evaluate shear strength, ultimate tensile strength, yield strength, and elastic modulus of A325 and A490 bolts at elevated temperatures. High temperature stress-strain curves are generated for A325 and A490 steel. Data from the tests is used to propose high temperature strength relations for A325 and A490 bolts.

From the test results it can be inferred that the strength properties of steel at elevated temperatures is influenced by chemical constitution and heat treatment of A325 and A490 bolts. Comparison of behavior of A325 with A490 bolts and bolt steel with conventional steel at elevated temperature indicates that high temperature properties of steel are highly influenced by the heat treatment of steel. A490 bolts are observed to retain higher strength and elastic modulus than A325 bolts at elevated temperature due to the strength advantage it gains from the heat treatment.

Stress-strain relation of A325 and A490 bolts indicates the increase in ductility of steel with increase in temperature. However it was observed that due to higher carbon content of A490 bolts, it behaves comparatively less ductile that A325 bolts. Comparison of test results with published test data and high temperature strength relations specified in codes and standards indicate influence of alloying elements such as molybdenum and chromium in enhancing high temperature mechanical properties of steel. Also, it is studied that the constitutive relations given by Eurocode and British code guidelines are based either on assumption or on limited test data. Conservative and unrealistic nature of these relations is discussed and to fill this knowledge gap new constitutive relations are proposed. These constitutive relations to determine high temperature mechanical properties of A325 and A490 steel can be used for modeling of connections or steel frames subjected to fire exposure.

**Chapter 5**

## Conclusions and Recommendations

### 5.1 Summary

High temperature thermal and mechanical properties of high strength steel are critical for predicting the fire response of bolted connections. Currently there is lack of high temperature property data in high strength bolts and often high temperature properties of conventional steel are used for these high strength steel bolts. To overcome this knowledge gap on high temperature properties, thermal and mechanical properties of high strength, Grade A325 and A490 steel bolts were measured. Commercially available equipments, 'HotDisk (TPS2500)' and 'Thermomechanical analyzer (Q400)', were used to measure high temperature thermal properties of steel in both heating as well as cooling phase. Thermal conductivity and specific heat were evaluated for A36, A325 and A490 steel in 20 to 735°C range, while as thermal expansion was measured in 20 to 1000°C range.

High temperature mechanical property tests were conducted to evaluate ultimate strength, shear strength and elastic modulus of high strength bolts in 20 – 800°C temperature range. This measured data provided augmented information on the high temperature stress-strain curves of A325 and A490 bolts.

Data from high temperature thermal and mechanical property tests was utilized to developed high temperature relations for thermal and mechanical properties of A325 and A490 bolts in heating and cooling phases. These relations can be used as input data in computer programs for evaluating fire response of bolted connections and structural systems.

## 5.2 Key Findings

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

- There is very limited information on the high temperature thermal and mechanical properties of bolts, specifically those fabricated with high strength steel.
- The thermal conductivity, specific heat and thermal expansion of high strength steel vary with temperature, chemical composition specifically carbon, and phase transition that occurs at elevated temperatures. Specifically,
    - A490 steel possesses lower thermal conductivity and higher specific heat than conventional steel as well as the A325 steel.
    - Thermal conductivity and specific heat of steel are invariant with the direction of heat flow and these properties remains same both in heating and cooling phase.
    - Thermal strain of conventional and high strength steel is constant throughout the cooling phase and follows a different pattern than that during heating phase.
- Strength and stiffness properties of high strength steel bolts at elevated temperature vary with the chemical composition as well as heat treatment of steel. Specifically,
    - No significant loss in ultimate strength and stiffness is observed for temperatures less than tempering temperature. As temperature increases above tempering temperature, strength reduces drastically to 5% of its room temperature capacity at 700°C and remains almost constant for 800°C.
    - A490 bolts retain higher strength than A325 bolts at elevated temperature.
    - Ductility of bolts increases considerably with increase in temperature above 500°C.

- High strength bolts possess higher strength than conventional steel up to tempering temperature however at higher temperatures strength of high strength steel bolts reduces drastically. Thus these heat-treated high strength bolts are more temperature sensitive than conventional structural steel.
- Data from high temperature thermal and mechanical property tests were utilized to propose high temperature property relations for Thermal conductivity, Specific heat, Thermal expansion, Ultimate and yield strength, Shear strength and Elastic modulus.

These property relations can be useful for undertaking fire resistance analysis of bolted connections.

### 5.3 Recommendations for future research

While this study has advanced the state-of-the-art with respect to thermal and mechanical properties of high strength A325 and A490 bolts at elevated temperatures, further research is required to fully characterize the effect of chemical composition and heat treatment on high temperature properties of A325 and A490 bolts. The following are some of the key recommendations for the research in this area:

- Chemical composition specifically carbon, chromium and molybdenum content of steel has influence on thermal and mechanical properties of steel. Hence more experimental studies are required to understand influence of various alloy elements on the high temperature thermal properties of bolts.
- Chemical composition of high strength steel and its heat treatment have significant influence on the high temperature properties of high strength steel. Hence in order to

determine that the proposed high temperature property relations are applicable to all A325 and A490 bolts available in the market manufactured by different manufacturers, it is highly important to explore more on the high temperature mechanical properties of A325 and A490 bolts from different manufacturers.

- Tempering temperature and chemical alloys such as chromium, molybdenum can improve fire response of bolts. Hence a study should be carried out to develop fire resistant bolts that can perform better than conventional and high strength steel bolts at elevated temperature.
- To establish the strength of bolted connection after fire it is essential to perform residual strength test on A325 and A490 bolts. To perform residual tension or shear test on the high strength steel bolts, the designed test setup can be used. The test specimen in these tests shall be heated to target temperature and then cooled down to room temperature. Mechanical loading should be applied on this air cooled specimen until failure and the load-deflection curves can be recorded to generate data on residual stress-strain curve, ultimate and yield strength, shear strength and elastic modulus of high strength steel bolts.

# Appendix

## Appendix

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### Appendix A.1

Constitutive relationships for high-temperature properties of steel as provided in EC 3 - Design of steel structures, Part 1-2: General rules - Structural fire design constitutive relationships for structural steel (2005).

### 1.1 High temperature mechanical properties of bolts

Following formulas are used to determine design shear and bearing resistance of bolt at elevated temperature.

Fire design resistance of bolts loaded with shear force

$$F_{v,t,Rd} = K_{b,\theta} F_{v,Rd} (\gamma_M / \gamma_{M,fi})$$

Fire design resistance of bolts loaded in bearin

$$F_{b,t,Rd} = K_{b,\theta} F_{b,Rd} (\gamma_M / \gamma_{M,fi})$$

Where

$F_{v,t,Rd}$ is determined from table 3.4 EN1993-1.8 is the design shear resistance of the bolt shear plane calculated assuming that the shear plane passes through the threads of the bolt (table 3.4 of EN1993-1-8)

$F_{b,t,Rd}$ is determined from table 3.4 EN1993-1.8

$K_{b,\theta}$, is the reduction factor determined for the appropriate bolt temperature (Table2.1 )

$\gamma_{M,fi}$ is the partial factor at fire temperature

**1.2 High temperature thermal properties of bolts**

Thermal Conductivity (W/m.°K)

For T < 800°C

$$k_s = 54 - 3.33 \times 10^{-2} T$$

For T ≥ 800°C

$$k_s = 27.3$$

Specific Heat (J/kg.°K);

For T < 600°C

$$c_s = 425 + 7.73 \times 10^{-1} T - 1.69 \times 10^{-3} T^2 + 2.22 \times 10^{-6} T^3$$

For 600°C ≤T < 735°C

$$c_s = 666 + \frac{13002}{738 - T}$$

For 735°C ≤T < 900°C

$$c_s = 45 + \frac{17820}{T - 731}$$

For 900°C ≤T < 1200°C

$$c_s = 650$$

These empirical formulas can simulate sudden increase in specific heat at 723°C that occurs due to the phase change of steel from BCC crystal structure to FCC structure.

Thermal Strain (per °C);

For T < 750°C

$$\Delta l/l = 1.2x10^{-5}\ T + 0.4x10^{-8} T^2 - 2.416x10^{-4}$$

For 750°C ≤ T ≤ 860°C

$$\Delta l/l = 1.1x10^{-2}$$

For 860°C ≤ T ≤ 1200°C

$$\Delta l/l = 2x10^{-57}\ T + 6.2x10^{-3}$$

**Appendix A.2**

Following equations are provided in ASCE manual (1992) to calculate thermal properties of steel.

Thermal Conductivity (W/m.°K);

For T < 900°C

$$k_s = -0.022T + 48$$

For T > 900°C

$$k_s = 28.2$$

Specific Heat (J/kg.°K);

For T < 650°C

$$c_s = (0.004T+3.3)\ 10^6/\rho_s$$

For 650°C ≤T < 725°C

$$c_s = (0.068T+38.3)\ 10^6/\rho_s$$

For 725°C ≤T < 800°C

$$c_s = (-0.086T+73.35)\ 10^6/\rho_s$$

For T > 800°C

$$c_s = 4.55x\ 10^6/\rho_s$$

where, $\rho_s$ is the density of steel.

The maximum specific heat obtained from these equations is 1500(J/kg.°K) at 750°C.

Thermal Strain (per °C);

For T < 1000°C

$$\Delta l/l = (0.004T+12)\ 10^{-6}$$

For T >1000°C

$$\Delta l/l = 16x10^{-6}$$

# Reference

## Reference


1. Beitel J. and Iwankiw N. (2005), "Analysis of Needs and Existing Capabilities for Full-Scale Fire Resistance Testing", National Institute of Standards and Technology GCR 02-843-1

2. Kirby B. (1992), "The Behaviour of High Strength Grade 8.8 Bolts in Fire", British Steel Technology, Swinden

3. Barnett J., Gewain R., Gilsanz R. and Nelson H. (2002), "Chapter 4: WTC 4, 5, and 6", Federal Emergency Management Agency

4. Eurocode 3: Design of Steel Structures, Part 1.2: General Rules Structural Fire Design (drafts), Document CEN, European Committee for Standardization, 1995

5. Nacewicz R. (2006), "Investigation of Fire Impact on Structural Steel through Case Studies"

6. Yu L. (2006), "Behavior of Bolted Connections During and After a Fire" University of Texas Austin

7. Gonzalez F. and Lange J. (2009), "Behavior of High Strength grade 10.9 bolts under fire Conditions" International Conference – Applications Structural Fire Engineering, Prague

8. Luecke W. (2005), "Mechanical Properties of Structural Steels" Federal Building and Fire Safety Investigation of the World Trade Center Disaster NIST NCSTAR 1-3D

9. S. Keira, F. Furumura, and T. Ave (1993), "Tests of Fire-Resistant Bolts and Joints" Journal of Structural Engineering, Vol. 119, No. 11

10. Kodur V.K.R. and Dwaikat M. (2010), "High-Temperature Properties of Steel for Fire Resistance Modeling of Structures", Journal of Materials in Civil Engineering, Vol. 22, No. 5

11. Buchanan A. (2001), "Structural Design for Fire Safety" John Willey and Sons Ltd., New York, USA. 420p.

12. Sagaradze V. (2005), "Effect of carbon content on the sterengh of carburized steel", Urals Polytechnical Institute, Termicheskaya Obrabotka Metallov No. 3,

13. Adebiyi, D., Adewuyi B. and Afonja A. (2009), "Effect of Carbon Content on the Grain Refinement of Locally Produced Plain Carbon Steel by Thermal Cycling", Journal of Applied Sciences Research, INSInet Publication

14. Al-Jabri K., Buick Davisonb J., Burgessb I. (2008), "Performance of beam-to-column joints in fire—A review", Fire Safety Journal 43

15. ASTM A325/A325M – 04b, "Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength", Annual Book of ASTM Standard, V.01.08

16. ASTM A490/A490M – 04a,'Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength", Annual Book of ASTM Standard, V.01.08

17. ASTM A27/A27M – 05, 'Standard Specification for Steel Casting, Carbon, for General Application", Annual Book of ASTM Standard, V.01.02,

18. ASTM A36/A36M – 05, 'Standard Specification for Carbon Structural Steel", Annual Book of ASTM Standard, V.01.04

19. ASCE (1992), "Structural Fire Protection." Manual No. 78, American Society of Civil Engineers, Structural Division, ASCE committee on fire protection, New York.

20. Kodur V., Dwaikat M., and Fike R. (2010), "High-Temperature Properties of Steel for Fire Resistance Modeling of Structures", Journal of Materials in Civil Engineering, Vol. 22, No. 5.

21. Touloukian Y. (1972), "Thermal Radiative Properties for Non Metallic Solids" Thermal Physical Properties, Vol. 8, pp. 142.

22. Yafei S., Yongjun T., Jing S., and Dongjie N. (2009), "Effect of Temperature and Composition on Thermal Properties of Carbon Steel", Control and decision conference, Chinese.

23. He Y., (2005), "Rapid thermal conductivity measurement with a hot disk sensor.", Assembly Materials Characterization Laboratory, Intel Corporation, Arizona.

24. Anis-ur-Rehman M., and Maqsood A. (2003), "Measurement of Thermal Transport Properties with an Improved Transient Plane Source Technique." International Journal of Thermophysics, Vol. 24, No. 3.

25. Kodur V., and Harmathy T. (2008), "Properties of Building Material.", Handbook of Fire Protection Engineering, 2008 edition.

26. Bentz D. and Prasad K. (2007), "Thermal Performance of Fire Resistive Materials I. Characterization with Respect to Thermal Performance Models." Report BFRL – NIST 7401. Gaithersburg, MD.

27. Mergia K., and Boukos N., (2008), "Structural, thermal, electrical and magnetic properties of Eurofer 97 steel", Journal of Nuclear Materials.

28. Log T., and Gustafsson S., (1995), "Transient Plane Source (TPS) Technique for Measuring Thermal Transport Properties of Building Materials" FIRE AND MATERIALS, VOL. 19.

29. Kodur V., and Khaliq W., (2003), "Effect of Temperature on Thermal Properties of Different Types of High Strength Concrete" Journal of Materials in Civil Engineering, v. 15, no. 2, NRC, Canada.

30. Crook W. (1927), "Alloy steel and their uses", Journal of chemical education, Vol 4, No 5.

31. Cooke G. M. E. (1988), "An Introduction to the Mechanical Properties of Structural Steel at Elevated Temperatures", Fire Safety Journal, 13 (1988) 45 – 54

32. Dai Q., Wang A., Cheng X., Cheng L. (2001), "Effect of alloying elements and temperature on strength of cryogenic austenitic steels", Materials Science and Engineering A311 205–210

33. Gulyaev A. (1967), Effect of Heat Treatment and Alloying Elements on the Structural Strength of Steel", Translated from Metallovedenie Termicheskaya Obrabotka Metallov, No. i0, pp. 66-72, October, 1967

34. Spyrou S., Davison B., Burgess N., and Plank R. (2004), "Experimental and analytical studies of steel joint components at elevated temperatures" FIRE AND MATERIALS Fire Mater. 2004; 28:83–94 (DOI: 10.1002/fam.846)

35. Spyrou S., Davison B., Burgess N., and Plank R. (2004), "Experimental and analytical investigation of the 'tension zone' components within a steel joint at elevated temperatures", Journal of Constructional Steel Research 60 (2004) 867–896

36. ASM International (2006), "Fundamentals of the Heat Treating of Steel" Practical Heat Treating, Second Edition

37. Li1 G., JiangS., Yin Y., Chen K., and LiM. (2003), "Experimental Studies on the Properties of Constructional Steel at Elevated Temperatures" JOURNAL OF STRUCTURAL ENGINEERING

38. Conton R. (1975), "High strength steel alloy" United States Patents

39. Kirby B. and Preston R. (1998), "High Temperature Properties of Hot-rolled, Structural Steels for Use in Fire Engineering Design Studies", Fire Safety Journal, 13 (1988) 27 – 37

40. Kraftmakher Y. (1994), "High-temperature specific heat of metals" Eur. J. Phys. 15 329

41. Éstrin É. and Mogutnov B. (2004), "Mechanical Properties of High-Carbon Alloy Steel near the Phase-Transformation Temperature" Doklady Physics, Vol. 49, No. 7, 2004, pp. 422–425.

42. Outinen J. and Makelainen P. (2002), "Mechanical Properties of Structural Steel at Elevated Temperatures and After Cooling Down", Second International Workshop, Structures in Fire (2002)

43. Twilt L. (1988), "Strength and Deformation Properties of Steel at Elevated Temperatures: Some Practical Implications", Fire Safety Journal, 13 (1988) 9 - 15 9

44. Smoljan B. (2003), "Prediction of mechanical properties and microstructure distribution of quenched and tempered steel shaft", 12th international Scientific Conference, Achievements in Mechanical and Material Engineering