



وزارة التعليم العالي والبحث العلمي
جامعة بابل
كلية الهندسة

تأثير التعرض إلى لهب النار على سلوك الانحناء ومقاومة القص
لعتبات خرسانية مسلحة اعتيادية المقاومة و عالية الأداء.

رسالة تقدم بها الطالب

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بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ

وَيَسْأَلُونَكَ عَنِ الرُّوحِ قُلِ الرُّوحُ مِنْ أَمْرِ رَبِّي وَمَا أُوتِيتُمْ مِنَ الْعِلْمِ إِلَّا قَلِيلًا

صَدَقَ اللَّهُ الْعَظِيمُ

سورة الإسراء، الآية (85)

Abstract

With the increasing use of high – performance concrete in buildings construction, the risk of exposing it to high temperatures during fires has increased significantly. The exposure of reinforced concrete beams to fire flame causes changes in their structural behavior.

This research aims to study the flexural behavior and shear strength of reinforced concrete beams made of normal strength concrete (NSC) and high performance concrete (HPC) with compressive strength (30,78) MPa respectively at (28) days age, and assessing the residual flexural and shear strengths after exposure to fire flame.

Twenty reinforced concrete shallow beam specimens having the dimensions (100*100*1000)mm and reinforced with both tension and web reinforcement to yield flexural failure with steel ratio of (0.0126).

Twenty reinforced concrete deep beam specimens having the dimensions (100*150*600)mm and reinforced with tension steel only (no web reinforcement) to present shear failure, and having steel ratio of (0.004).

The concrete specimens were subjected to fire flame at temperature levels of (400 °C) and (700 °C) at two periods of exposure (1, 1.5) hour then cooled either by air and water. Firing and testing the beam specimens were conducted at age of (60) days.

Residual flexural and shear strengths, load- deflection curves and crack pattern, for the two types HPC and NSC after exposure to fire flame were all recorded and discussed.

Results indicate remarkable reduction in flexural and shear strengths after exposure to fire flame. The residual flexural strength is (84- 88%), (70- 72%) for HPC, and (86- 91%), (84- 88%) for NSC with (1.0) and (1.5) hour exposure periods respectively at fire temperature (400 °C), while the residual

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at fire temperature (700 °C) is (50-58%), (37-38%) for HPC, and (53-60%), (26-41%) for NSC with (1.0) and (1.5) hour exposure periods respectively. The residual shear strength is (90-98%), (78-93%) for HPC, and (78- 93%), (75- 77%) for NSC with (1.0) and (1.5) hour exposure periods respectively at fire temperature (400 °C), while the residual at fire temperature (700 °C) is (50-55%), (40-45%) for HPC, and (54-57%), (38-46%) for NSC with (1.0) and (1.5) hour exposure periods respectively.

Load – deflection curves indicate changing response with the fire exposure, but the behavior of the HPC beams remain superior in spite of their sensitivity to high temperatures.

It is found that the ACI 318/ 2005-code equations are safe to predict bending moment capacity and shear strength for fire temperature (400 °C), but become unable to predict them at fire temperature (700 °C).

The shear strength prediction of reinforced concrete beams without web reinforcement by the three codes (ACI, Canadian, New Zealand), Zsutty equation and Rebeiz equation, it is found that the equations are unconservative to predict shear strength after exposure to fire flame except the New Zealand code equation which was found able to predict shear strength after exposure to fire flame temperature for the two types of concrete HPC and NSC.

The results of this research show that the residual bending moment capacity for HPC is more sensitive to fire flame temperature than NSC, while the residual shear strength for NSC is more sensitive to fire flame temperature than HPC.

Flexural failure was characterized by widening of the vertical cracks and subsequent crushing of the top fiber concrete, while shear failure occurred by sudden excessive opening wide diagonal cracks and characterized as true shear failure that occurs in beams with $a/d \leq 1$.

الخلاصة

زيادة استخدام الخرسانة العالية الأداء في أنشاء الأبنية جعل تعرضها إلى درجات الحرارة العالية إثناء الحرائق يكون بالغ الأهمية. تعرض العتبات الخرسانية المسلحة إلى لهب النار المباشر يسبب تغيرات في سلوكها الإنشائي، هذا البحث يهدف إلى دراسة سلوك الانحناء ومقاومة القص لعتبات خرسانية مسلحة: اعتيادية المقاومة (NSC) وخرسانة عالية الأداء (HPC) وبمقاومة انضغاط (30، 78) ميكا باسكال بعمر (28) يوم، مع تقييم مقاومة عزم الانحناء والقص المتبقية بعد التعرض إلى لهب النار.

لقد جرى تحضير عشرين نموذج من العتبات الخرسانية المسلحة الضحلة بأبعاد (1000*100*100) ملم بتسليح شد وأطواق للحصول على فشل الانحناء و بنسبة حديد تسليح طولي (0.0126)، كذلك جرى صب عشرين نموذج من العتبات الخرسانية المسلحة العميقة بأبعاد (600*150*100) ملم بتسليح شد فقط (بدون تسليح الأطواق) لتمثل فشل القص وبنسبة حديد تسليح (0.004).

عرضت النماذج الخرسانية إلى لهب النار في مستويات حرارة (400 ، 700) درجة مئوية في فترتين من التعرض (1، 1.5) ساعة، ثم بردت بواسطة الهواء أو الماء. وجرى حرق و فحص نماذج العتبات في عمر (60) يوم.

من خلال الفحوصات والإعمال المختبرية تم دراسة مقاومات عزم الانحناء والقص المتبقية، منحنيات الحمل والانحراف وأنماط التشقق لكلا النوعين من الخرسانة العالية الأداء والخرسانة الاعتيادية المقاومة بعد التعرض إلى لهب النار المباشر وتم تسجيل وعرض ومناقشة النتائج.

لقد أشرت النتائج انخفاض ملحوظ في مقاومات القص و عزم الانحناء بعد التعرض إلى لهب النار، حيث كانت مقاومة عزم الانحناء المتبقية للخرسانة العالية المقاومة هي (84-88%) و (70-72%) و للخرسانة الاعتيادية المقاومة هي (86-91%) و (84-88%) بفترات تعرض (1، 1.5) ساعة على التوالي بعد التعرض إلى درجة حرارة (400) درجة مئوية، بينما للخرسانة العالية المقاومة وبعد الحرق إلى (700) درجة مئوية مقاومة عزم الانحناء المتبقية هي (50-58%) و (37-38%) و للخرسانة الاعتيادية المقاومة هي (53-60%)، (26-41) بفترات تعرض (1، 1.5) ساعة على التوالي.

إما مقاومات القص المتبقية للخرسانة العالية المقاومة هي (90-98%) و (78-93%) و للخرسانة الاعتيادية المقاومة هي (78-93%) و (75-77%) بفترات تعرض (1، 1.5) ساعة على التوالي بعد التعرض إلى درجة حرارة (400) درجة مئوية، بينما للخرسانة العالية المقاومة وبعد الحرق إلى

(700) درجة مئوية مقاومة القص المتبقية هي (50-55%) و (45-45%) و للخرسانة الاعتيادية المقاومة هي (54-57%) و (83-46%) بفترات تعرض (1، 1.5) ساعة على التوالي. إما منحنيات الحمل – الانحراف كانت الاستجابة متغيرة مع درجة التعرض للنار، لكن سلوك العتبات العالية المقاومة تبقى الأفضل على الرغم من التحسس لدرجات الحرارة العالية. النتائج العملية وجدت إن معادلات الكود الأمريكي أمينة لتنبؤ سعة عزم الانحناء ومقاومة القص لحد درجة حرارة (400) درجة مئوية، لكن تصبح قاصرة للتنبؤ في درجة حرارة (700) درجة مئوية. النتائج العملية لمقاومة القص ولعتبات خالية من الأطواق الحديدية قورنت مع (الكود الأمريكي، الكندي، النيوزلندي) ومعادلات الباحث زوتي ورييز. نتائج المقارنة وجدت إن المعادلات تكون غير دقيقة لتنبؤ مقاومة القص بعد التعرض إلى حرارة لهب النار ماعدا معادلة الكود النيوزلندي وجدت قدرة لتنبؤ مقاومة القص بعد التعرض إلى حرارة لهب النار ولكلا النوعين من الخرسانة عالية المقاومة واعتيادية المقاومة نتائج هذا البحث بينت بان سعة عزم الانحناء أكثر حساسية إلى لهب النار بالنسبة للخرسانة العالية المقاومة من الخرسانة الاعتيادية المقاومة بينما مقاومة القص أكثر حساسية إلى لهب النار بالنسبة للخرسانة الاعتيادية من الخرسانة العالية المقاومة. شكل الفشل بالنسبة للانحناء تميز بتوسع التشققات العمودية يتبعها تكسر الحافة العليا للخرسانة بينما فشل القص يحدث بتوسع التشققات القطرية وميز كفشل قص حقيقي للنماذج ذو نسبة فضاء العمق إلى العينة اقل من (1).

INTRODUCTION

1.1 Introduction

In buildings, high performance concrete structural members are designed to satisfy the requirements of serviceability and safety limit states. One of the major safety requirements in building design is the provision of appropriate fire safety measures for structural members (**National Building Code of Canada, 1995**).

With the development and application of high-performance concrete (HPC), understanding of its behavior when subjected to fire is needed to insure its safe application (**Lin et- al, 1996**)

A few recent, dramatic incidents have emphasized the need for safety concerning reinforced concrete structures made of both normal strength concrete (NSC) and high-performance /high strength concrete (HPC/HSC). For example, the fire in the railway tunnel across the English channel, fall 1996, in addition to the tunnels, other structures and infrastructure are at risk of having a fire, such as viaducts, high-rise buildings, containment shells (**Alberto Meda et- al, 2002**). Many studies devoted to understanding the physical and mechanical properties of HPC is more temperature-sensitive than ordinary concrete (**Zhang, 2000**), even if some results are a bit contradictory (**Chan et- al, 1996**)

The fire safety of RC structures largely depends on their fire resistance, which in turn depends on the combustibility and fire resistance of their main structural elements, i.e., beams and columns. As structure elements, beams are subjected to flexural and shearing loads. The residual bending moment and shear force capacities of fire-damaged concrete beams are important factors in determining the safety of the structure.

1.2 Research Significance

There is indeed little research about temperature gradient and exposure of concrete in direct contact with fire flames.

The present work is an attempt to investigate the effect of exposure of (HPC) to fire flame on its flexural behavior and shear strength.

Two concrete mixes and (40) reinforced concrete beams were investigated at different temperature levels, different exposure periods and different shear span to depth ratio (a/d).

Many variables are encountered in this investigation, which covers the following aspects:

- 1- studying the fire flame effect on flexural behavior and shear strength of reinforced NSC and HPC beams with different steel ratios.
- 2- studying the fire flame effect on the immediate deflection of the reinforced NSC and HPC beams and comparing the results with control beams.
- 3- studying the fire effect on the mechanical properties of NSC and HPC, such as compressive strength, splitting tensile strength, modulus of rupture, and modulus of elasticity when cooled in air and water.

1.3 Research Layout:

In this investigation, the research work presented in this thesis is given through five chapters:

Chapter one provides a general introduction

Chapter two is concerned with literature review on high-performance concrete, including the effect of fire endurance on flexural behavior and shear strength of reinforced concrete beams.

Chapter three deals with materials and experimental work, which includes the program of testing.

Chapter four includes analysis of test results and their discussion.

Chapter five contains conclusions obtained from the test results and some possible recommendations for further work.

REVIEW OF AVAILABLE RESEARCHES

2.1 Introduction

High performance concrete is not a revolutionary material, nor does it contain ingredients which are not used in the concrete considered this far (Neville, 1995). High performance concrete is a concrete made with appropriate materials combined according to selected mix design and properly mixed, transported, placed consolidated and cured. The resulting concrete will give excellent performance in the structure in which it will be placed, environment to which it will be exposed, and with loads to which it will be subjected for its design life, and with economical cost.

2.2 High-Performance Concrete

2-2-1 Definition

The old name of high performance concrete is " high strength concrete" which the compressive strength is forming the main demand of the concrete production. Another properties are added to the high strength of the structure such as durability, early strength, impermeability and modulus of elasticity of the concrete (Neville, 1995). High-Performance concrete is defined as concrete that meets special performance and uniformity requirements that can not always be achieved routinely by using conventional materials and normal mixing, placing, and curing practices. High-performance concrete is a relatively new material. Several different definitions of high-performance concrete have been proposed. Currently

there is no one definition that is universally accepted either within the United States or in other countries. Some of these definitions are summarized below (NISTR 6295, 1999)

1. Strategic High way Research Program (SHRP) definition :

a. High-performance concrete shall have one of the following strength characteristics:

1. 4-hour compressive strength greater than or equal to 20 MPa .
2. 24-hour compressive strength greater than or equal to 35 MPa.
3. 28-day compressive strength greater than or equal to 70 MPa.

b. High-performance concrete shall have a durability factor greater than 80 % after 300 cycles of freezing and thawing.

c. High-performance concrete shall have a water-cementitious materials ratio less than or equal to 0.35.

The SHRP definition encompasses specific strength, durability, and mixture proportioning characteristics. It should be noted that this definition was developed primarily to address requirements for high way construction.

2. NIST/ACI Workshop definition :

High-performance concrete is concrete having desired properties and uniformity that cannot be obtained routinely using only traditional constituents and normal mixing, placing, and curing practices. As examples these properties may include:

1. Ease of placement and compaction without segregation.
2. Enhanced long-term mechanical properties.
3. High early-age strength.
4. High toughness.
5. Volume stability.
6. Long life in severe environments.

This is a more general definition that attempts to include a variety of concretes having special properties not attainable by ordinary concrete.

3. University of Tokyo definition:

In this definition, high-performance concrete is characterized as a “forgiving concrete” that compensates for poor construction practices and structural detailing, and has the following features;

1. Ability to fill forms with little or no external compactive effort;
2. Cohesive mixture with low segregation;
3. Minimum cracking at early ages due to shrinkage and thermal strains;
4. Sufficient long-term strength and low permeability.

This definition is a reflection of the Japanese emphasis on constructability as well as strength and durability of concrete.

4. Prestressed Concrete Institute definition (PCI Committee on Durability 1994):

High-performance concrete is concrete with or without silica fume having a water-cement ratio of 0.38 or less, compressive strength at or above 55.2 MPa (8 000 psi) and permeability 50 % lower (by AASHTO T-259 or T-277 methods) than that of conventional mixtures.

5. Civil Engineering Research Foundation definition (CERF Technical Report 1994):

Unlike conventional concrete, high-performance concrete meets one or more of these requirements:

1. Places and compacts easier.
2. Achieves high strengths at early ages.
3. Exhibits superior long-term mechanical properties such as strength, resistance to abrasion or impact loading, and low permeability.
4. Exhibits volume stability and thus deforms less or cracks less.

5. Lasts longer when subjected to chemical attack, freezing and thawing, or high temperatures.

6. Demonstrates enhanced durability.

This definition is an outgrowth of the earlier NIST/ACI workshop definition.

With development and application of high performance concrete (HPC), understanding of its behavior when subjected to fire is needed to insure its safe application (**Jahren, 1989**). HPC exhibits superior performance in many aspects, e.g. possesses high strength, durability, and workability (**Mehta and Aitcin, 1990**). However, HPC inferior thermal behavior, with its dense microstructure, has not been fully understood. Poor fire behavior is a threat to HPC practical application in many types of engineering structures, including offshore platform, tunnels and high-rise buildings. Fire behavior is the main obstacle to HPC further development. In many countries, research projects are either planned or in process on this specific topic (**Jensen et- al, 1997**).

2-2-2 The Benefits of HPC

There are potential benefits arising from the use of high performance concrete in structures (**The Benefits of High Performance Concrete, 1998**).

1- More slender elements can be obtained.

2- Lower concrete volumes.

3- Reduced percentage of reinforcement.

4- Smaller and less foundations.

5- More rapid construction.

6- Reduced maintenance improved durability by increased strength, lower permeability, reduced chloride ion penetration, corrosion inhibition, increasing resisitivity up to more than 500%, sulfate resistance.

2.3 Mechanical Properties of High Performance Concrete

An extended comprehensive knowledge of the fundamental properties of High-Performance concrete is necessary. The mechanical properties of High-Performance concrete are among those fundamental properties. A study was undertaken by **(Carette and Malhotra, 1983)** to investigate early age strength development of concrete incorporating fly ash and condensed silica fume. The amounts of the fume ranged from 0 to 20% by obtained weight of the Portland cement plus fly ash. A total of thirty 0.06 m³ concrete mixtures with water-(cement + fly ash) ratios ranging from 0.40 to 0.80 were made; 240 cylinders were tested in compressive and 180 prisms were tested in flexure. A supplementary series of six concrete mixtures was made to determine the effect of silica fume and fly ash on the long-term strength development of concrete. Test data showed that the incorporation of condensed silica fume increased the compressive strength of concrete at all ages as compared with the compressive strength of the control concrete (70% Portland cement +30% fly ash). At 7 days, the loss of compressive strength due to the partial replacement of cement by the fly ash was completely overcome by the addition of 10% condensed silica fume for concretes with water-(cement + fly ash) ratios ranging from 0.40 to 0.60; 15 to 20% silica fume was required for concretes with higher water-(cement + fly ash) ratios. At 28 day, regardless of the water-(cement + fly ash) ratios, the latter age appear to be impaired by the use of condensed silica fume indicating availability of sufficient lime for the fly ash pozzolanic activity. The strength development of silica fume concrete and its prediction under varying temperature conditions was tested by **(Wen et- al, 1989)** who investigated three silica fume concrete mixes. A temperature matched curing rig has been improved for simulating different temperature regimes.

The strength predictions of the SFC under changing temperature regimes show good agreement with test results, but adjustment of the empirical activation energy at high temperatures is necessary. Results indicated that the strength development under varying temperature conditions can be reasonably accurately predicated at an early age up to 70% of its 28-day strength at 20°C. The accelerating effect of high concrete temperature (>40°C) on the pozzolanic reaction of silica is significant. The early age splitting strength /cube compressive strength (s/c) relationship of the SFC is different from that of the ordinary concrete. This is a result of the differing contributions to the cube compressive and tensile strength by the superplasticizer and the silica fume, and by the two-stage strength development pattern of the SFC. Mixing methods may also have some effect.

Baalbaki et- al, 1991 studied the influence of different types of crushed rocks on high performance concrete. The relevant characteristics are; cleanliness, grain, size, shape, texture, and chemical reactivity. Three series of HSC of the same composition, but made with different aggregate (Dolomitic Limestone, Quartzite, and Sandstone), were tested for compressive strength and elastic modulus of different mixes at 28 and 91 days. Results showed that the elastic modulus of high-performance concrete is strongly influenced by the elastic properties of coarse aggregate. The good bond at the interface of the coarse aggregate and mortar results in a composite material, whose components are cement paste and coarse aggregate. However, in NSC, the transition weakness primarily conditions the elastic properties.

Xie et- al, 1995 investigated the mechanical properties of three high-strength concretes containing silica fume. The concretes were designed to have a 28 day compressive strength of 60, 90 and 120 MPa. They used cement content of 333.7, 391.6 and 449.4 kg/m³, w/c ratio of 0.321, 0.283

and 0.216 and silica fume of 40, 43.6 and 49.8 kg/m³ respectively. Compressive strength for cylinders, splitting tensile strength for cylinders, notched beam and triaxial compressive tests were performed for these contents.

Results indicated that although the tensile strength increases with the increase in compressive strength, high strength concrete exhibits less ductility than normal strength concrete in both tension and compression.

Hotton, 1984 presented the results of research on the influence of silica fume (SF) replacement by weight of cement on physical properties of concrete. Normal density concrete was designed with total cementitious materials content (cement plus silica fume) of 400 kg/m³. Mixes used throughout this study incorporated 0,10,15 and 20 percent SF.

Results showed that with water cementitious ratio of 0.35 and slump 60-75mm, compressive strength at 15% SF replacement was 75.2 MPa at 28 day and 76.4 MPa at 56 day. Beyond 56 day, the strength development of SF concrete practically stopped. He indicated that this was due to self-desiccation effects. Also Hotton concluded that the long term strength gain of SF concrete was very low, and 5-year strength was + 12% of the 28 day strength compared to 5% increase between 28 day and 5 year for reference Portland cement.

Iravani, 1996 also studied the mechanical properties of high-performance concrete. He used four different concrete mixes, type I and III Portland cements were used with (w/c + silica fume) ratios of 0.4, 0.24, 0.26, and 0.21, and silica fume contents of 0, 45, and 54 kg/m³ and superplasticizer with contents of 7.5, 14.9, and 18.1kg/m³ respectively. Results indicated that for high-performance concrete without silica fume, the ratio of compressive strength gain before 28-day increases and the ratio of compressive strength gain after 28-day decreases as the 28-day

compressive strength increases. The ratios of 7 to 28 days compressive strengths suggest that a significant strength gain occurs between 7 and 28 days for the ultra-high strength silica fume high-performance concretes. For high-strength silica fume, the mix with type III cement develops a higher compressive strength than the mix with type I cement for the same mix design. For ultra-high strength silica fume high-performance concrete, the mix with type III cement developed lower or equal compressive strength than the mix with type I cement for the same mix design.

2.4 Mechanisms of Shear Transfer in Reinforced Concrete Beams:

The behavior of reinforced concrete beams as stated in the (**ACI-ASCE Committee 426, 1973**) report depends on the method of shear transmission. Consideration of principal shear mechanisms is relatively recent and therefore relative contribution of these shear-carrying components is tentative.

The main types of shear transfer mechanisms are the following:

2-4-1 Shear Transfer by Concrete Shear Stress:

This occurs in uncracked members or in the uncracked portions of structural members. The interaction of shear stress with tensile and compressive stresses produces principal stresses, which may cause cracking or crushing failure of the concrete.

2-4-2 Shear Transfer by Aggregate Interlock:

This happens when shear must be transferred across a definite plane or surface where slip may occur. If the plane under consideration is an existing crack or interface, failure usually involves slippage or relative

movement along the crack or plane (**Bunni, 1994**) If the plane is located in monolithic concrete, a number of diagonal cracks occur across the interface and failure involves a truss action along the plane.

2-4-3 Dowel Action:

When the longitudinal reinforcement crosses a cracked part shearing displacement will be resisted by as dowel force in the bars. This gives rise to tensile stresses in the surrounding concrete, and these stresses produce splitting cracks along the reinforcement. This in turn decreases the stiffness of the concrete around the bar, and therefore the dowel force also decrease(**Bunni, 1994**).

2-4-4 Arch Action:

Arch action permits the transfer of vertical concentrated force to a reaction in a deep member, which reduces the contribution of the other types of shear transfer. The longitudinal tension reinforcement in this case acts as a tie (**Bunni, 1994**).

2-4-5 Shear Reinforcement:

The truss analogy is used to view the action of web reinforcement although this analogy helps to simplify design concepts and methods, it does not consider the various types of shear transfer mechanisms properly. In addition to any shear carried by stirrup itself, the shear reinforcement may contribute significantly to the capacity of the member by restricting the widening of inclined cracks, thus increasing the interface shear transfer, dowel action and arch action (**ACI-ASCE Committee 426, 1973**).

2.5 Reinforced Concrete Simple Deep Beams

Kong et- al, 1970 tested 35 simply supported reinforced concrete deep beams under two point top loads. The shear span to depth ratio (a/d) varied from 0.23 to 0.7. The effect of using seven different types of web reinforcement on deflection, crack widths, crack patterns, failure mode and ultimate loads in shear was studied.

It was found that the effectiveness of the various types of web reinforcement depends on the (l_n/d) and (a/d) ratios. Also, it was found that for low (l_n/d) and (a/d) ratios, only horizontal web reinforcement at close spacing closed to the bottom of the beam is effective.

Zsutty, 1971 recognized that the shear test data are not homogeneous due to the fact that there are two separate types of beam behavior. Accordingly, he segregated the data and performed separate regression for short beams and long beams. For long beams (i.e., slender beams), the ultimate shear strength prediction was found according to the Eq (2-1)

$$V_c = 2.2 (f'_c \rho_w d/a)^{1/3} b_w d \quad \dots\dots\dots(2-1)$$

Rebeiz, 1999 presents an alternative shear strength prediction equation for RC members. He uses the techniques of dimensional analysis, interpolation function, and multiple regression analysis. The analysis showed that the best model for the ultimate shear strength prediction was found according to the Eq (2-2).

$$V_c = [0.4 + 1.7 (f'_c \rho_w d/a)^{1/2}] b_w d \quad \dots\dots\dots(2-2)$$

Tan et- al, 1995 tested 19 reinforced concrete deep beams with compressive strengths in the range between 41 and 59 MPa. These beams were tested under two point top loading. All the beams were singly reinforced with main steel percentage $\rho = 1.23$ percent and with nominal percentage of shear reinforcement $\rho_v = 0.48$ percent. The beams were tested

for different seven shear span to depth ratios, a/d , ranging from 0.27 to 2.7, and four different effective spans to depth ratios, l_n/d , ranging from 2.15 to 5.38. Test results indicated that the l_n/d ratio has little influence on the magnitude of the failure load. But for beams with a/d ratio greater than 1.0 the flexural failure mode becomes dominant with increasing l_n/d ratio.

Aziz, 1997 tested twenty-eight reinforced crushed stone concrete deep beams under two equal point loads. The compressive strength of concrete ranged from 26.65 to 45.2 MPa. The tested specimens were divided into three groups with and without web reinforcement. The beams were tested for different shear span to depth ratios ranging from 1.05 to 2.22, and different effective span to effective depth ratios ranging from 2.53 to 7.41. The investigation objectives were to study the effect of shear span to effective depth ratio, effective span to effective depth ratio, compressive strength of concrete and web reinforcement. It was found that the ultimate shear stress decreases with increasing, a/d , ratio and maximum size of aggregate, while it increases with increasing compressive strength of concrete and amount of shear reinforcement.

Ashraf et- al, 1986 investigated the shear capacity of reinforced concrete beams using high strength concrete. A total of 18 beams were tested: 15 without web reinforcement and 3 with web reinforcement in the form of vertical stirrups. In addition to concrete strength, other variables included longitudinal steel ratio and shear span to-depth ratio. Test results are compared with strength predicted using the equations of ACI 318 for beams without web steel. He found that the ACI building code equations may overestimate shear strength by 10 to 30 percent.

2.6 Modes of Failure

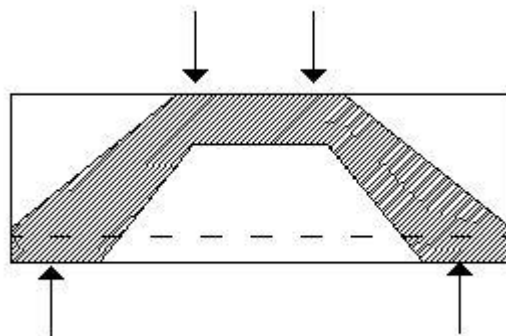
Shear failures of beams are characterized by the occurrence of inclined cracks. In some cases inclined cracking is immediately followed by member failure. In other cases, the inclined cracks stabilize and substantially more shear force may be applied before the member fails.

The type of failure in beams depends essentially on the shear span to depth ratio (a/d) (**AL-Musawi, 1989**).

The modes of failure are as follows:

2-6-1 True Shear Failure:

This failure happens in deep beams, which have a shear span to depth ratio (a/d) less than 2 (**ACI Committee 318, 2005**), (**Canadian Standards Association, 1984**), (**New Zealand Standards Association, 1982**). In this case, shear stress has a predominant effect. After inclined cracking occurs these beams tend to behave like a tied –arch the load is carried by direct compression extending the shaded area of Fig (2-1a) and by the tension in the longitudinal steel. Several modes of failure are possible for the tied- arch system shown in Fig (2-1b); they are anchorage failure, crushing failure at the reaction, and failure of the arch rib (**ACI-ASCE Committee 426, 1973**).



(a)

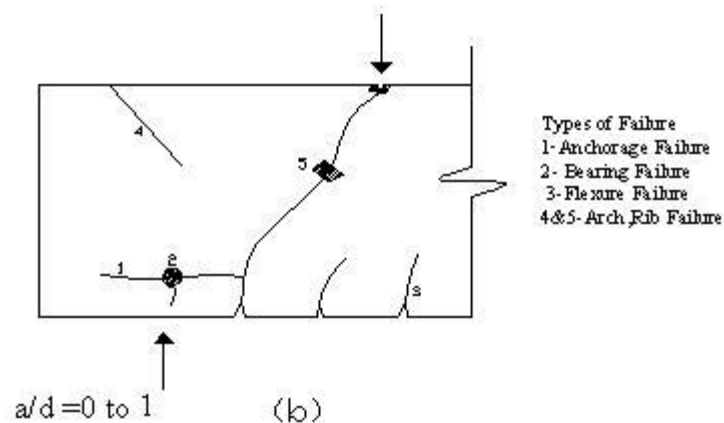


Fig.(2-1) Typical shear failure in deep beams (ACI-ASCE Committee 426, 1973).

2-6-2 Shear- Tension or Shear –Compression Failure:

Just as for deep beams, for beams ($1 < a/d < 2$), the ultimate shear capacity also exceeds the inclined cracking capacity. Failure occurs at some load higher than that caused the inclined crack to form. Then failure eventually results, either by an anchorage failure at the tension reinforcement, called a shear tension failure or in the concrete near compression face, called a shear – compression failure, Fig (2-2) shows this type of failure (**ACI-ASCE Committee 426, 1973**).

2-6-3 Diagonal Tension Failure:

For normal beams, ($2.5 < a/d \leq 6$), vertical cracks are the first to form followed by inclined flexure- shear cracks. The diagonal tension failure happens when one of the cracks continues to propagate through the beams with further load until at some stage it becomes unstable and extends through the beams as shown in Fig (2-3) (**ACI-ASCE Committee 426, 1973**).

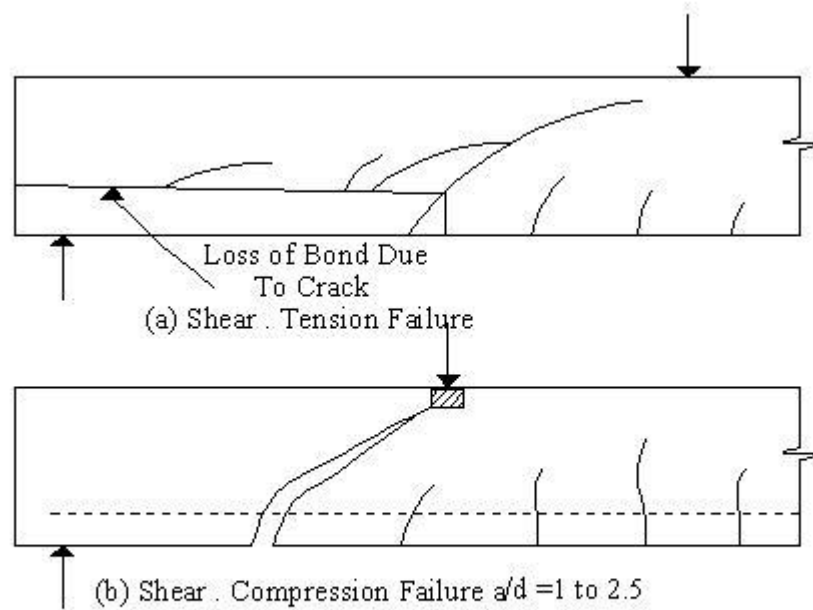


Fig (2-2). Typical shear failure in short beams (ACI-ASCE Committee 426, 1973).

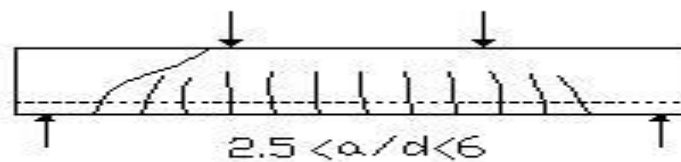


Fig (2-3) Diagonal tension failure(ACI-ASCE Committee 426, 1973).

2-6-4 Flexure Failure:

This failure happens for beams having a shear span to depth ratio (a/d) exceeding a value of six. These beams are called "slender beams".

This type of failure starts with yielding of the tension reinforcement and ends by crushing of the section of maximum bending moment, as shown in Fig (2-4) (ACI-ASCE Committee 426, 1973).

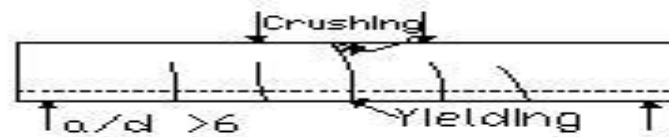


Fig.(2-4) Flexural failure (ACI-ASCE Committee 426, 1973).

2.7 Flexural Strength of Concrete Beams

Four basic assumptions are made when deriving a general theory for the flexural strength of reinforced concrete sections. **(Park and Paulay, 1975), (Nilson and win, 1997), (Hibbeler, 2005)**.

- 1- Plane sections before bending remain plane after bending.
- 2- The stress-strain curve for the steel is known.
- 3- The tensile strength of the concrete may be neglected (uncracked section).
- 4- The stress-strain curve for concrete, defining the magnitude and distribution of compressive stress, is known.

The first assumption, Bernoulli's principle, implies that the longitudinal strain in the concrete and the steel at the various points across a section is proportional to the distance from the neutral axis.

The second assumption means that the stress-strain properties of the steel are well defined. Normally a bilinear stress-strain curve

The third assumption is very nearly exact. Any tensile stress that exists in the concrete just below the neutral axis is small and has a small lever arm.

The fourth assumption is necessary to assess the true behavior of the section. Since the strains in the compressed concrete are proportional to the distance from the neutral axis, the shape of the stress-strain curves, indicate the shape of the compressive stress block at various stages of loading.

The rise in temperature inside beams, as a response to its exposure, depends on number of factors such as the moisture content in concrete and the composition of the cement and aggregate. Also, the development of temperature in a beam depends on the heating conditions and the heat transfer characteristics of the environment (**Ali Nadjai, 2005**).

However, these factors cannot be obtained conveniently and used with confidence in any general design rules.

The residual bending moment of fire-damaged concrete beams are important factors in determining the safety of the structure. In order to realize the effects of fire on bearing capabilities of RC beams, this present simulates the residual bending moment of RC beams heated on bottom surfaces.

Shuaib and lue, 1987 studied the effect of shear on the flexural resistance capacity and failure mode. Fifty-four reinforced concrete beams using high-strength concrete ($f_c > 42\text{MPa}$) were tested under monotonically increasing loads. The beams were singly reinforced without shear (web) reinforcement. Test results indicate that shear reduces the flexural resistance of beams. A "valley of diagonal failure" is clearly observed when the relative flexural strength M_f/M_n is plotted against the shear span-to-depth ratio a/d . test results suggest that the relative flexural strength of high-strength concrete beams is inversely proportional to steel ratio and directly proportional to the concrete to steel ratio and directly proportional to the concrete strength f_c for tensile steel ratios less than 1.88 percent.

Shuaib and Roy, 1991 studied the flexural behavior of high-strength light weight concrete beams, with compressive strength ranging from 35.9 N/mm^2 to 75.9 N/mm^2 with reinforcement ratio $0.18 < \rho / \rho_b < 0.54$. They compared the experimental data with the ACI building code approach and examined the applicability of the ACI approach for flexural design provision. They found that the ACI approach for flexural design has a good

accuracy and is suitable for structural members made of these special materials.

Ghali and Azarnejad, 1999 studied the effect of compressive strength of concrete and reinforcement percentage on predicted deflection for any concrete strength. The experimental data, measured by (**Washa and Fluck, 1952**) with all beams that were simply supported and subjected to uniform loads, being used to verify their work. In both the ACI and CEB – FIP method, the deflections of a member can be determined more accurately from the values of the mean curvature at a number of sections using simple geometric deflection – curvature relations.

Karem, 2006 studied flexural behavior of hybrid reinforced concrete beams. Three groups of simply supported beams are tested with (3000 × 275 × 175) mm dimensions, each of which consists of four beam specimens identical in size but different in constituent proportions. The experimental variables for all groups of normal strength, high strength and hybrid strength have a nominal concrete compressive strength of (21 or 75) N/mm², reinforcement ratio between (0.0095 - 0.0356), bar diameter (16,25)mm and finally number of bars (2,3). This study includes an experimental data for immediate deflection, ductility and strain for the three groups, experimental data for their concrete compressive strength, modulus of rupture and modulus of elasticity are also included. From the results obtained, it can be concluded that the increase in concrete compressive strength affects the immediate deflection values for beams with constant reinforcement ratio. The three methods for the prediction of immediate deflections ACI, CEB – FIP and BS give good predicted values. On the other hand, the improved model for the prediction of immediate deflections using the effective moment of inertia.

2.8 Effect of Fire**2-8-1 Introduction**

The objective of fire safety is to protect life and property. Fires can occur at any time in buildings, and the safety of occupants and maintaining the integrity of the structure are of major importance. Building codes prescribe detailed measures for the fire safety of structural members because when other means for containing fire fail, such as a fire suppression system, structural integrity is the last line of defense. Code-based structural fire safety requirements refer to fire resistance which is defined as the ability of a structural element to maintain its load-bearing functions under standard fire conditions. The fire resistance rating of a structural member is the elapsed time it exhibits resistance with respect to structural integrity, stability, and temperature transmission while exposed to standard fire conditions. The measured fire resistance of a structural member or assembly is dependent on the geometry of elements, materials used in construction, load intensity, fire exposure, and the characteristics of a given furnace.

2-8-2 Effect of Fire on Normal Strength Concrete.

Khan and Royles, 1986 studied the behavior of reinforced concrete beams after subjecting them to elevated temperatures. They investigated the load-deflection relationship, cracks pattern and steel to concrete bond. Prismatic concrete beams (960x 140x 66 or 107mm) were used. 8-mm plain bars and 16mm torbars were used to reinforce them. The specimens were heated to temperature ranges from 20-800°C at a slow rate of heating (2°C/min) for one hour exposure duration. They found that the effect of temperature is insignificant at temperatures ranging from 100 to 200°C, but the strength decreases significantly between 350 to 500°C. Compared with ambient condition, the flexural strength characteristics weakened by 50% of the original strength. They also noticed that the thermal cracks appeared

in a honeycomb fashion all over the surface and they seemed to originate from the top to the bottom edge and terminate near the mid-depth of beam, moreover, they also found that the deflection cracks were more prominent in the shear zone of the smaller section beams, whereas for larger beams, flexural tensile cracking developed in the mid-span region. They also found that the deflection cracking initially followed some of the thermally induced cracks.

Asa'ad, 1987 studied the behavior of structural reinforced concrete specimens subjected to elevated temperature. Four types of reinforced concrete samples were used. Singly and doubly reinforced concrete beams having the dimension of (100x 100x 1100mm) were used. Continuous beams (100x 150x 1300mm) and structural frame with outer dimensions of (900x 750mm) were used. The frame had a cross-section of (100x 150mm) for the beam and (100x100mm) for the column. The specimens were subjected to temperatures of 150, 300, 600,750 and 900°C at the age of 30 or 90 days and tested in flexure after cooling. The researcher found that both flexural strength and stiffness decreased with the temperature increase. He also noticed that the use of top reinforcement had limited this decrease. Moreover, he observed that the increase in temperature led an increase in the magnitude of moment redistribution in continuous beams.

Essa, 1999 studied the effect of burning by fire flame on some mechanical properties of concrete. The investigated properties were concrete compressive strength, density, ultrasonic pulse velocity (U.P.V) and Schmidt rebound hammer number. The specimens were heated to two temperature levels (500 °C) (achieved by subjecting the cubes to direct fire flame from petroleum gas burner) and (800 °C) (achieved by using an oven). The heating durations were 1 and 2 hrs. for the specimens exposed to (500 °C), while it was 1hr. for the specimens exposed to (800 °C). He

found that the reduction in compressive strength is higher in specimens heated to (800 °C) than those heated to (500 °C) (1 and 2hr.). Also, he found that the reduction in density increases with time of exposure. The (U.P.V) decreases with time of exposure and value of temperature.

Umran, 2002 investigated the fire flame exposure effect on some mechanical properties of concrete. The specimens were subjected to fire flame ranging between (25-700°C). Three temperature levels of (400, 500 and 700 °C) were chosen with four different exposure duration of 0.5, 1, 1.5 and two hours without any imposed loads during heating. The specimens were heated and cooled under the same regime and tested after exposure to fire flame at ages (30, 60 and 90 days). Compressive strength of 150 mm cubes and flexural strength of (100x100x400 mm) prisms were measured. Ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (Ed) were tested also. He found that the residual compressive strength ranged between (70-85%) at 400 °C, (59-78%) at 500 °C and (43-62%) at 700 °C. The flexural strength was found to be more sensitive to fire flame exposure than the compressive strength. The residual flexural strength was in the range of (67-78%) at 400 °C, (40-67%) at 500 °C and (20-45%) at 700 °C. He also found that ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (Ed) were more sensitive to fire flame than compressive strength. He also noticed that exposure time after one hour has a significant effect on residual compressive strength of concrete.

2-8-3 Effect of Fire on High Performance Concrete.

Castillo and Durrani, 1990 investigated the effect of transient high temperature on strength and load- deformation behavior of high performance concrete. Compressive strength tests were made on (51×102mm) cylinders of normal and high strength concretes subjected to temperatures up to (800 °C) and loaded to failure. They concluded that

HSC shows a (15 to 20)% loss of compressive strength when exposed to temperatures in the range of (100 to 300 °C) and NSC recovers its strength between (300 and 400 °C) reaching maximum value of (8 to 13)% above the room temperature strength. The HSC progressively loses its compressive strength which drops to about 30 % of its original compressive strength when temperature reached 800°C.

Chan and Lox, 2000 carried out an experimental program to study the mechanical properties and pore structure of high performance concrete (HPC) and normal – strength concrete after exposure to high temperature. After the concrete specimens were subjected to temperature of 800 °C, their residual compressive strength was measured. The porosity and pore size distribution of the concrete were investigated by using mercury intrusion porosimetry. The test results showed that (HPC) had higher residual strength, although the strength of (HPC) degenerated more sharply than the normal – strength concrete after exposure to high temperature. They found that the changes in pore structure could be used to indicate the degradation of mechanical property of (HPC) subjected to high temperature. Another paper published by the same authors (**Lox and Chan, 2000**) investigated the effect of heating and cooling regimes on residual strength and micro structure of normal strength and high performance concrete after they were exposed to high temperatures, 800, 1100°C and two cooling regimes. The test results obtained showed that the residual strength of both (HPC) and (NSC) dropped sharply after exposure to high temperatures. They observed that water cooling which resulted in a significant thermal shock, caused a bit more severed deterioration in strength compared to furnace cooling. They found that the thermal shock was not necessarily the primary cause for spalling in (HPC). They used mercury intrusion porosimetry to measure variation in the pore structure of concrete. They also noticed the significant changes in the cumulative pore volume curves

before and after high temperatures in both (NSC) and (HPC). Moreover ,they found that the cumulative pore volume of (HPC) increased more remarkably than of (NSC).

Habeeb, 2000 studied the effect of high temperatures on some mechanical properties of high strength concrete (HSC) such as compressive strength, flexural strength and dynamic modulus of elasticity (Ed). Three design strengths were investigated 40, 60 and 80 MPa.. The specimens were heated slowly to five temperature levels (100, 300, 500, 600 and 800 °C), and to three exposure periods 1, 2 and 4 hours .

He found that :

- 1- (HSC) is more sensitive to high temperatures than (NSC).The residual compressive strength ranged between (90 – 106%) at 100°C, (72-103%) at 300 °C, (55 – 87%) at 500 °C and (22-66%) between (600-800 °C).
- 2- The flexural strength was found to be more sensitive to high temperature exposure than compressive strength, the residual flexural strength was in the range of (92 – 98%), (52-98%) and (29-47%) at 100° C, 300° C and 500 °C respectively and (2-30%) at 600 –800 °C .

2-8-4 Effect of Fire on Steel Reinforcement.

Holmes and Anchor, 1982 studied the effect of elevated temperatures on the strength and stiffness properties of four reinforcing steels of varying size. The test program was designed to provide data on three major parameters (yield stress, ultimate strength, and elastic modulus).They found that the normalized results for the yield stress, ultimate strength and elastic modulus for all sizes are as follows: -

- 1 – There was no significant change in the normalized values below 300°C.

2 – There was 50 % reduction in both the yield stress and ultimate strength when obtained between 520°C and 580°C and between 540°C and 700°C for the elastic modulus.

Edward, and Gamble, 1986 investigated behavior of grade 60 reinforcing bars of diameter (12.7 mm) after exposure to fire. The bars were heated to temperatures of (500-800 °C), the furnace was held at peak temperature for about one hour and then slowly cooled simulating one possible condition in a severe fire. The bars were tested in tension after they had been cooled. The yield stresses were reduced to a minimum of 73% of the original strength and the ultimate strength was reduced to a minimum of 83% of the as rolled strength. The heating and cooling of the steel did not change the nature of the stress-strain curves. The modulus of elasticity remains constant at room and elevated temperatures.

2-8-5 Effect of Fire on Flexural Behavior and Shear Strength

Muhammed, 2007 studied residual strengths of reinforced normal strength, high strength and light weight concrete panels exposed to high temperature. Twenty two reduced scale reinforced concrete slab specimens divided into three main groups each of which is divided into two subgroups representing the NSC, HSC and LWC types of corresponding cube compressive strengths of 50, 94 and 37 N/mm² respectively, are prepared in the laboratory, and having two different steel ratios 0.002 and 0.005 for flexure and punching testes respectively. These specimens are subjected to different temperature – time exposures using an electric furnace.

Residual flexural and punching shear strengths, load – deflection curves, crack patterns, failure characteristics and details, and effects of temperature level exposure are all recorded, investigated and discussed. Effects of concrete type and structural failure type when slabs are exposed to different temperature levels are also discussed. Results indicate reduction of

strengths with exposure to high temperature. Residual punching strength percentages at maximum temperature exposure (700°C) are 33.3%, 53.8% and 69.2% for NSC, HSC and LWC slabs respectively. The corresponding residual flexural strength percentages are 44.5%, 38.7% and 45.5%.

Hsu and Lin, 2006 investigated residual bearing capabilities of five-exposed reinforced concrete beams. The analysis method includes combining thermal and structural analyses for assessing the residual bearing capabilities, flexural and shear capacities of reinforced concrete beams after fire exposure. The thermal analysis uses the finite difference method to model the temperature distribution of a reinforced concrete beam maintained at high temperature. The structural analysis, using the lumped method, is utilized to calculate the residual bearing capabilities, flexure and shear capacities of reinforced concrete beams after fire exposure. This novel scheme for predicting residual bearing capabilities of fire-exposed reinforced concrete beams is very promising in that it eliminates the extensive testing otherwise required when determining fire ratings for structural assemblies.

Moetaz et al, 1996 fabricated four reinforced beams 20cm deep, 12cm wide and 180cm long. The beams were reinforced with 2 ϕ 10mm ($f_y=360\text{Mpa}$) as the main reinforcement, 2 ϕ 10 mm ($f_y=260\text{Mpa}$) as the secondary reinforcement, and ϕ 8 mm stirrups with 8cm spacing. The beams were installed in the fire test chamber 40 days after casting. During the fire test, the beams were not loaded and exposed to fire at 650°C Figure (2-5). The chamber was controlled so that the same average temperature-time curve was followed for all beams. Beams were exposed to fire for 30,60 and 120 minutes and the control beam was not exposed to fire. After cooling to room temperature, the beams were tested to failure by applying two increasing transverse loads Figure (2-6).

Table (2-1) presents the test and modeling results for beams (B,B₁,B₂ and B₃) that were exposed to fire for 0,30,60 and 120 minutes, respectively. Figure (2-7) shows the decreases in residual bending moment for experimental and modeling with duration of fire. Experimental and modeling results follow. Figure (2-8) diagrammatically compares the residual bending moment obtained between experimental and modeling results also. Agreement between experimental and modeling results in Figure (2-8) shows that using the calculation procedure in this study is suitable.

**Table(2-1)Ultimate bending moment from experiment and modeling
(Moetaz et al, 1996)**

Properties	Beams			
	B	B1	B2	B3
Fire exposure time (min)	0	30	60	120
Ultimate loads from test (t)	5.95	5.25	4.80	3.65
Ultimate bending moment from experiment (kN-m)	16.78	14.81	13.54	10.50
Ultimate bending moment from modeling (kN-m)*	15.25	13.76	13.09	11.82

*Calculation from test ultimate loads as loading in figure(2-8)

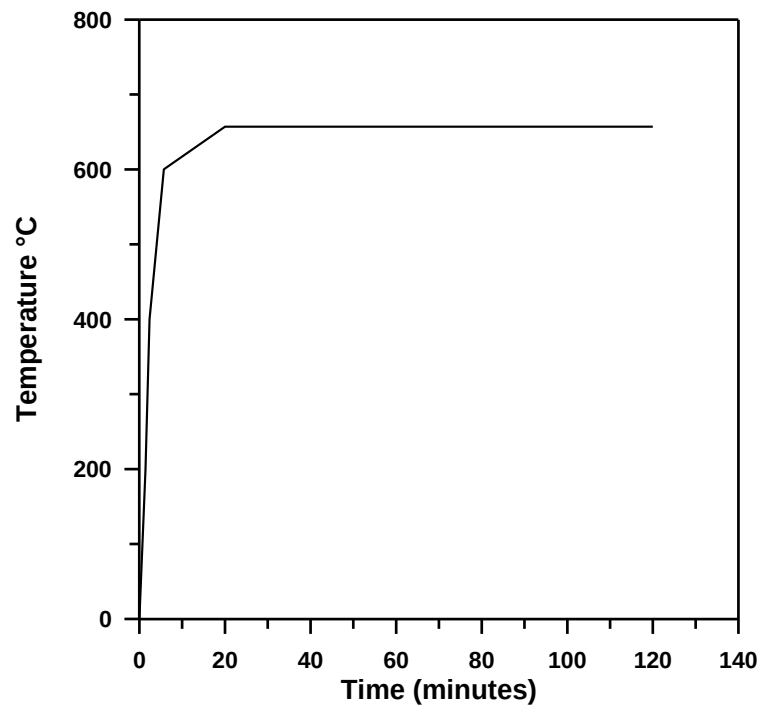


Figure (2-5) Time-temperature curve (Moetaz et al, 1996)

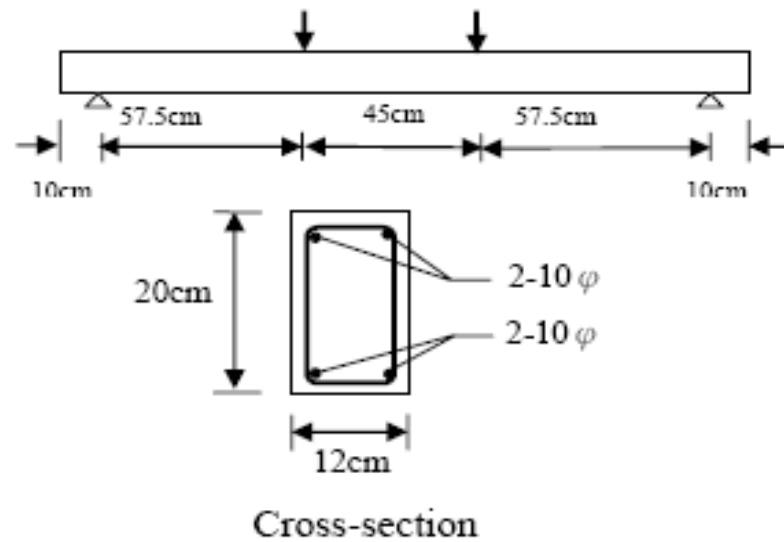


Figure (2-6) Fire flexural test conditions (Moetaz et al, 1996)

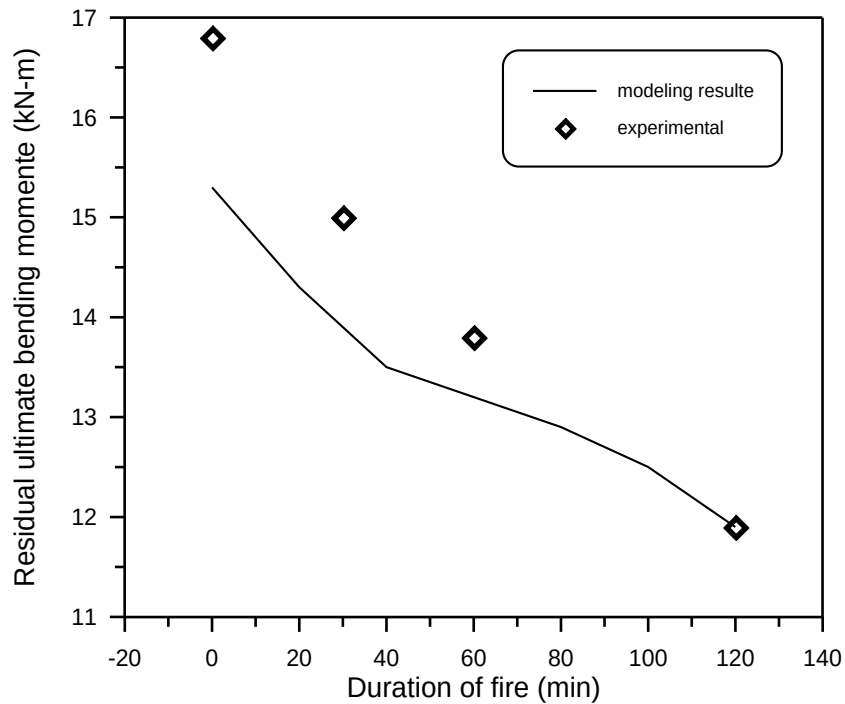


Figure (2-7) The decreases in residual bending moments with duration of fire (**Moetaz et al, 1996**)

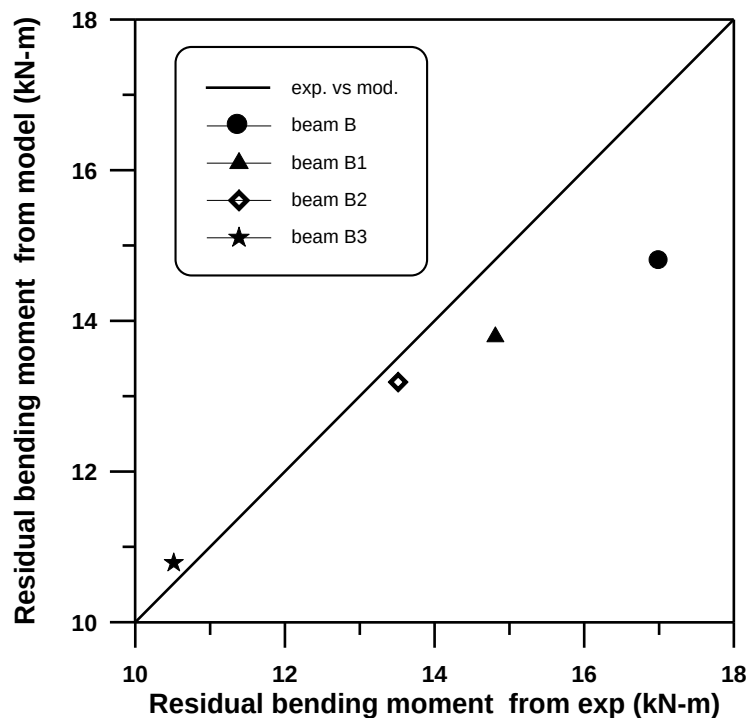


Figure (2-8) Comparison of residual bending moments obtained from experimental and modeling results (**Moetaz et al, 1996**)

Lin et al, 1999 fabricated 35 full-scale RC beams with various dimensions and parameters. All beams were cured under the same

conditions in laboratory. After curing for 28 days, the beams were placed in a gas furnace. The beams were heated according to the ASTM E- 119 standard temperature-rise curve (ASTM E05.11, 1982) through three surfaces; the beam tops were insulated. After natural cooling the beams were loaded using the two-point load method and tested to failure. The calculated and experimental data follow similar trends Figure (2-9). The calculated values are typically less than experimental values, indicating that modeling results are conservative. The results showed residual shear strength with duration of fire. The modeling results revealed that the shear reinforcement provided the main shear strength in common states but decreased quickly when yielding strength has influenced by high temperature. The covered concrete could delay the influence of high temperature on shear reinforcement. Increasing the thickness of concrete cover is helpful to protect the damage of fire on shear strength.

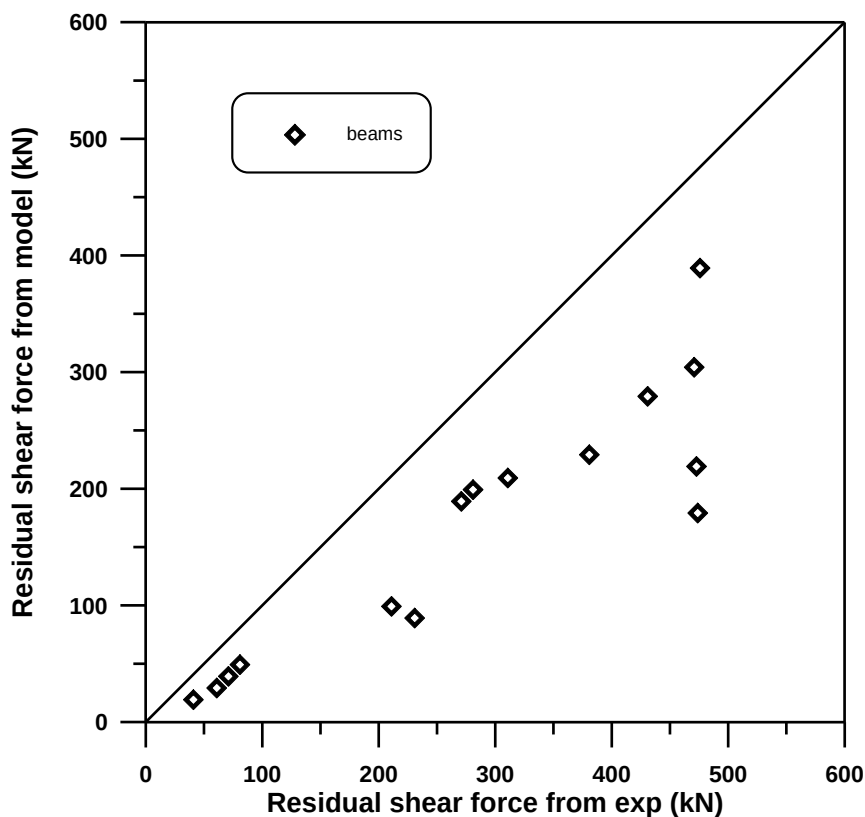


Figure (2-9) Comparison of experimental residual shear forces and modeling results (Lin et al, 1999)

EXPERIMENTAL WORK AND TEST PROGRAM

3.1 Introduction

In this chapter, the details of the experimental program of the present work are presented. It includes details of materials used, specimen preparation and tests procedure. Two mixes with design compressive strength of 30 and 78 MPa were chosen. Experimental tests were:

1. Compressive strength;
2. Splitting tensile strength;
3. Modulus of rupture;
4. Modulus of elasticity; and
5. Load-deflection relationship.

The variables were :

1. Shear span to depth ratio;
2. Longitudinal reinforcement ratio;
3. Fire flam temperature;
4. Period of exposure; and
5. Depth of beam.

Forty beam specimens were cast, moist cured for 7 days, air-dried in the laboratory. They were tested for each mix at age of 60 days before and

after exposure to fire flame at two temperature levels, 400 and 700°C and for two exposure periods of 1 and 1.5 hours at the bottom surface of beam.

3-2 Materials

3-2-1 Cement

Al_Sharqiya Ordinary Portland Cement (OPC) (ASTM TYPE1) manufactured in Kingdom of Saudi Arabia (KSA) was used throughout this research. Tables (3-1) and (3-3), show the chemical analysis and the physical properties respectively for the cement used in this study. Testing of cement was conducted in the laboratories of Consultant Engineering Bureau in Babylon University. The compliance of the cement is examined according to the Iraqi specification (**IQS No.5:1984**). The chemical analysis of the cement is listed in Table (3-1). The compounds of cement calculated according to Bogue equations are listed in Table (3-2).

Table (3-1) Chemical analysis of the cement

Oxide	%	IQS (No.5:1984)
Calcium oxide CaO	64.2	
Silicon dioxide SiO ₂	21.3	
Aluminum oxide Al ₂ O ₃	4.0	
Ferric oxide Fe ₂ O ₃	4.80	
Magnesium oxide MgO	1.56	≤ 5.0
Sulphur trioxide SO ₃	2.27	≤ 2.8
Loss on Ignition (L.O.I)	1.26	≤ 4.0
Lime Saturation Factor (L.S.F.)	0.91	0.66-1.02
Insoluble Residue (I.R)	0.83	≤ 1.5

Table (3-2) Bogue potential compound

Tricalcium silicate C3S %	59.20
Dicalcium silicate C2S %	16.49
Tricalcium aluminates C3A %	2.49
Teteracalcium alumina ferrite C4AF %	14.59

Table (3-3) Physical properties of the cement

Physical properties	Test results	IQS (No.5:1984)
Specific gravity	3.15	
Fineness: specific surface, Blaine . (m ² /kg)	350	>=250
Setting time(Initial)(min)	120	>=45 min
Setting time(Final)(min)	230	<=600 min
Compressive strength, MPa , fc at 3days	22.8	>=15
Compressive strength, MPa , fc at 7days	31.6	>=23

3-2-2 Fine Aggregate

The grading and particle shape of fine aggregate are significant factors in production of HPC. Fine aggregate with rounded particle shape and smooth texture are preferable in HSC (**ACI Committee 363, 1997**) and (**Carrasquillo et- al, 1987**), sand with fineness modulus of about (2.5-3) is preferable to obtain a workable concrete mix with limited amount of water, and for enough fine material (cement) in the mix to obtain the required consistency.

Natural Sand from Al-Najaf region was used. The results of physical and chemical properties of the sand are listed in Table (3-4). Its grading conformed to the Iraqi specification (**IQS, No.45:1984**).

Table (3-4): Properties of fine aggregate.

Sieve size (mm)	Passing %	IQS (No.45:1984) Zone (3) Limits
10.00	100	100
5.00	96	90-100
2.36	86	85-100
1.18	76	75-100
0.60	60	60-79
0.30	19	12-40
0.15	6	0-10
Finer than 0.15	0	0-5
Property	Results %	IQS (No.45:1984) Limits
SO ₃ content	0.25	≤0.5
Fineness modulus	2.82	
Specific gravity	2.60	
Absorption %	1.4	

3-2-3 Coarse Aggregate (gravel)

Crushed gravel with maximum size of 19_{mm} and 14_{mm} was used as coarse aggregate in normal and high performance concrete. The ideal coarse aggregate used should be clean, generally cubical, or angular, 100% crushed aggregate with a minimum of flat and elongated particles (**ACI Committee 363, 1997**). Many researchers (**Aitcin, 1980**) and (**AL-Hassani, 2002**) had shown that the smaller size aggregate produces higher

strength values, therefore, the maximum coarse aggregate size is chosen to be (14_{mm}) in high performance concrete. It was obtained from Al-Nibaii quarries. Table (3-5) and (3-6) shows the grading of coarse aggregate for NSC and HPC respectively which conforms to the Iraqi specification (**IQS, No.45:1984**).

Table (3-5) Grading and physical properties of coarse aggregate used in NSC

Sieve size, mm	Cumulative passing	IQS (No.45:1984) Limits
37.5	100	100
19	100	95-100
9.5	58	30-60
4.75	6.45	0-10
Sulfate Content: 0.08% max 0.1%		
Specific gravity: 2.62		
Absorption : 0.5%		

Table (3-6) Grading and physical properties of coarse aggregate used in HPC

Sieve size, mm	Cumulative passing	IQS (No.45:1984) Limits
19	100	100
14	100	90-100
10	71	50-85
5	6	0-10
Sulfate Content: 0.051% max 0.1%		
Specific gravity: 2.65		
Absorption : 0.47%		

3-2-4 Water

Tap water was used throughout this work for both making and curing the specimens.

3-2-5 Admixtures

3-2-5-1 Superplasticizer : High Range Water Reducing Admixture (HRWRA)

For HPC production, the water content of the mix is needed to be reduced and at the same time maintaining suitable workability, which can be achieved by using superplasticizer (**ACI Committee 363, 1997**) and (**Neville, 2000**) and to compensate for the associated reduction in water content and workability of the concrete mix.

A superplasticizer (SP) of sulphonated melamine and naphthalene formaldehyde condensates, known as (GLENIUM-51) is used in this investigation as high range water reducing superplasticizer. The technical description is shown in Table (3-7).

**Table (3-7) Technical description of the superplasticizer
Glenium - 51**

Colour	Light brown
Density	1.1 gm/cm ³
Chloride Content	Free from chlorides
pH value	6.6
Subsidiary effect	Hardening accelerator
Form	Viscous Liquid
Storage	Should be stored in original containers and at above 5 °C
Dosage	The normal dosage is between (0.5-0.8) liters per 100kg of cement. Dosage outside this range are permissible subject to trial mixes

The above description is according to information supplied by the manufacturing company. Laboratory test results indicate that the admixture is high range water reducing admixture, conforming to the

(ASTMC494, 1999) **Type F** specification. The dosage used in this investigation was 1.15% by weight of cement

3-2-5-2 Silica Fume

Silica fume, also known as microsilica, is a by product of the reduction of high-purity quartz with coal in electric furnaces in the production of silicon and ferrosilicon alloys. Silica fume consists of very fine vitreous particles with a surface area in the order of (20,000 m²/kg) when measured by nitrogen absorption techniques. Silica fume is used in concrete to improve its properties. It has been found that silica fume improves:

- 1- Compressive strength of concrete;
- 2- Bond strength between concrete ingredients and between concrete and steel reinforcement;
- 3- Abrasion resistance; and
- 4- Reduces permeability; and therefore helps in protecting reinforcing steel from corrosion.

Silica fume is available as a densified powder or in a water-slurry form. It is generally used at 5 to 12% by mass of cementitious materials as a partial replacement for concrete structures that need high strength or significantly reduced permeability to water (**National Ready Mixed Concrete Association, 2000**)

The benefits result from adding silica fume are related to changes in the microstructure of the concrete. These changes result from two different but equally important processes:-

- I. Physical contribution; Adding silica fume brings millions and millions of very small particles to a concrete mixture, it fills the spaces between cement grains. This phenomenon is frequently referred to as particle packing or micro-filling. Even if silica fume

did not react chemically, the micro-filler effect would bring about significant improvements in the nature of the concrete.

- II.** Chemical contribution; Because of its very high amorphous silicon dioxide content, silica fume is a very reactive pozzolanic material in concrete. As the Portland cement in concrete begins to react chemically, it releases calcium hydroxide. The silica fume reacts with this calcium hydroxide to form additional binder material (calcium silicate hydrate (C-S-H)) which is very similar to the calcium silicate hydrate formed from the Portland cement (**Neville, 1995**)

The chemical composition of silica fume used in this investigation is shown in Table (3-8). The silica fume used in this work conforms to the chemical and physical requirements of (**ASTM C1240-2003**) as shown in Tables (3-9) and (3-10).

Table (3-8): Chemical analysis of silica fume*

Oxide composition	Oxide content%
SiO ₂	95.95
Al ₂ O ₃	0.02
Fe ₂ O ₃	0.01
K ₂ O	0.07
CaO	1.21
MgO	0.01
SO ₃	0.22
L.O.I.	2.5

*The chemical tests carried out at the National Center of Geological Survey and Mining.

**Table (3-9): Chemical requirements of silica fume
according to ASTM C 1240-03.**

Oxide composition	Test result	Limit of ASTM C 1240-2003 specification requirement
SiO ₂ , percent	95.95	Min, 85
Moisture content, percent	0.8	Max, 3
Loss on ignition, percent	2.5	Max, 6

**Table (3-10): Physical requirements of silica fume
according to ASTM C 1240-03.**

Physical properties	Test result	Limit of ASTM C 1240-2003 specification requirement
Percent retained on 45- μ m (No.325) sieve.	7	Max,10
Accelerated pozzolanic Strength Activity Index with Portland cement at 7 days.	196	Min,105
Specific surface, m ² /g	20	Min,15

3-3 Concrete Mixes

Two mixes of concrete were investigated in this work, namely 30 and, 78 MPa at age (28) days.

The normal concrete was designed according to British mix design method **BS 5328 part2: 1991** specifications.

The high performance concrete was made under controlled laboratory conditions with a constant water cementitious materials ratio (0.3) and designed to give a characteristic compressive strength of (75-80) MPa.

The cement content was 525 kg/m³ and w/c ratio to maintain a slump of (180-200)mm was (0.3). After many trails, amix proportion of (1:1.22:2) by weight was adopted. Superplasticizer (SP) was added in 1.15%.by weight of cement.

Silica fume was added to the concrete mix to increase strength, and to improve concrete properties both in the fresh and hardened state. In this investigation a value of 5% silica fume content by weight of cement was add to the amount of Portland cement .The details of the two groups of mix proportion are shown in Table (3-11).

Table(3-11) Mix proportion of concrete

Group	Super-Plasticizer %	Cement materials		Water kg/m ³	Fine Agg kg/m ³	Coarse Agg kg/m ³	W/C or W/C+ SF ratio	Target Strength MPa	Slump mm
		Cement kg/m ³	SF kg/m ³						
NSC	-	350	-	189	834	1181	0.54	30	80
HPC	%1.15	525	26	165	640	1050	0.3	78	200

3.3.1 Mixing of Concrete

Mixing method is important to obtain the required workability and homogeneity of the concrete mix. Concrete is mixed in a drum rotating laboratory mixer with a capacity of 0.08m³.

The interior surface of the mixer was cleaned and moistened before placing the materials. Superplasticizer was added with the mixing water, and the silica fume also was added with the cement content, after mixing with cement to ensure uniform dispersion of the powder through the cement particles. Initially coarse aggregate and fine aggregate are poured into the mixer, the dry materials were well mixed for about (1) minute to attain uniform mix. Then 50% of the mixing water with superplasticizer to wet the materials. After that, the cement with silica fume are added at this stage followed by the mixing water with superplasticizer, then the remaining 25% of the mixing water is added gradually to the mix, the total mixing time is in the range of 3.5 minutes.

3.3.2 Compaction

After mixing, the concrete was poured into the moulds with two layers, each layer was compacted by using a plunger mechanical vibrator (3000 vibration per minute) having a metal rod with diameter of 50mm for 5 seconds for each insertion until no air bubbles emerge from the surface of the concrete. The moulds were leveled by hand trawling and covered with polyethylene sheet in the laboratory for about 24hours.

3.3.3 Concrete Casting and Curing

The concrete casting and curing procedure is described below:

1- Beam specimens and control specimens were treated with oil before putting the reinforcement or casting control specimens to prevent adhesion with concrete after hardening.

2- Before mixing, all the quantities were weighted and backed in a clean plastic container.

3- Concrete poured in the moulds in two layers, and each layer was compacted using a plunger mechanical vibrator. The upper surface of concrete was smoothly finished after casting was completed using hand trowel.

4- After casting of concrete, a polyethylene sheet was used to cover the beams and control specimens in order to avoid plastic shrinkage cracks within 24 hours and then burlaps sacks were placed over the beams and wetted down. The burlap sacks were monitored and kept wet until the fully seven days had past, the beams and control specimens subjected to drying in uncontrolled laboratory conditions until age 60 days.

3.4 Testing of Concrete Mechanical Properties

3.4.1 Compressive Strength Test :

The compressive strength test was performed according to (**BS. 1881, part 116, 1989**).100 mm cubes were tested using standard testing machine with a capacity of 3000 kN. The load was applied gradually and increased continuously at a constant rate. Each cube was weighted before and after heating to determine its density. Each compressive strength value was the average of the strength of three cubes.

3.4.2 Splitting Tensile Strength Test

The splitting tensile strength was conducted on cylinders of 100mm diameter×200mm height. The average of three test specimens was taken.

The test was carried out in accordance with (**ASTMC496-86 ,1989**) using the compression testing machine at stress rate of about 1.5 N/mm² per minute. Two bearing strips of nominal 3mm thick plywood, approximately 25mm wide and length equal to specimen length, were

placed below and above the specimen which was placed between the bearing blocks of the testing machine. The specimen was loaded up to failure.

The splitting tensile strength was calculated as follows:

$$F_{sp} = \frac{2P}{\pi DL} \quad \dots\dots\dots(3-1)$$

F_{sp} = splitting tensile strength, (MPa)

P = maximum applied load, (N)

D = diameter of concrete cylinder, (mm)

L = length of concrete cylinder, (mm)

3.4.3 Flexural Strength Test

Concrete prisms of dimensions (100*100*400)mm were casted according to **BS 5328:4:1990** procedure. The prisms were cast, demolded and cured in a similar manner as the compressive strength cubes. Modulus of rupture test according to BS 1881 was performed using two point load.

The modulus of rupture of specimens is calculated , as follows (**ASTM C78-84, 1989**)

1- If fracture initiates in the tension surface within the middle third of the span length, then:

$$f_r = \frac{PL}{bd^2} \quad \dots\dots\dots(3-2)$$

f_r = The modulus of rupture, (N/mm²).

P = Maximum applied load, (N)

b = Average width of specimen, (mm).

d = Average depth of specimen, (mm).

2- If fracture occurs in the tension surface outside the middle third of the span length by no more than 5% of the span length, then:

$$f_r = \frac{3Pa}{bd^2} \quad \dots\dots\dots(3-3)$$

a = Average distance between line of fracture and the nearest support measured on the tension surface of the prism,(mm).

3- If fracture occurs in the tension surface outside the middle third of the span length by more than 5% of the span length, the results of the test are discarded.

3.4.4 Static Modulus of Elasticity

The static modulus of elasticity was determined according to (**ASTM C-469, 1984**) specifications. A total number of 42 cylinders (150* 300)mm were tested. All specimens were cast into three layers, then demolded and cured as for the compressive strength cubes. The top face of cylinder was well-finished and smoothed using electric grinding machine to avoid any loss of strength. The specimen was placed in the compression machine with the strain-measuring equipment attached. The compress meter used has a gage length of 200mm and with an accuracy of 0.002mm. The specimen was loaded two times, during the first load, which was primarily for set and check dial gage, the second load was taken. The load was applied at a constant rate to 40% of the ultimate load, the load and deformation were recorded to determine the modulus of elasticity, the results are plotted on a graph, in which the modulus of elasticity that determined according to the following equation (3-4).

$$E_c = \frac{S_2 - S_1}{\varepsilon_2 - \varepsilon_1} \dots\dots\dots(3-4)$$

E_c =modulus of elasticity (GPa).

S_2 =stress corresponding to 40 % of ultimate load (MPa).

S_1 =stress corresponding to the longitudinal strain of 50×10^{-6} (MPa).

ε_2 =longitudinal strain produced by S_2 .

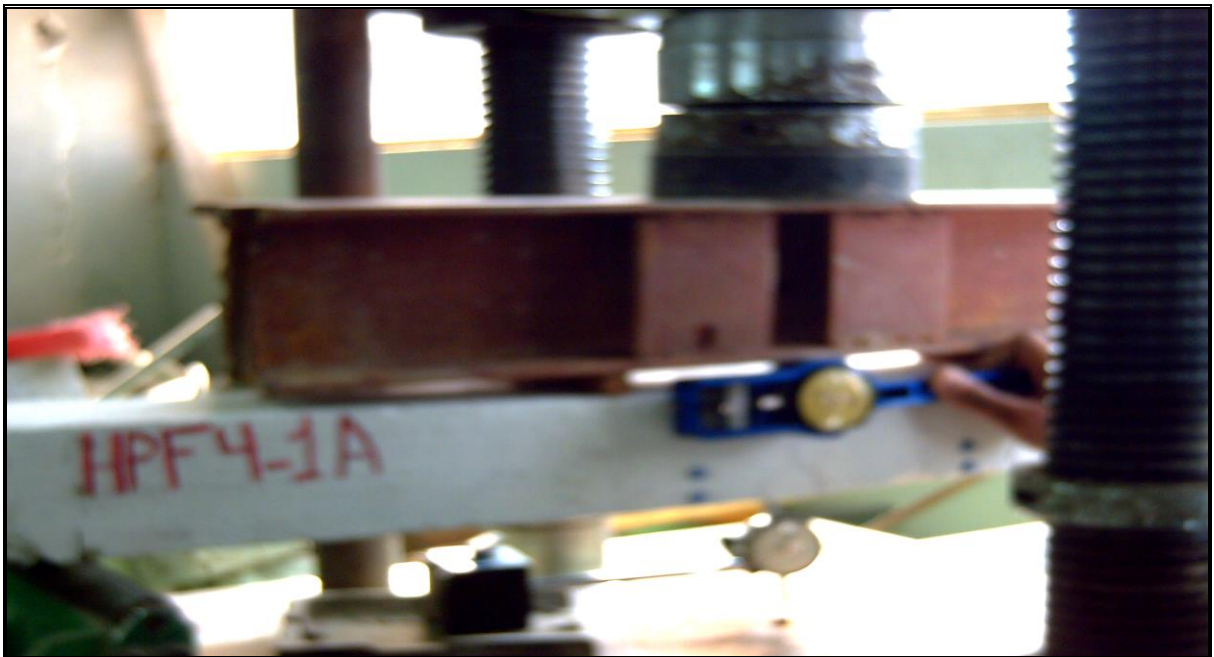
$\varepsilon_1 = 50 \times 10^{-6}$

The test was conducted in laboratory of Babylon Technical Institute

3.5 Beams Test Setup

After being burnt, the beam specimens were transported to the Babylon Technical Institute for testing them in flexure and shear.

The beam specimens were tested in two point load. Beam specimens were tested as simply supported beams with 900mm span for flexural failure and 450mm for shear failure in 1000kN capacity hydraulic machine. The load were distributed through steel bearing plate 300mm and 250mm in flexural beam and shear respectively to cover the entire beam width. To observe the crack development, beam specimens were painted white with emulsion paint before testing, cracks were traced by pencil, Plates (3-1) and (3-2) show the test setup in flexure and shear.



Plate(3-1)Test setup for flexural strength



Plate(3-2) Test setup for shear strength

3.5.1 Instrumentation

During testing, the main characteristics of the structural behavior of the specimens were detected at every stage of loading. At each test, the first cracking load and ultimate load were recorded. The devices and the measurements were recorded for each instrument are presented in Table (3-12). In addition; the purpose of each measuring device used in the test setup is expressed.

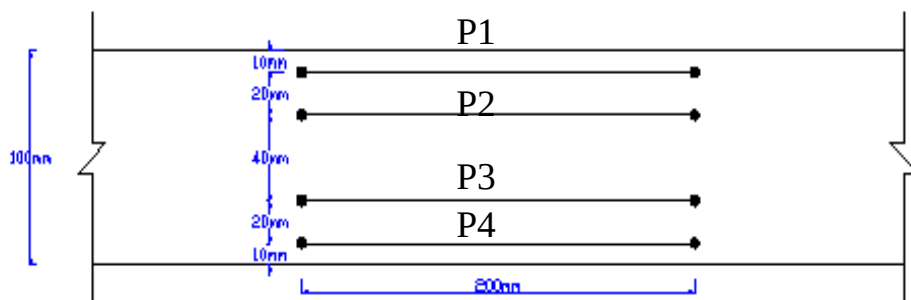
Table (3-12) Instrumentation details

Device	Measurement	Purpose
Demec points (p1-p4) and extensometer	Strain	To monitor the strain in concrete surface
Dial gauge	Deflection	To record the beam deflection

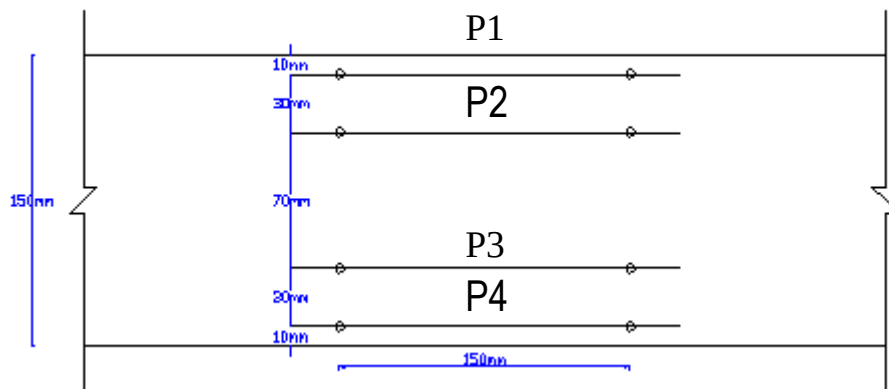
The measurements, which were recorded during the tests, as following:

- strains in concrete.
- Deflection in mid span of beam.

Deflection of the beam specimen was measured at mid-span using a dial gage with travel distance of 25mm and accuracy of 0.025mm. An ELE mechanical strain gage having an accuracy of 0.002mm was used to measure the deflection. Four pairs of demec discs were used for each tested beam specimen. Demec discs were calibrated using an accompanying special ruler. Arrangement and distribution of these demec discs are shown in Figure (3-1)



(A) Side view of tested normal beam in flexure



(B) side view of tested deep beam in flexure

Figure (3-1) :Arrangement of fixed demec steel discs on sides of tested beams

3.5.2 Reinforced Concrete Beams Testing Procedure

The beams were tested in flexure by two point load. The load was applied using I-section steel beam supported by two steel bars that divide the load to two equal point loads. Before loading, initial reading of deflection dial gage and mechanical strain gage (extensometer) were obtained. Deflection was measured at mid span by using one dial gauge as shown in Figure (3-2), (3-3). Demec strains gauges were usually distributed along the depth of one side at the middle of the beam as shown in Fig (3-1) at each load increment, observations of crack development on the concrete beams were marked with a heavy felt pen and the mechanical demec strains extensometer readings were taken. the load was continued until ultimate load (defined as the highest capacity beyond which loading dropped).

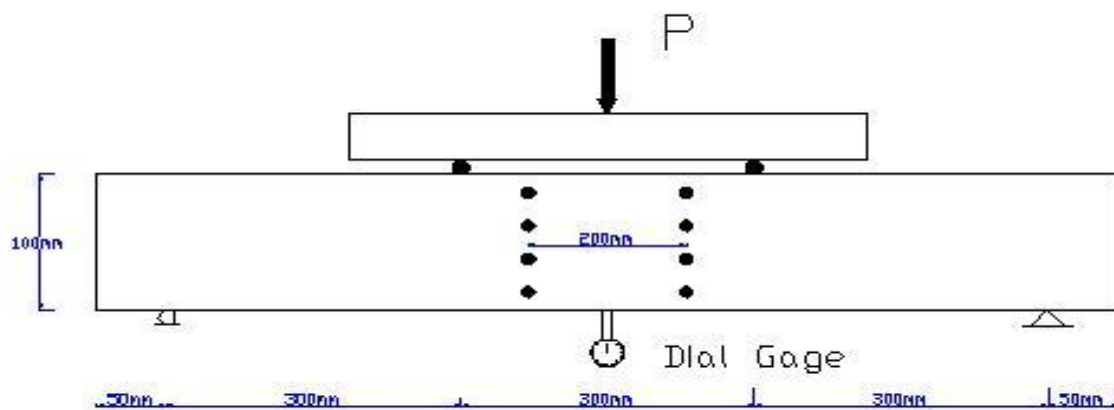


Figure (3-2) Loading and test setup of normal beam in flexure

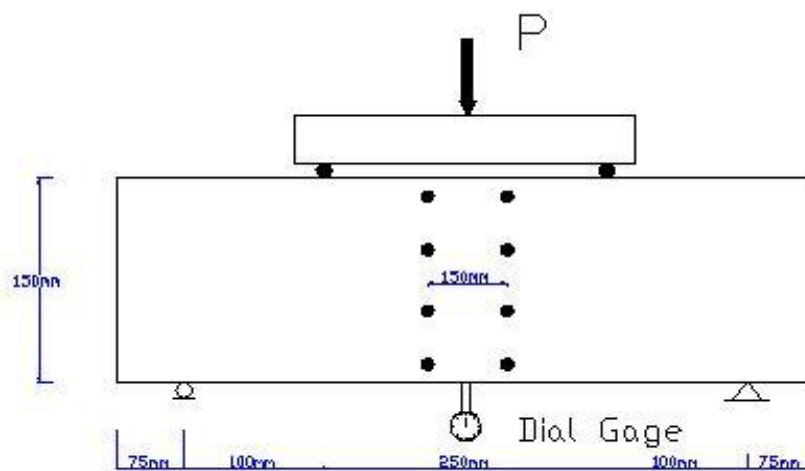


Figure (3-3) Loading and test setup of deep beam in flexure

3.6 Heating and Cooling

The reinforced concrete beams and control specimens were burnt with direct fire flame from a net of methane burners inside a brick stove with dimensions of (2000*1600*800)mm (length * width * height respectively) as shown in Plate (3-3). The bare flame was intended to simulate the heating condition in an actual fire. The intensity of the flame was adjusted to raise the specimens to different temperatures. When the target temperature was reached, the temperatures were continuously recorded by two digital thermometers, one of them was positioned in flame contact with the bottom, while the other was at the top of the specimen as shown in plate (3-4). For the cooling regime , the fire flame was switched off at the end of the exposure time .

The specimens were removed immediately after being extinguished and picked by steel frame .

Apart of them allowed to be cool outside the stove for 2 hours.

Usually a day before testing, the beam was cleaned, painted and marked at support points and strain measurement points to clarify the propagation of cracks and the crack viewing would become easier.

The other part of specimens were sprinkled with water immediately for 2 hours.



Plate (3-3) Brick stove of methane burners network used for burning specimens



Plate (3-4) Measurement of the temperature in the stove

3-7 Testing Reinforced Concrete**3-7-1 Beam Specimens Preparation**

Forty simply supported reinforced concrete beam specimens were cast and cured under laboratory conditions at the construction materials laboratory of the civil engineering department of Babylon University. The reinforcing bars were cut to the desired length, and 90-degree hooks were formed at the ends of each bar. Stirrups made from 6mm diameter plain bars were provided to prevent the shear failure. Eight beams were retained as reference beams for 60 days , thirty-two beams were exposed to fire flame with different periods of exposure and different temperature levels of exposure. The control specimens including cubes, cylinders and prisms were cast from each batch of concrete in addition to the beam specimens as mentioned before.

3-7-2 Beam Specimens Details

The beams were simply supported. The beams were 1000mm length, 100mm height and 100mm width in the case of flexural failure as shown in Figure (3-4). Deep beams were 600mm length, 150mm height and 100mm width to obtain shear failure, as shown in Figure (3-5).

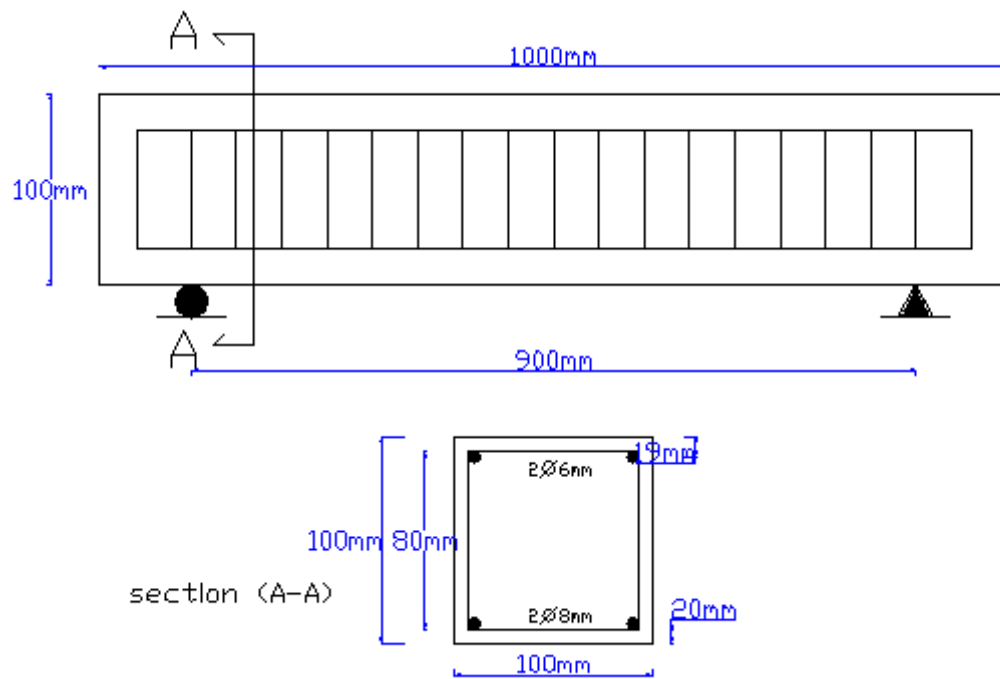


Figure (3-4) Reinforcement details of normal reinforced concrete beam

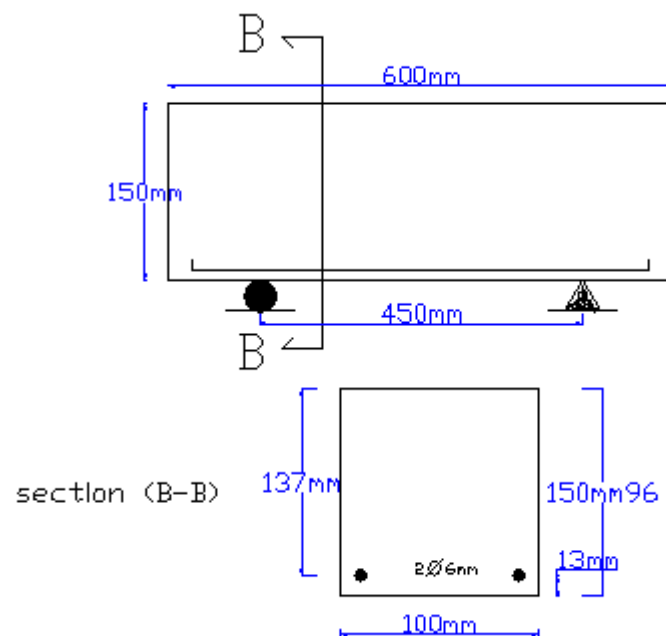


Figure (3-5) Reinforcement details of deep reinforced concrete beam.

TEST RESULTS AND DISCUSSION

4.1 Introduction

The main objective of the current research is to investigate the effect of fire flame exposure on flexural behavior and shear strength of reinforced NSC and HPC beams. In this chapter, the test results from the experimental program will be presented and discussed.

In the research program, forty beam specimens were tested to study the flexural and shear strength after fire flame exposure. Eight beam specimens were cast as reference beams for comparing the performance of HPC beams with NSC beams.

The specimens were tested at age 60 days, two temperature levels of 400 and 700 °C were chosen with two different exposure durations of (1) and (1.5) hours.

Test results were analyzed based on cracking behavior, vertical mid span deflection, strain distribution across the depth of the reinforced concrete beam at mid-span section, and failure mode.

4.2 Cracking Behavior

Crack formation and propagation were detected throughout testing to assess the behavior of HPC specimens in comparison with the behavior of NSC and also behavior of the specimens exposed to fire flame with control beams.

However, this study focuses on the behavior of HPC in flexure and shear strength after exposure to fire flame. Only the first crack load and the cracking pattern of the specimens are presented in the following subsections.

4.3 Analysis of Beam Specimens

In order to determine the effect of fire flame exposure on flexural behavior and shear strength of reinforced HPC beams, it was decided to test singly reinforced, simply supported beams with no web reinforcement. Keeping in view the size and capacity of the loading frame in the Babylon Technical Institute. The specimens were similar in dimensions and reinforcement for NSC and HPC to present flexural failure, and also similar between NSC and HPC specimens to present shear failure.

4.3.1 Flexural Analysis of NSC and HPC Beam Specimens

The ultimate design bending moment of the test specimens were calculated according to the ultimate design method(**ACI-318/2005**) code. The beams were loaded as simply supported with symmetrical two point loads. The design of beams considers that the beam will fail in flexure (yielding of steel reinforcement) before shear failure, therefore vertical stirrups were included.

Ultimate moment capacity- M_u -can be found according to the following ACI-318/2005 code formula .

$$M_n = A_s f_y d \left(1 - 0.59 \rho \frac{f_y}{f_c} \right) \dots \dots \dots (4 - 1)$$

where $f_c = 0.85 f_{cu}$ for NSC.

$$f_c = 0.9 f_{cu} \quad \text{for HPC.}$$

Calculation of bending moment capacity for NSC and HPC specimens before exposure to fire flame temperature is given in Appendix A.

4.4 Analysis of Deep Beam Specimens

These beams were designed to reveal shear failure. This failure happens in deep beams, which have a shear span to depth ratio (a/d) less than 2 (**ACI 318M- 2005**). In the current research shear span to depth ratio (a/d) was (0.73), the span length is (600mm), the type of loading used in the test is two point symmetrical load. The failure line is outside middle third span just to present actual shear failure. Inclined cracks occur after applying the load near the reaction support at an angle of approximately 45° .

4.4-1 Shear Analysis for NSC and HPC Deep Beam Specimen

The ultimate design shear capacity for deep beam test specimens is calculated according to (**ACI-318/2005**) code. The design of beams considers that the beams will fail in shear before flexure failure by yielding of steel reinforcement, therefore the specimens were cast without web reinforcement.

Design of cross-sections subjected to shear failure shall be based on:

$$\phi V_n \geq V_u \dots \dots \dots (4 - 2)$$

Where ϕ strength reduction factor equal to 0.85

Where V_u is factored shear force at the section considered and V_n is nominal shear strength computed by :

$$V_n = V_c + V_s \dots\dots\dots (4 - 3)$$

Where V_c is nominal shear strength provided by concrete.

V_s = shear strength provided by web reinforced.

Shear strength V_c shall be computed by:

$$V_c = 0.17\sqrt{f_c'}bwd \dots\dots\dots (4 - 4)$$

Calculation of shear strength capacity for NSC and HPC specimens before exposure to fire flame temperature is given in Appendix A.

4.5 Effect of High Temperatures on Compressive Strength of Concrete

In order to obtain a better understanding of the behavior of reinforced concrete beams in flexure and shear after exposure to fire flame, it is important to study the mechanical properties of concrete subjected to fire flame firstly.

In the current study, three cubes of concrete with dimensions of (100mm) for HPC and NSC were fired at each level of heating for each type of concrete to obtain the compressive strength of concrete after heating and to detect the effect of fire flame on the compressive strength of NSC and HPC. Table (4-1) shows the test results for NSC and HPC, while Figures (4-1) to (4-4) show the relationship between compressive strength and fire temperature for NSC and HPC. It is obvious from the results that the compressive strength decreases with exposure to fire temperatures.

Table (4-1) Measured values of cube compressive strength before and after fire flame exposure

Type of concrete	Age at exposure (days)	Period of exposure (hour)	Cube Compressive strength (MPa)			fcua/fcub* Ratio		Type of cooling
			Temperature °C			2/1	3/1	
			25 (1)	400 (2)	700 (3)			
NSC	60	1	42	35.5	27	0.84	0.65	Air
				33.5	19	0.82	0.46	Water
		1.5		35	22	0.83	0.54	Air
				31.5	12	0.77	0.29	Water
HPC	60	1	89	65	45	0.73	0.50	Air
				52.5	38	0.59	0.43	Water
		1.5		52	42.5	0.58	0.48	Air
				45.5	35.5	0.51	0.40	Water

* fcua: cube compressive strength after burning

fcub: cube compressive strength before burning

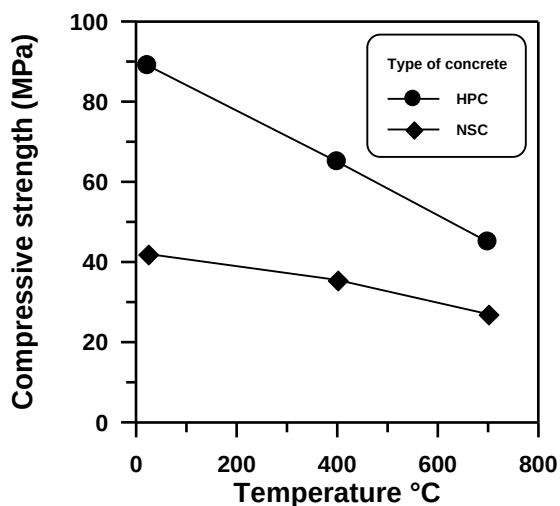


Fig (4-1): Relationship between compressive strength and fire temperature for (1) hour period of exposure and air cooling.

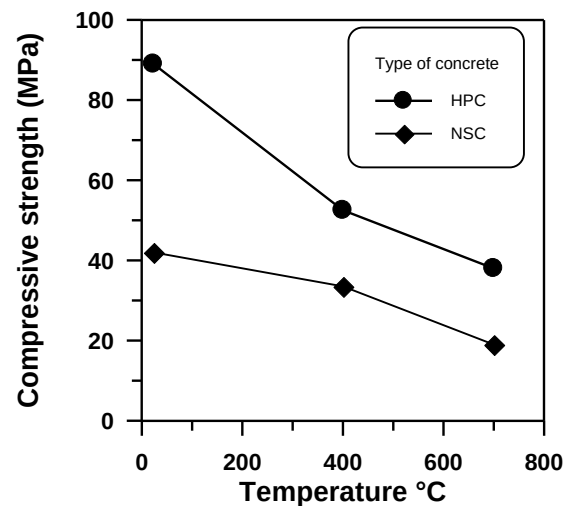


Fig (4-2): Relationship between compressive strength and fire temperature for (1) hour period of exposure and water cooling.

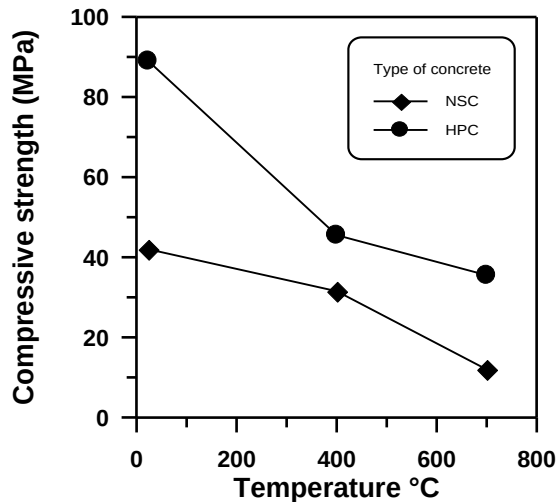


Fig (4-3): Relationship between compressive strength and fire temperature for (1.5) hour period of exposure and air cooling

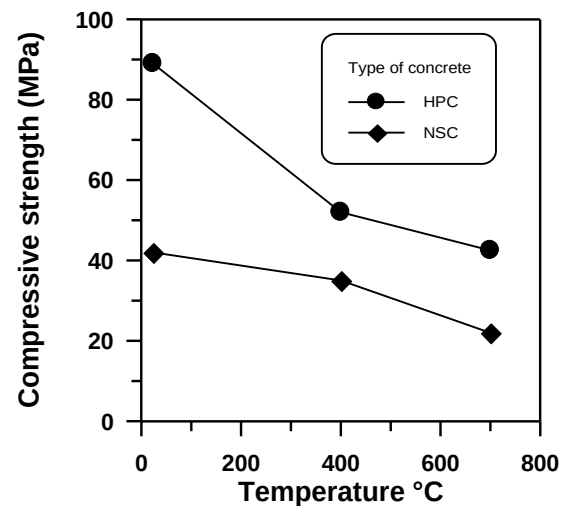


Fig (4-4): Relationship between compressive strength and fire temperature for (1.5) hour period of exposure and water cooling

4.5.1 Burning at 400 °C Fire Temperature:

The residual compressive strength values were recorded for the two series NSC and HPC at age 60 days and for two periods of exposure, then cooled with air and water.

For NSC, the percentage of the residual compressive strengths was (83-84%), (77-82) for air and water cooling respectively. This result agreed with that obtained by (Umran-2002) (70-85%).

For HPC, the percentage of the residual compressive strengths was (58-73%), (51-58) for air and water cooling respectively, which agreed with (Habeeb-2002) (55-87)%, (Castillo, 1990), 70%.

4.5.2 Burning at 700 °C Fire Temperature:

For NSC, the percentage of the residual compressive strengths was (54-65%), (29-46) for air and water cooling respectively.

For HPC, the percentage of the residual compressive strengths was (48-50%), (40-43) for air and water cooling respectively.

This difference shows that NSC has better resistance to high temperatures than HPC at temperatures up to (400 °C). When the

temperature is raised up to (700 °C) the NSC resistance to high temperatures is superior compared with HPC. The surface cracks in HPC appear more than NSC in number, length and depth with increase of temperature. The concrete specimens cooled in water exhibited more deterioration than the air-cooled specimens, the further reduction in strength was ranging from (2-4%), (8-10%) for NSC and (7-13%), (5-7%) for HPC. This is because of the destructive thermal shock produced when sprinkling the hot specimens in water (**Al-Ausi and Faiyadh-1985**) and the penetration of the water into the concrete pores and cracks. The decrease in compressive strength of concrete can be attributed to the break- down of interfacial bond due to incompatible volume changes between cement paste and aggregate during heating and cooling (**Moreley and Royels-1983**).

4.6 Splitting Tensile Strength

The splitting tensile strength results are summarized in Table (4-2) for both (NSC and HPC).

Figures (4-5) to (4-8) show the relationship between splitting tensile strength and fire flame temperature. Two cylinders of (100*200)mm were tested at (60) days age.

The tensile strength governs the cracking behavior and affects other properties such as stiffness and durability of concrete. The tensile strength is determined either by direct tensile test or by indirect tensile tests such as flexural or split cylinder tests (**Carrasquillo et al, 1987**).

Table (4-2) Results of splitting tensile strength before and after firing for NSC and HPC cylinders

Type of	Age at	Period of	Splitting tensile		Type of
---------	--------	-----------	-------------------	--	---------

Concrete	exposure (days)	exposure (hours)	strength (MPa)			fspa/fspb*		cooling
			Temperature °C			Ratio		
			25 (1)	400 (2)	700 (3)	2/1	3/1	
NSC	60	1	4.28	3.72	2.37	0.80	0.51	Air
				3.52	2.12	0.76	0.46	Water
		1.5		3.6	2.19	0.78	0.47	Air
				2.32	1.89	0.5	0.41	Water
HPC	60	1	6.89	5.4	2.76	0.78	0.40	Air
				4.62	2.68	0.67	0.39	Water
		1.5		4.3	2.68	0.62	0.39	Air
				4.21	2.55	0.61	0.37	Water

*fspa:splitting tensile strength after burning

fspb:splitting tensile strength before burning

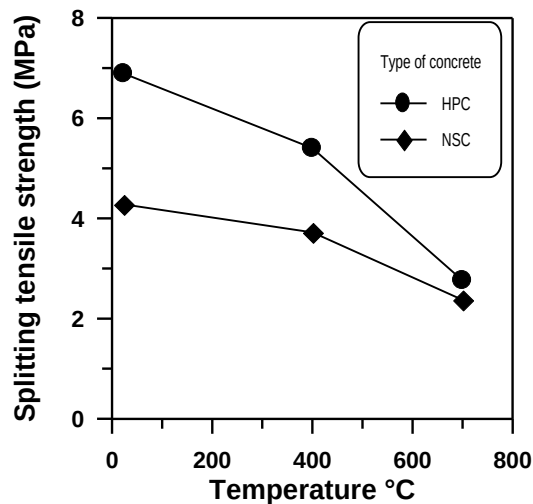


Fig (4-5): Relationship between splitting tensile strength and fire temperature for (1) hour period of exposure and air cooling

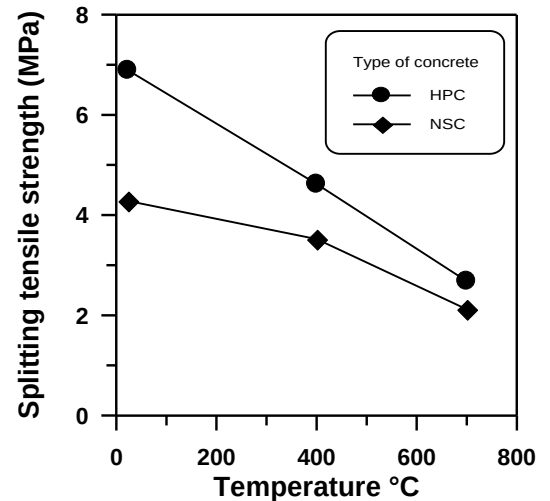


Fig (4-6): Relationship between splitting tensile strength and fire temperature for (1) hour period of exposure and water cooling

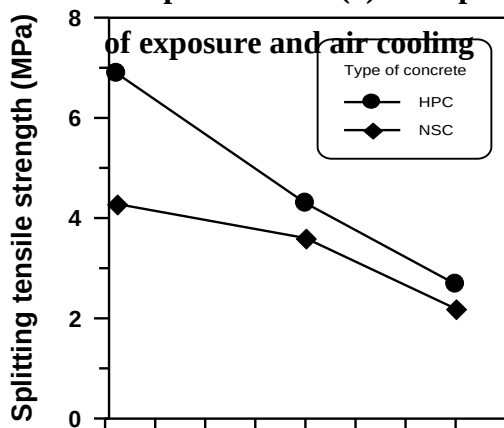


Fig (4-7): Relationship between splitting tensile strength and fire temperature for (1.5) hour period of exposure and air cooling

Fig (4-8): Relationship between splitting tensile strength and fire temperature for (1.5) hour period of exposure and water cooling

4.6.1 Burning at 400 °C Fire Temperature:

For NSC, the percentage of the residual splitting tensile strength was (78-80%), (50-76) for air and water cooling respectively. These results agreed with that obtained by other investigators, **(Neville and Brooks,1987)** 85% , **(Takeuchi, 1993)** 75% .

For HPC, the percentage of the residual splitting tensile strength was (62-78%), (61-67) for air and water cooling respectively.

4.6.2 Burning at 700 °C Fire Temperature:

Raising the temperature to 700 °C, the residual splitting tensile strength was (51-78%), (41-46%) for NSC with air and water cooling respectively, while the residual splitting tensile strength was (40-39%), (37-39) for HPC with air and water cooling respectively. It is obvious from the results that splitting tensile strength for HPC decreases about (8-11%) more than NSC. This result is consistent with the rule that, HPC is more sensitive to high temperatures because of its reduced porosity and low thermal diffusivity. The reduction in the splitting tensile strength can be attributed to the formation of tensile stresses during the contraction of the hardened cement paste upon cooling, which, when superimposed onto the already existing tensile stresses formed during heating would cause an increase in the amount and rate of crack formation.

4.7 Flexural Strength (Modulus of Rupture)

The flexural strength was measured by (100*100*400)mm normal strength concrete and high performance concrete prisms. Table (4-3) and Figures (4-9) to (4-12) show the relationship between residual flexural strength and fire flame temperature. The residual flexural strength (fra) after burning at different temperatures is expressed by the ratio fra/frb where frb is the flexural strength of concrete at the same age without burning.

Table (4-3) Test values of flexural strength before and after firing for NSC and HPC prisms.

Type of Concrete	Age at exposure (days)	Period of exposure (hour)	Flexural strength (MPa)			fra/frb* Ratios		Type of cooling
			Temperature °C			2/1	3/1	
			(1) 25	400 (2)	700 (3)			
NSC	60	1	4.54	1.91	1.23	0.42	0.27	Air
				1.73	1.1	0.38	0.24	Water
		1.5		1.78	0.77	0.39	0.17	Air
				1.68	0.73	0.37	0.16	Water
HPC	60	1	7.04	2.82	1.48	0.40	0.21	Air
				2.67	1.41	0.38	0.20	Water
		1.5		2.6	1.30	0.37	0.18	Air
				2.41	0.9	0.34	0.13	Water

* fra: flexural strength after burning

frb: flexural strength before burning

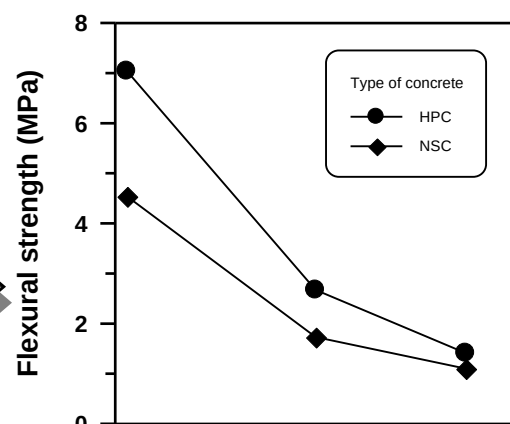
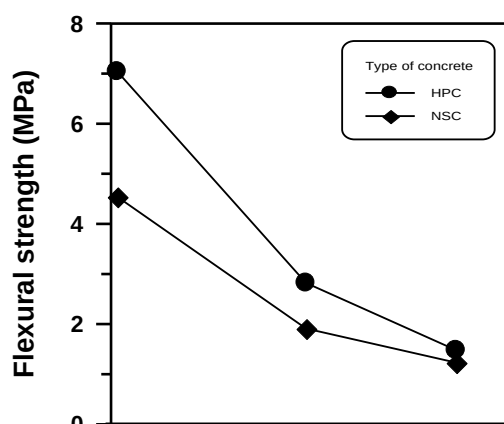


Fig (4-9): Relationship between flexural strength and fire temperature for (1) hour period of exposure and air cooling

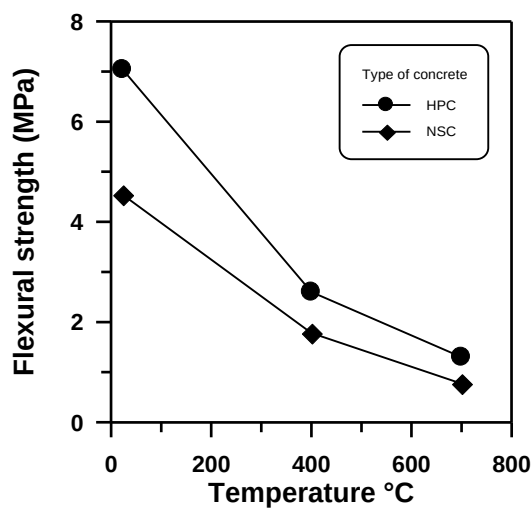


Fig (4-11): Relationship between flexural strength and fire temperature for (1.5) hour period of exposure and air cooling

Fig (4-10): Relationship between flexural strength and fire temperature for (1) hour period of exposure and water cooling

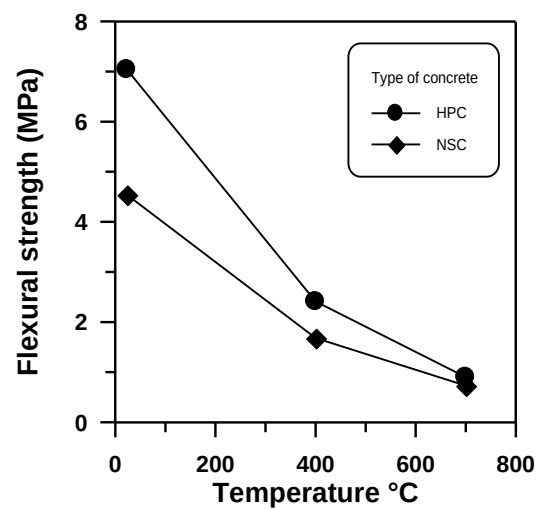


Fig (4-12): Relationship between flexural strength and fire temperature for (1.5) hour period of exposure and water cooling

4.7.1 Burning at 400 °C Fire Temperature:

The residual flexural strength values were calculated for the two types of concrete (NSC and HPC) after exposure to fire flame at age (60 days) then cooled by air or water.

For NSC, the residual flexural strengths were in the range of (37- 42%) (37- 38%) for air and water cooling respectively. The effect of fire was noticed to be in a range more than that obtained by **(Umran, 2002) 71%**. But the results agreed with that obtained by **(Purkiss,1984)** for NSC, who found 40% residual flexural strength.

For HPC, the residual flexural strengths were in the range of (34- 40%) (34- 38%) for air and water cooling respectively. In HPC hair- cracks and pink color were observed in the prisms. This trend is similar to that noticed by **(Habeeb, 2000)**.

4.7.2 Burning at 700 °C Fire Temperature:

For NSC, the residual flexural strengths were in the range of (16- 27%) (16- 24%) for air and water cooling respectively.

For HPC, the residual flexural strengths were in the range of (13- 21%) (13- 20%) for air and water cooling respectively. It was observed that HPC is more sensitive to the effect of fire flame than NSC.

It was noticed that when the specimens were sprinkled with water, the width of cracks increases and also, the number of cracks in HPC specimens has more intensity than that observed in NSC. HPC specimens possessed residual flexural strength close to that obtained by **(Habeeb-2000)**.

The secant modulus of elasticity is defined as the slope of the chord of the uniaxial stress-strain curve between the point corresponding to 40 percent of the maximum stress and the point corresponding to a strain of 0.00005. The modulus of elasticity of the NSC and HPC mixes is measured by (150*300) mm cylinders at 60-days according to **(BS 1881:part 121:1983)**.

It is generally agreed that the elastic modulus of concrete increases with its compressive strength. The modulus is greatly affected by the properties of the coarse aggregate, the higher the amount of coarse aggregate with a high elastic modulus, the higher would be the modulus of elasticity of concrete. The elastic modulus also increases with concrete age. Test results of the modulus of elasticity are summarized in Table (4-4). Figures (4-13) to (4-16) illustrates the relationship between modulus of elasticity and fire flame temperature for NSC and HPC. It can be seen that the reduction in values of concrete modulus of elasticity is more significant than of the compressive and flexural strength at identical fire flame temperatures.

Calculation of modulus of elasticity for HPC specimen before exposure to fire flame temperature is given in Appendix A – (A-5).

**Table (4-4) Test values of modulus of elasticity before and after firing
for NSC and HPC mixes**

Type of concrete	Age at of exposure (days)	Period of exposure (hours)	Modulus of elasticity (GPa)			Eca/Ecb* Ratios		Type of cooling
			Temperature °C					
			(1) 25	400 (2)	700 (3)	2/1	3/1	
NSC	60	1	30.97	21.97	8.52	0.71	0.27	Air
				21.05	7.75	0.68	0.25	Water
		1.5		19.92	6.83	0.64	0.22	Air
				18.25	6.31	0.59	0.20	Water
HPC	60	1	41.33	19.38	7.85	0.47	0.19	Air
				17.94	7.39	0.434	0.18	Water
		1.5		16.78	6.63	0.43	0.16	Air
				16.95	6.41	0.41	0.15	Water

* Eca : Concrete modulus of elasticity after burning

Ecb : Concrete modulus of elasticity before burning

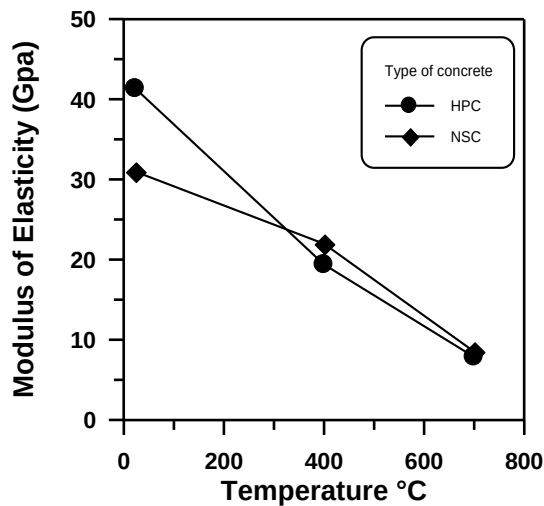


Fig (4-13): Relationship between modulus of elasticity and fire temperature for (1) hour period of exposure with air cooling.

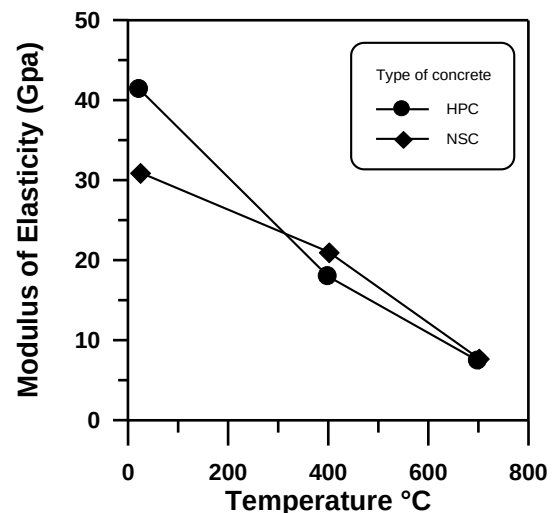


Fig (4-14): Relationship between modulus of elasticity and fire temperature for (1) hour period of exposure with water cooling.

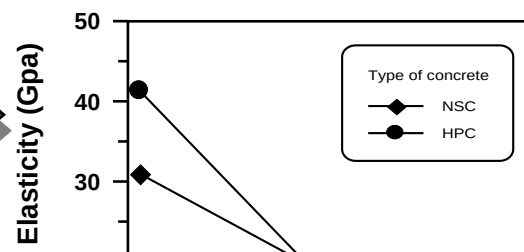
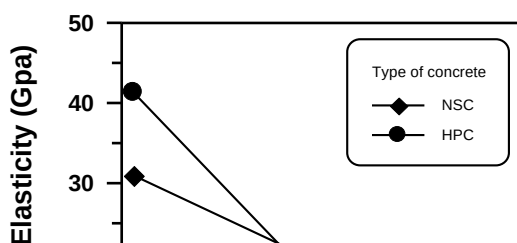


Fig (4-15): Relationship between modulus of elasticity and fire temperature for (1.5) hour period of exposure with air cooling.

Fig (4-16): Relationship between modulus of elasticity and fire temperature for (1.5) hour period of exposure with water cooling.

4.8.1 Burning at 400 °C Fire Temperature:

The residual modulus of elasticity was calculated for both NSC and HPC exposed to fire at (60days) and for two periods of exposure then cooled by air and water.

For fire temperatures around 400 °C, there was a significant reduction in the concrete modulus of elasticity due to effect of fire flame. The percentages of residual value of modulus of elasticity were in the ranges of (64-71%) and (59-68%) for NSC with air and water cooling respectively, while the percentages of residual value of modulus of elasticity were in the ranges of (43-47%) and (41-43%) for HPC with air and water cooling respectively.

4.8.2 Burning at 700 °C Fire Temperature:

From previous tables and figures, it can be observed that more reduction occurs in modulus of elasticity under 700 °C fire flame temperature.

For NSC, the residual modulus of elasticity was (22-28%) and (20-25%) for air and water cooling respectively.

For HPC, the residual modulus of elasticity was (16-19%) and (15-18%) for air and water cooling respectively.

The reduction in modulus of elasticity of concrete can be attributed to the increase in the amount of crack formation due to fire flame exposure, especially in HPC.

4.9 Description of Experimental Program of Beam Specimens

As mentioned previously, the test beam specimens are divided into two groups according to concrete type, and then these groups are subdivided into two subgroups according to the failure type that the specimen is designed to suffer, and finally they are divided according to the heating level, duration of fire flame and cooling method. These groups are described in Table (4-5) and (4-6).

Table (4-5) Designation of normal strength concrete beam specimens.

Major Group	Number of Specimen	Steel Ratio	Design Failure Type	Temperature Stage °C	Period of Exposure (hr)	Type of Cooling	Beam Specimen designation
NSC	2	0.0126	Flexure (yielding of steel reinforcement)	23	-	-	NFB
	4			400	1	Air	NF4-1A
						Water	NF4-1W
					1.5	Air	NF4-1.5A
						Water	NF4-1.5W
	4			700	1	Air	NF7-1A
						Water	NF7-1W
					1.5	Air	NF7-1.5A
						Water	NF7-1.5W
	2	0.004	Shear	23	-	-	NSB
	4			400	1	Air	NS4-1A
						Water	NS4-1W
					1.5	Air	NS4-1.5A
						Water	NS4-1.5W
	4			700	1	Air	NS7-1A
						Water	NS7-1W
					1.5	Air	NS7-1.5A
						Water	NS7-1.5W

Table (4-6) Designation of high performance concrete beam specimens.

Major	Number	Steel	Design	Temper-	Period of	Type	Beam
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Group	of Specimen	Ratio	Failure Type	ature Stage °C	Exposure (hr)	of Cooling	Specimen designation
HPC	2	0.0126	Flexure (yielding of steel reinforcement)	23	-	-	NFB
	4			400	1	Air	HPF4-1A
						Water	HPF4-1W
					1.5	Air	HPF4-1.5A
						Water	HPF4-1.5W
	4			700	1	Air	HPF7-1A
						Water	HPF7-1W
					1.5	Air	HPF7-1.5A
						Water	HPF7-1.5W
	2	0.004	Shear	23	-	-	HPSB
	4			400	1	Air	HPS4-1A
						Water	HPS4-1W
					1.5	Air	HPS4-1.5A
						Water	HPS4-1.5W
	4			700	1	Air	HPS7-1A
						Water	HPS7-1W
					1.5	Air	HPS7-1.5A
						Water	HPS7-1.5W

4.10 Residual First Crack and Ultimate Load of Flexural Strength for Beams Exposed to Fire Flame

The beam specimens reinforced with steel ratio of (0.0126) are designed to fail in flexure for the two types of concrete (NSC and HPC). The test results of the first crack load and ultimate load of the reference beam specimens and those exposed to fire are shown in Tables (4-7), (4-8) for NSC and HPC respectively.

From the results, it is clear that the values of the first crack and ultimate loads decrease when exposed to fire flame.

At fire temperature 400 °C, for NSC, the residual first crack and ultimate loads are (68-88%), (87-91%) for (1) hour exposure respectively, while the residual first crack and ultimate loads are (59-85%), (84-88%) for (1.5) hour exposure.

At fire temperature 700 °C, for NSC, the residual first crack and ultimate loads are (47-55%), (52-61%) for (1) hour exposure respectively, while the residual first crack and ultimate loads for (1.5) hour exposure are (35-41%), (26-42%) respectively.

The fire flame temperature applied to concrete beams causes evaporation of free moisture in the concrete. With a continual exposure to fire, the temperature inside the beam increases and the strength of concrete decreases. In certain cases, the pressure generated by conversion of moisture within beams may be too high for the surface layer of concrete to resist, and it may spall causing a reduction in concrete compressive strength. This reduction in the compressive concrete strength and yield strength of steel is causing a reduction in flexural strength with increase fire flame temperature.

Table (4-7): Test results of the first crack load, ultimate load and deflection for reference beams and beams exposed to fire flame for normal strength concrete

Specimen Identification	Temperature Stage °C	First Crack Load (kN)	Percentage Residual First Crack Load (%)	Ultimate Load (kN)	Percentage Residual Ultimate Load (%)	Max Deflection at mid span (mm)
NFB	23	8.5	100	29.100	100	6.125
NF4-1A	400	7.5	88.2	26.5	91	7.925
NF4-1W		5.8	68.2	25.2	86.6	7.95
NF4-1.5A		7.2	84.7	25.600	88	7.98
NF4-1.5W		5.0	58.8	24.500	84.2	8.10
NF7-1A	700	4.7	55.3	17.6	60.5	8.55
NF7-1W		4.0	47	15.400	53	8.625
NF7-1.5A		3.5	41.2	12.200	42	10.25
NF7-1.5W		3.0	35.3	7.600	26.1	10.625

For HPC at fire temperature (400 °C), the residual first crack and ultimate loads are (69-77%), (85-88%) for (1) hour exposure respectively, while the residual for (1.5) hour exposure are (57-61%), (70-71%) respectively.

At fire temperature 700 °C, the residual first crack and ultimate loads are (27-53%), (50-58%) for (1) hour exposure respectively, while the residual first crack and ultimate loads are (21-30%), (37-38%) for (1.5) hour exposure. It is obvious from the results that the effect of fire flame on the ultimate load (flexural strength) in beam specimens of normal strength concrete is less than the effect on the high performance concrete because the compressive strength decrease more in HPC than NSC.

Also, for fire temperature 700 °C, the decrease in the first crack load for (NSC) beam specimens is less than that for (HPC) beam specimens.

Table (4-8): Test results of the first crack load, ultimate load and deflection for reference beams and beams exposed to fire flame for high performance concrete.

Specimen Identification	Temperature Stage °C	First Crack Load (kN)	Percentage Residual First Crack Load (%)	Ultimate Load (kN)	Percentage Residual Ultimate Load (%)	Max Deflection at mid span (mm)
HPFB	23	15	100	38.100	100	10
HPF4-1A	400	11.6	77.3	33.5	88	8.7
HPF4-1W		10.3	68.7	32.2	84.5	9.375
HPF4-1.5A		9.1	60.7	27.5	72.2	7.25
HPF4-1.5W		8.5	56.7	26.8	70.3	7.75
HPF7-1A		8.0	53.3	22.2	58.3	8.00
HPF7-1W	700	4.0	26.7	19.0	49.83	8.45
HPF7-1.5A		4.5	30	14.6	38.3	9.125
HPF7-1.5W		3.1	20.7	14.3	37.5	9.675

4.11 Residual First Crack and Ultimate Loads of Shear Testing Beams Exposed to Fire Flame

The beam specimens reinforced by (0.004) steel ratio with a/d ratio of (0.73) are designed to fail in shear for the two types of concrete (NSC and HPC).

The test results of the first crack and ultimate loads of the reference beam specimens and those exposed to fire flame are shown in Tables (4-9) and (4-10) for NSC and HPC respectively.

From the results, it is clear that the values of residual first crack load decreases when the beams are exposed to fire flame.

At fire temperature 400 °C, for NSC, residual first crack and ultimate loads are (98-99%), (78-93%) for (1) hour exposure respectively, while the residual first crack and ultimate loads are (91-89%), (75-77%) for (1.5) hour exposure respectively. At fire temperature 700 °C, for NSC, residual first crack and ultimate loads are (63%), (51-57%) for (1) hour exposure

respectively, while the residual first crack and ultimate loads in (1.5) hour exposure are (42-43%), (38-47%) respectively.

From Tables (4-8) and (4-9), It is obvious that shear strength undergoes more reduction to fire flame than flexural strength by about (5-8%).

Table (4-9): Test results of the first crack load, ultimate load and deflection for reference beams and beams exposed to fire flame for normal strength concrete.

Specimen Identification	Temperature Stage °C	First Crack Load (kN)	Percentage Residual First Crack Load (%)	Ultimate Load (kN)	Percentage Residual Ultimate Load (%)	Max Deflection at mid span (mm)
NSB	23	19.2	100	32.5	100	1.29
NS4-1A	400	19.0	99	30.2	92.9	0.72
NS4-1W		18.8	98	25.2	77.5	0.81
NS4-1.5A		17.5	91.1	24.3	74.8	1.375
NS4-1.5W		17.1	89.1	25.1	77.2	2.25
NS7-1A	700	12.0	62.5	18.5	57	3.45
NS7-1W		12.0	62.5	17.5	50.8	3.75
NS7-1.5A		8.3	43.2	15.1	46.8	3.96
NS7-1.5W		8.0	41.7	12.4	38.2	4.42

For HPC at (400 °C) burning temperature, the residual first crack and ultimate loads are (82-100%), (90-98%) for (1) hour exposure respectively, while the residual first crack and ultimate loads for (1.5) hour exposure are (69-76%), (88-92%) respectively.

At 700 °C burning temperature, for HPC, the residual first crack load and ultimate loads are (71-72%), (50-55%) for (1) hour exposure respectively, while the residual first crack load and ultimate loads for (1.5) hour exposure are (26-29%), (40-45%) respectively.

It is clear from the ultimate load test results that shear strength decreases more than flexural strength because the values of shear strength are directly related to compressive strength which decreases with raising fire flame temperature for beam specimens.

Table (4-10): Test results of the first crack load, ultimate load and deflection for reference beams and beams exposed to fire flame for high performance concrete.

Specimen Identification	Temperature Stage °C	First Crack Load (kN)	Percentage Residual First Crack Load (%)	Ultimate Load (kN)	Percentage Residual Ultimate Load (%)	Max Deflection at mid span (mm)
HPSB	23	35.0	100	62.5	100	2.5
HPS4-1A	400	35.0	100	61.1	97.8	1.625
HPS4-1W		28.8	82.3	56.1	89.8	2.375
HPS4-1.5A		26.5	76	57.3	91.7	1.475
HPS4-1.5W		24.2	69	55.6	88.46	1.15
HPS7-1A	700	25.3	72.3	34.5	55.2	3.25
HPS7-1W		25.0	71.4	31.5	50.4	2.375
HPS7-1.5A		10.3	29.4	28.2	45	1.55
HPS7-1.5W		9.1	26	25.3	40.5	1.45

Figures (4-17) to (4-20) show the effect of fire flame temperature on the residual first crack and ultimate loads. For (1) and (1.5) hour periods of exposure. HPC reveals more reduction in first crack and ultimate loads. This can be attributed to hydration of free lime and formation of Ca(OH)_2 due to absorption of moisture by effect of fire flame, so expansion takes place, which causes cracking and reduces the ultimate load capacity.

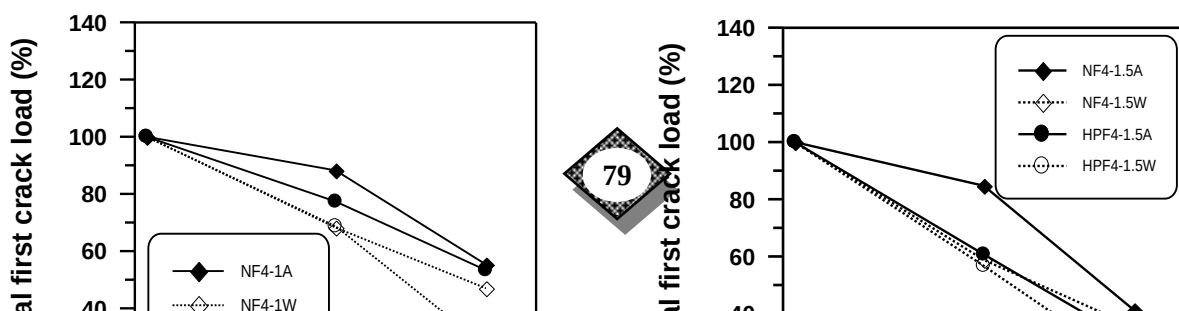


Figure (4-17): Effect of fire flame on the first crack load for the shallow beam specimens for (1) hour period of exposure

Figure (4-18): Effect of fire flame on the first crack load for the shallow beam specimens for (1.5) hour period of exposure

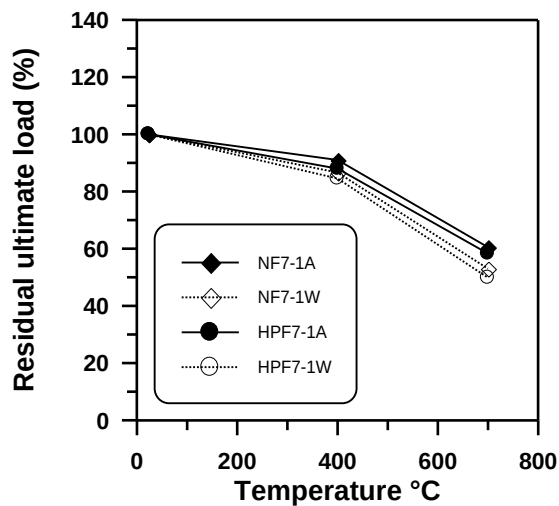


Figure (4-19): Effect of fire flame on the ultimate load for the shallow beam specimens for (1.5) hour period of exposure

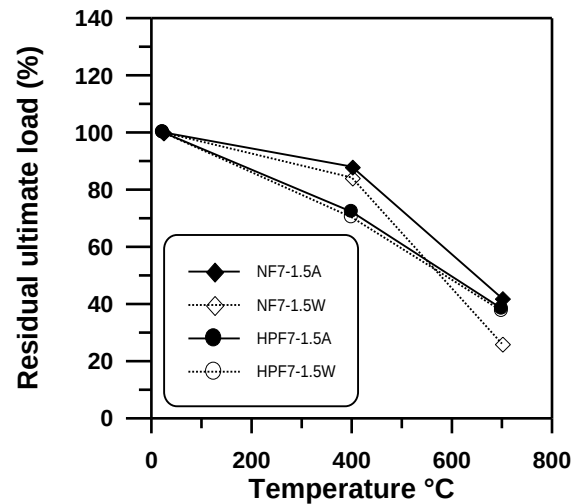


Figure (4-20): Effect of fire flame on the ultimate load for the shallow beam specimens for (1.5) hour period of exposure

4.12 Load-Deflection Relationship

4-12-1 Flexural Beam Specimens

The load-deflection relationship of flexural beam specimens having a steel ratio (0.0126) for NSC and HPC beams are presented in Figures (4-21) to (4-25).

Figure (4-21) shows the behavior of the flexural beam specimens at temperature (23 °C). The two beam specimens NSC and HPC are seem to have a close initial linear elastic behavior.

For burning temperature (400 °C), Figure (4-22) shows that the elastic behavior of the (NF4-1A) and (HPF4-1A) specimens are close up to approximately the same yield points, and the HPC beam gives less deflection than NSC beam specimens. The beam specimens (NF4-1W) and (NF4-1A) show difference in elastic and plastic stages. This difference is because of the pronounced cracking initiated when sprinkling the specimens with water. For burning temperature (700 °C), Figure (4-23) shows that the load deflection relations of beam specimens are softer in general compared with those of lower burning temperature. The deflection of the specimens which were sprinkled with water are higher than values of specimens which were cooled with air. The behavior of the HPC beams remain superior in spite of their sensitivity to high temperatures, as recorded in compressive strength tests.

The deflection progresses rapidly in the beam specimens (NF4-1A) and (NF4-1W) more than in beam specimens (HPF4-1A) and (HPF4-1W).

When the fire flame temperature increases to 700 °C the ultimate load values decrease, and also the deflection values increase rapidly as shown in Figure (4-24) and (4-25).

4-12-2 Deep Beam Specimens

The load-deflection relationships of deep beam specimens which have a steel ratio (0.004), for NSC and HPC are presented in Figures (4-26) to (4-30) for (23 °C, 400 °C, 700 °C).

At temperature (23 °C), Figure (4-26) shows that load deflection curves exhibit three stages in NSC rather than in HPC.

The first stage is the elastic or uncracked stage, the second stage is multiple cracking stage, and the third is the plastic stage. When the deep beam specimens arrived ultimate load stage sudden cracking sound is heard upon failure of HPC specimens rather than in NSC specimens.

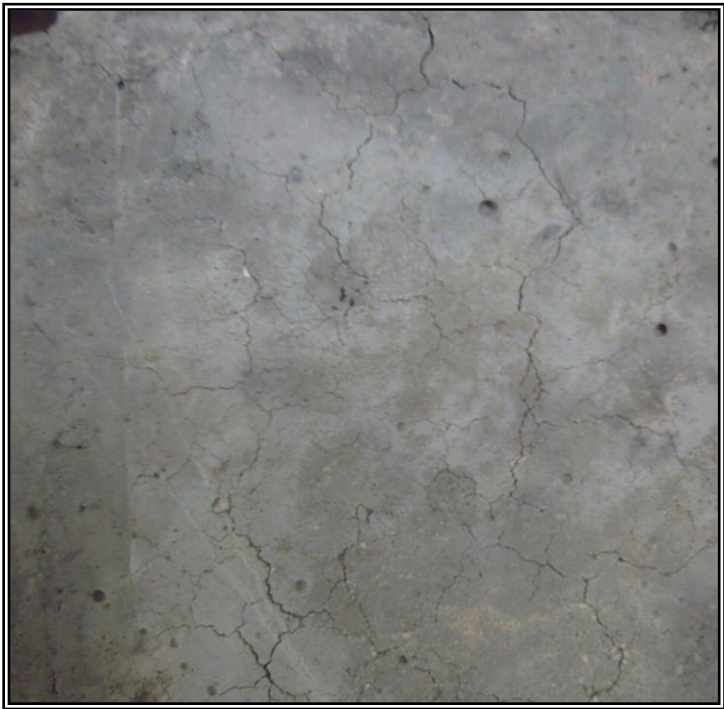
For burning temperature (400 °C), Figure (4-27) shows that the load-deflection behavior of the (NS4-1A), (NS4-1W), (HPS4-1A) and (HPS4-1W) specimens is very close to each other especially in the elastic stage .

The deflection values in the two specimens NSC and HPC are very low compared with the values of the flexure beam specimens.

The (HPS4-1.5W) specimen which was cooled by water gives deflection values less than that of the specimen (HPS4-1.5A). This trend is opposite in NSC.

The HPC specimens exposed to burning temperature (700 °C) for period of fire flame exposure (1) hour then cooled by water give deflection values less than that which were cooled by air while opposite trend was recorded in NSC. Also HPC specimens with period of fire flame exposure of (1.5) hour then cooled by water gives deflection values more than that for specimens cooled by air while opposite trend was recorded in NSC.

The behavior of the HPC beam specimens shows brittle behavior, and the crack formation in HPC beam specimens at high temperature is more intensive than that of NSC. These observations were noticed in the HPC beam specimens as shown in Plate (4-1).



NSC



HPC

Plate (4-1) Crack formation in NSC and HPC

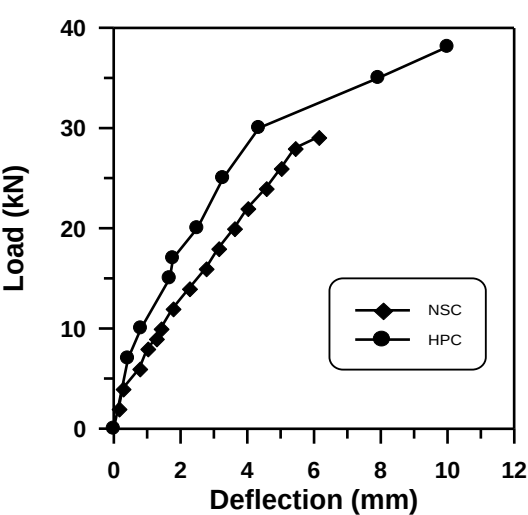


Figure (4-21) Load-deflection relationship for NSC and HPC flexural beam specimens at (23 °C)

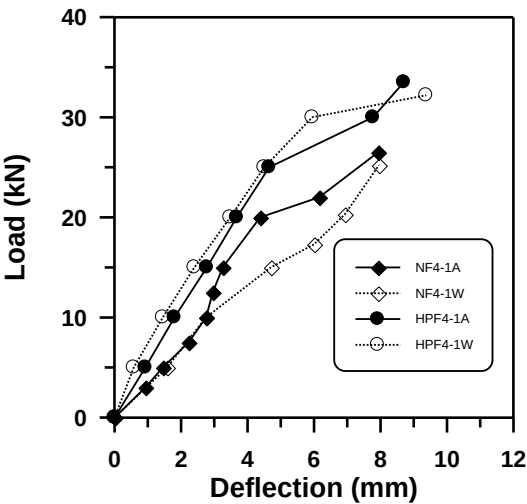


Figure (4-22) Load-deflection relationship for NSC and HPC flexural beam specimens after burning at (400 °C)

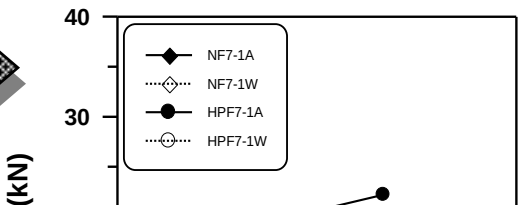
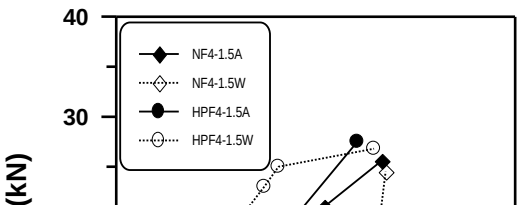


Figure (4-23) Load-deflection relationship for NSC and HPC flexural beam specimens after burning at (400 °C)

Figure (4-24) Load-deflection relationship for NSC and HPC flexural beam specimens after burning at (700 °C)

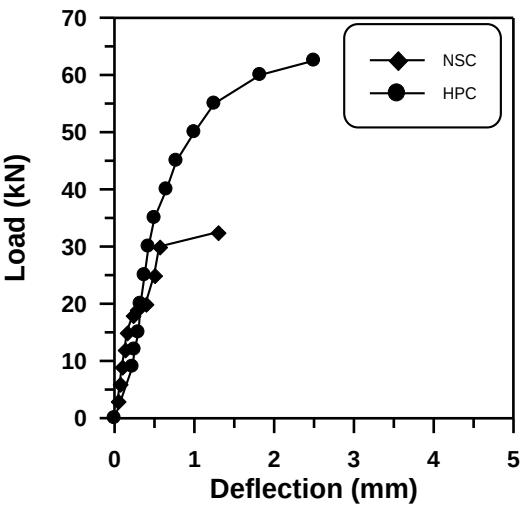
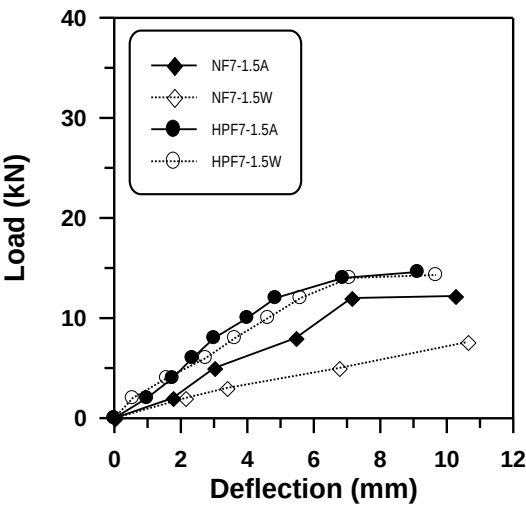


Figure (4-25) Load-deflection relationship for NSC and HPC flexural beam specimens after burning at (700 °C)

Figure (4-26) Load-deflection relationships for NSC and HPC deep beam specimens at (23 °C)

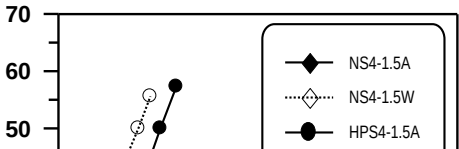
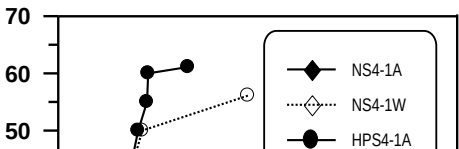


Figure (4-27) Load-deflection relationship for NSC and HPC deep beam specimens after burning at (400 °C)

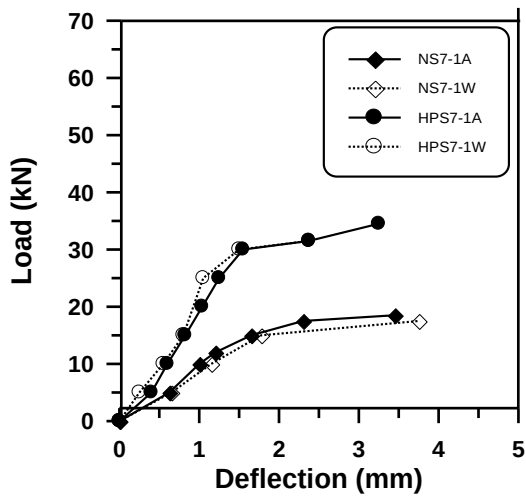


Figure (4-29) Load-deflection relationship for NSC and HPC deep beam specimens after burning at (700 °C)

Figure (4-28) Load-deflection relationship for NSC and HPC deep beam specimens after burning at (400 °C)

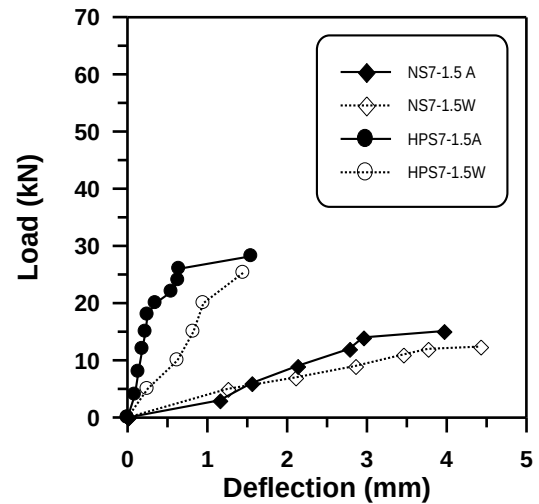


Figure (4-30) Load-deflection relationship for NSC and HPC deep beam specimens after burning at (700 °C)

Figure (4-31) to (4-35) shows the difference in deflection at identical load for flexural and deep beam specimens. The load deflection curves for deep beams came closer from shallow beams. This means that shear is more sensitive to fire flame for NSC and cracking of flexural beam specimens is more intensive in action than the deep beam specimens. The closer increased when the fire temperature is rise from (400 °C) to (700 °C) This can be attributed to the fact that heating causes a reduction in beam stiffness which is essentially due to the reduction in the modulus of elasticity of concrete, which means that load- deflection curves for NSC are more sensitive to high temperatures compared with HPC.

Figure (4-36) to (4-40) shows that the load- deflection curves for deep beams showed less sensitivity to fire flame temperature than NSC, therefore the load- deflection curves for deep beams become away from the load- deflection curves for shallow beams.

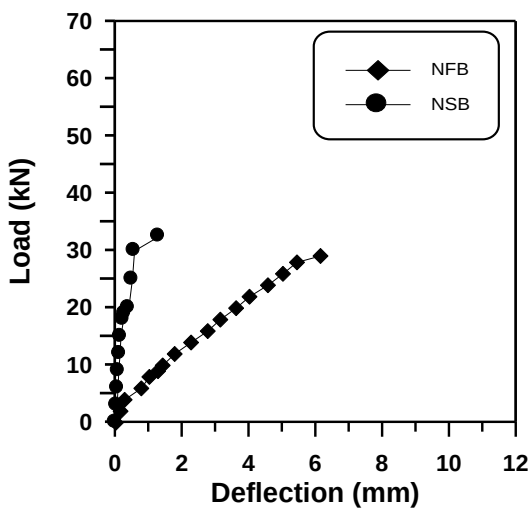


Figure (4-31) Load-deflection relationship for NSC shallow beam and deep beam specimens at (23 °C)

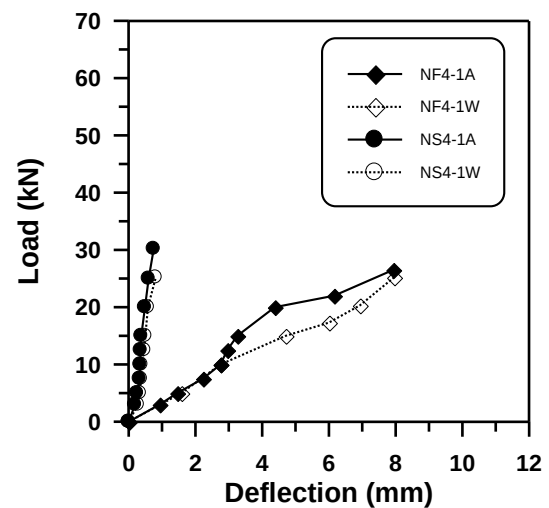


Figure (4-32) Load-deflection relationship for NSC shallow beam and deep beam specimen after burning at (400 °C)

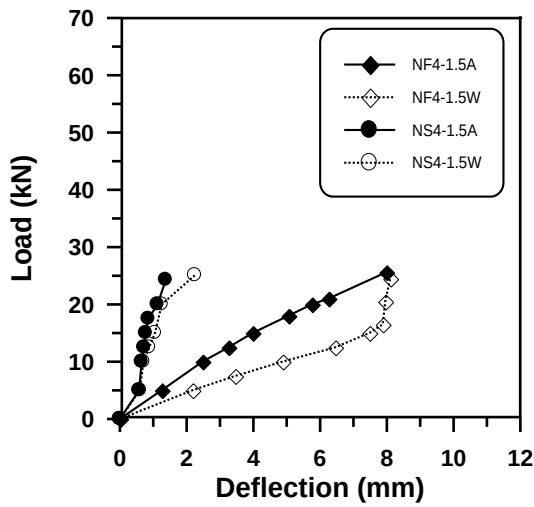


Figure (4-33) Load-deflection relationship for NSC shallow beam and deep beam specimens after burning at (400 °C)

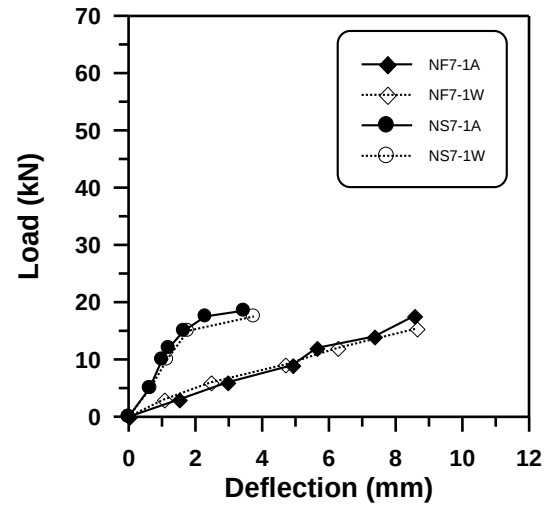


Figure (4-34) Load-deflection relationship for NSC shallow beam and deep beam specimens after burning at (700 °C)

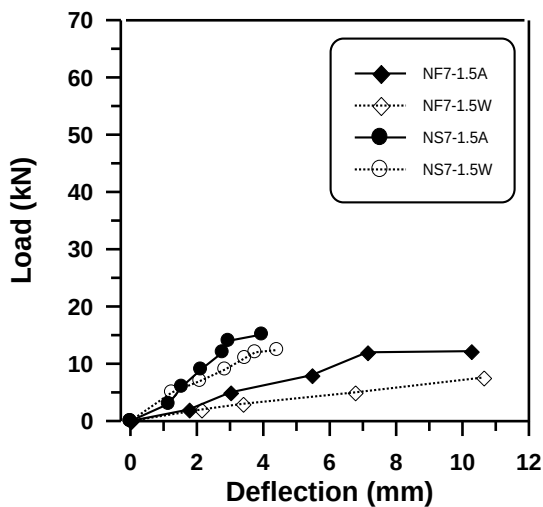


Figure (4-35) Load-deflection relationship for NSC shallow beam and deep beam specimens after burning at (700 °C)

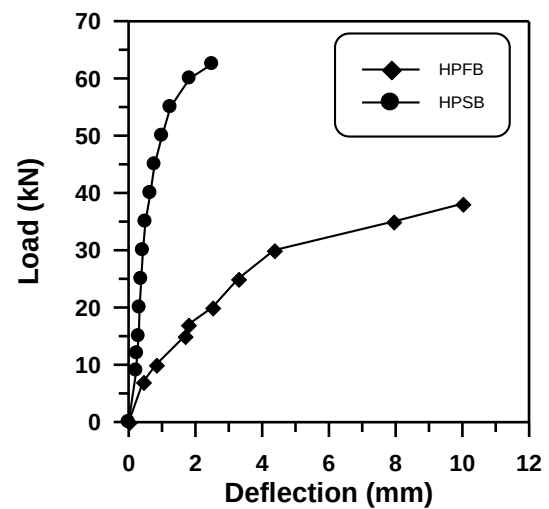


Figure (4-36) Load-deflection relationship for HPC shallow beam and deep beam specimens at (23 °C)

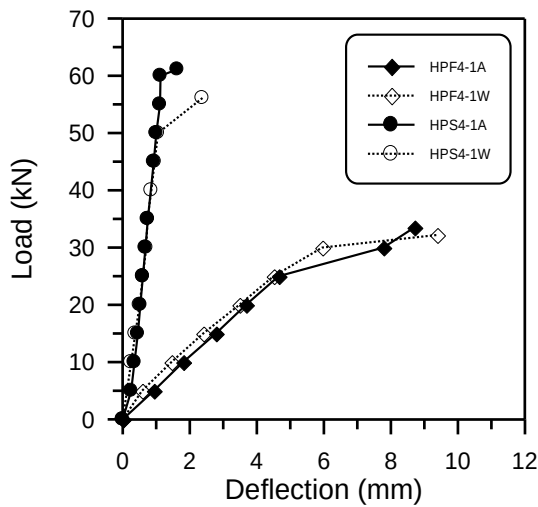


Figure (4-37) Load-deflection relationship for HPC shallow beam and deep beam specimens after burning at (400 °C)

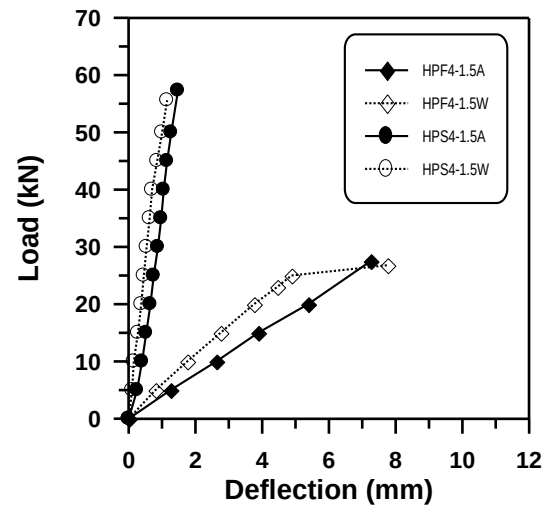


Figure (4-38) Load-deflection relationship for HPC shallow beam and deep beam specimen after burning at (400 °C)

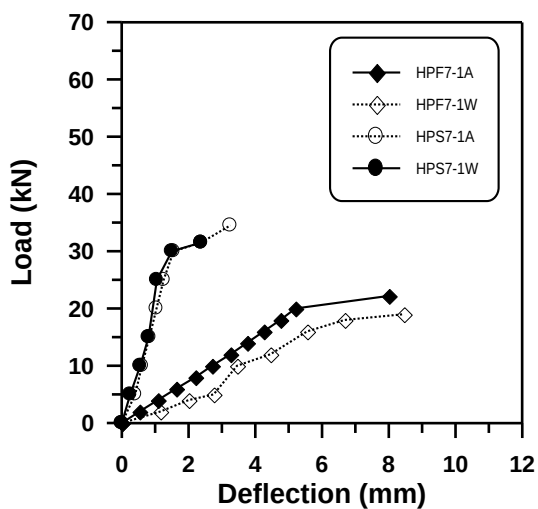


Figure (4-39) Load-deflection relationship for HPC shallow beam and deep beam specimens after burning at (700 °C)

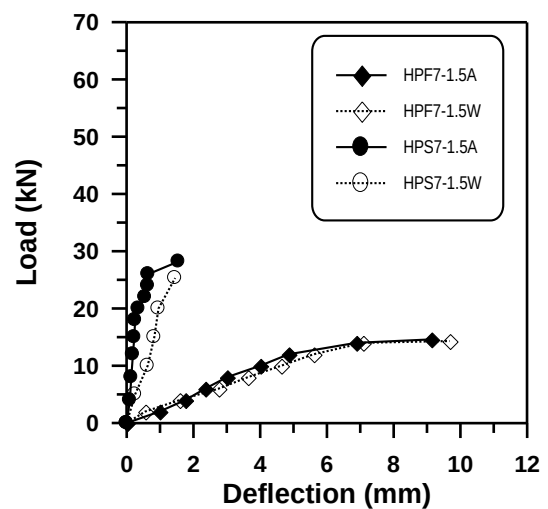


Figure (4-40) Load-deflection relationship for HPC shallow beam and deep beam specimens after burning at (700 °C)

4.13 Concrete Strain Distribution

The strain distribution along the depth of the beam cross section had been measured by gauged extensometer using four pairs of demec points. These demec points were placed on both sides of the mid span section of the beam with a gauged distance between each pair of (200mm) for flexural beam specimens and (150mm) for deep beam specimens. The concrete strain distribution along the depth of some tested beams at different load levels for flexural and deep beam specimens are shown in Figure (4-41) to (4-52).

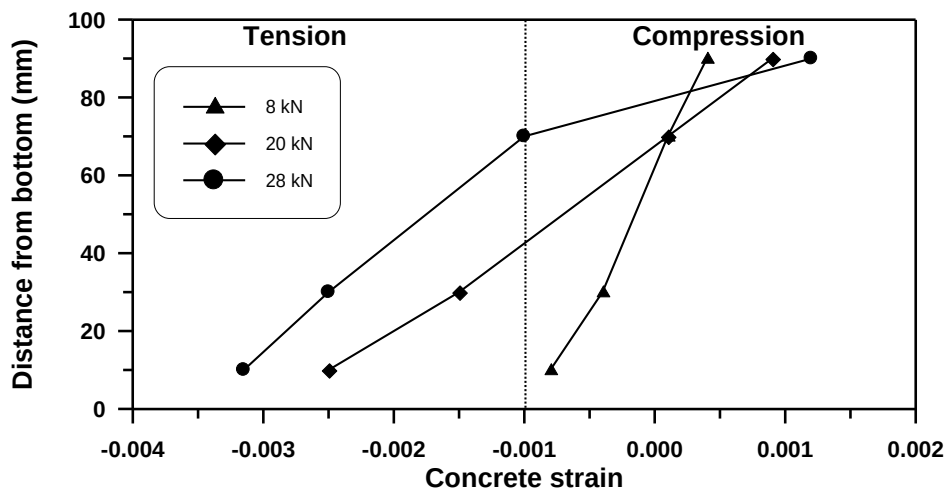


Figure (4-41) Concrete strain distribution at different load levels for flexural beam (NFB)

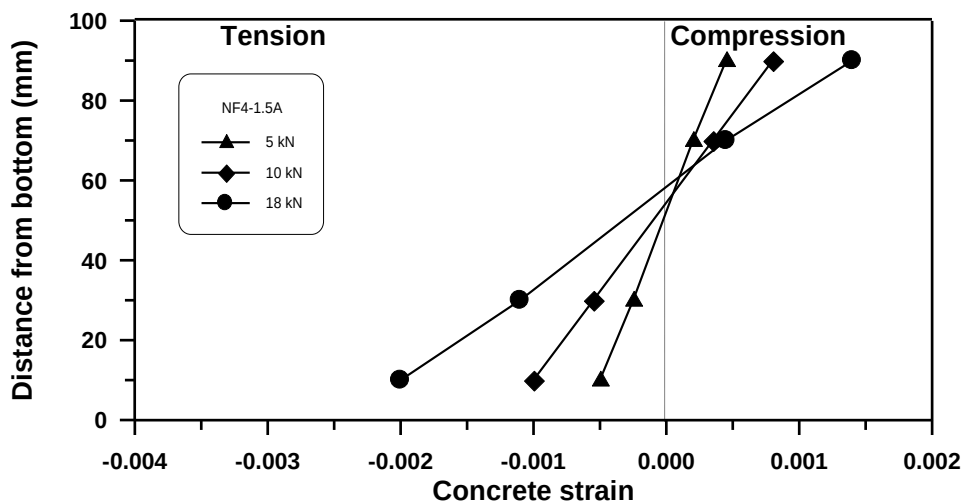


Figure (4-42) Concrete strain distribution at different load levels for flexural beam (NF4-1.5A)

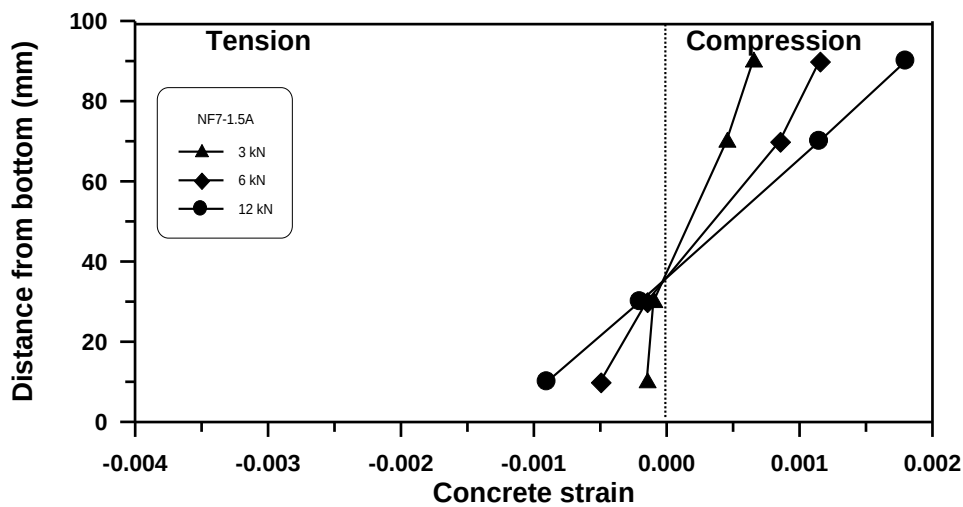


Figure (4-43) Concrete strain distribution at different load levels for flexural beam (NF7-1.5A)

Figures (4-41), (4-42) and (4-43) shows the variation of strain at mid-span section across the depth at different load stages for flexural beams in NSC. As shown in these figures the variation of strain across the depth is linear. At the same time the neutral axis was shifted towards the tension zone when the fire flame temperature increased.

Figure (4-44) shows the strain distribution of HPC flexural beam. It can be seen that when the load increases the neutral axis moves towards the compression zone. At burning temperature (400 °C) as shown in Figure (4-45), the neutral axis was shifted away towards the compression zone. Also the strains which occur are more intense in tension zone than in compression zone, while at burning temperature (700 °C) the strains are more intense in compression zone than in tension zone as shown in Figure (4-46).

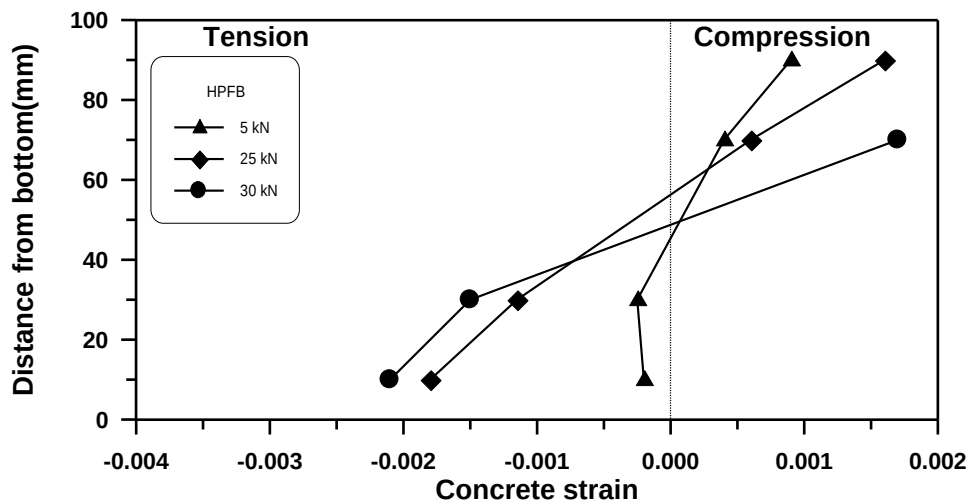


Figure (4-44) Concrete strain distribution at different load levels for flexural beam (HPFB)

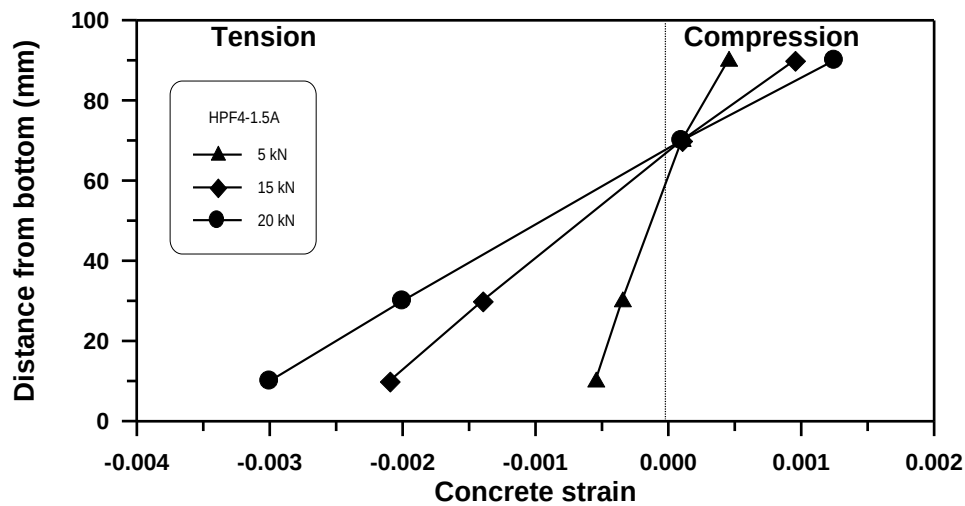


Figure (4-45) Concrete strain distribution at different load levels for flexural beam (HPF4-1.5A)

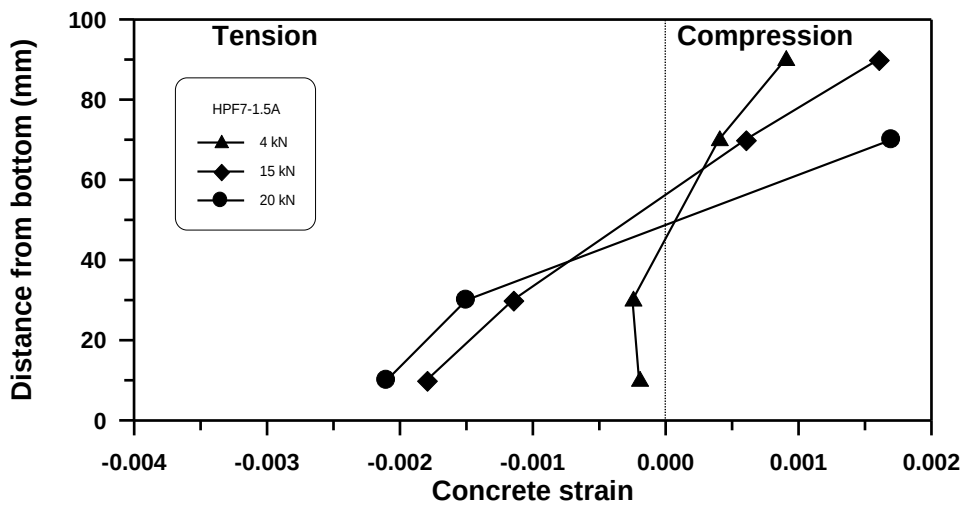


Figure (4-46) Concrete strain distribution at different load levels for flexural beam (HPF7-1.5A)

Figures (4-47), (4-48) and (4-49), shows the strain distribution at mid-span across section a long the depth at different load stages for NSC deep beams. As shown in these figures the strains are very low in deep beams compared with flexural beams. Also when the fire flame temperature increases the strains in deep will be beam more pronounced clearly. The specimens (NS4-1.5A) and (NS7-1.5A) shows that the neutral axis is always more closer to the top fiber.

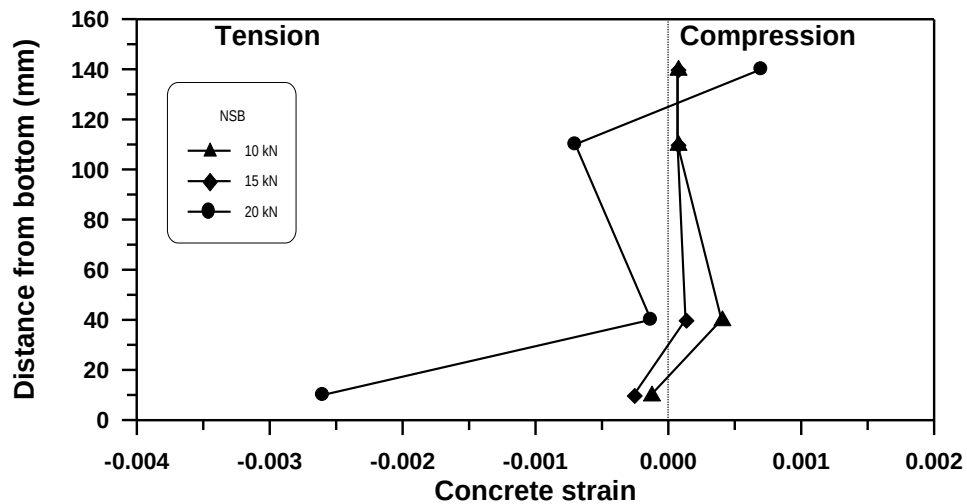


Figure (4-47) Concrete strain distribution at different load levels for (NSB) deep beam

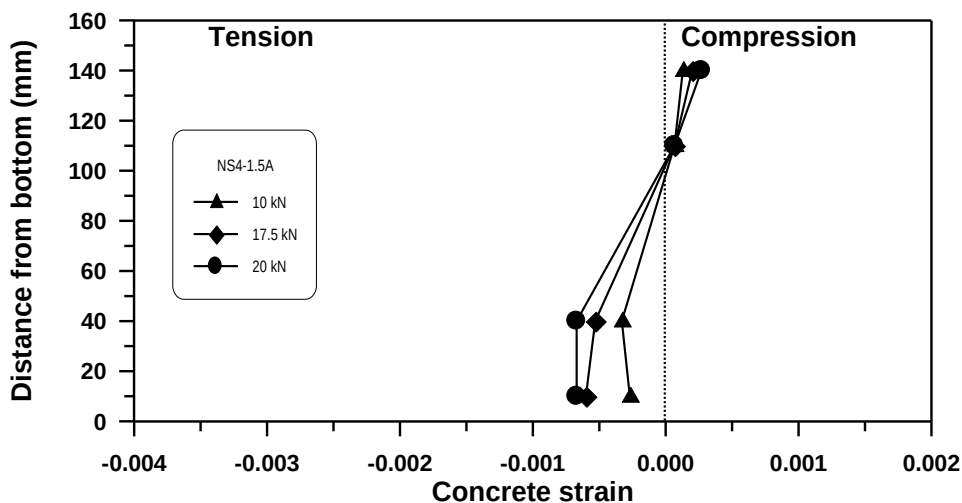


Figure (4-48) Concrete strain distribution at different load levels for deep beam (NS4-1.5A)

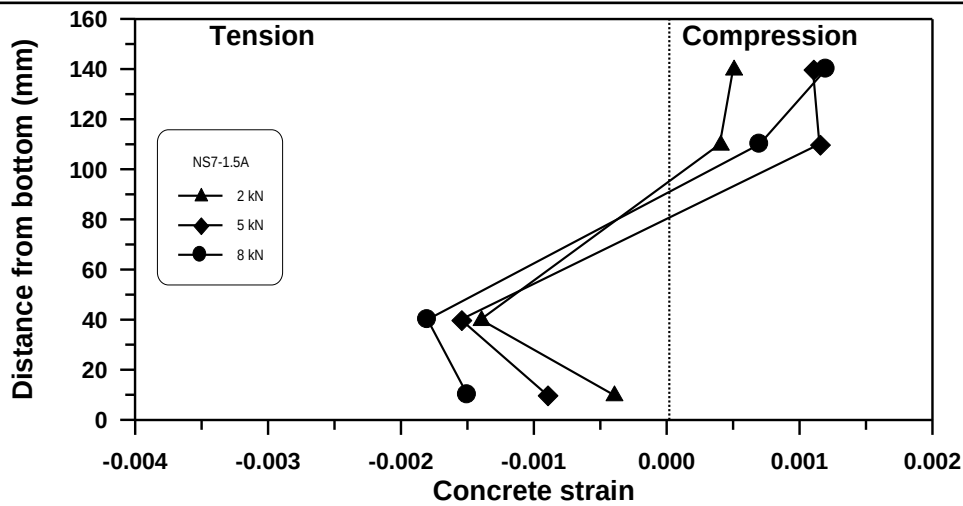


Figure (4-49) Concrete strain distribution at different load levels for deep beam (NS7-1.5A)

Figure (4-50) gives the strain distribution of HPC deep beam. It can be seen that the strain distribution is approximately erratic in compression and tension zone throughout the loading range, and also becomes nonstable at higher loads. At burning temperature (400 °C) Figure (4-51) shows that the neutral axis shifts towards the compression zone more than the specimen (NS4-1.5A) when the loads increased. This indicated that the strains increased in HPC when exposure to fire flame. At burning temperature (700 °C)[Figure (4-52)] when the load is low the neutral axis is more closer to bottom fiber because a crack occurs early in that temperature.

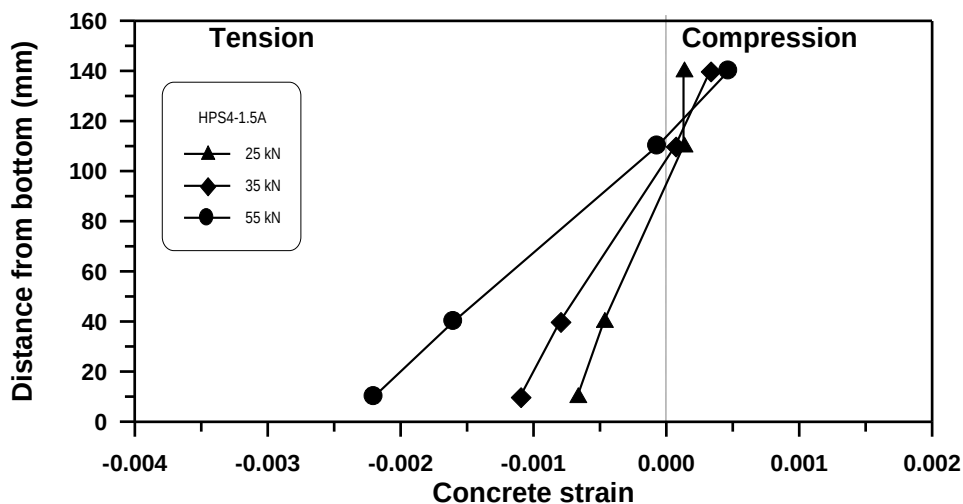


Figure (4-50) Concrete strain distribution at different load levels for deep beam (HPSB)

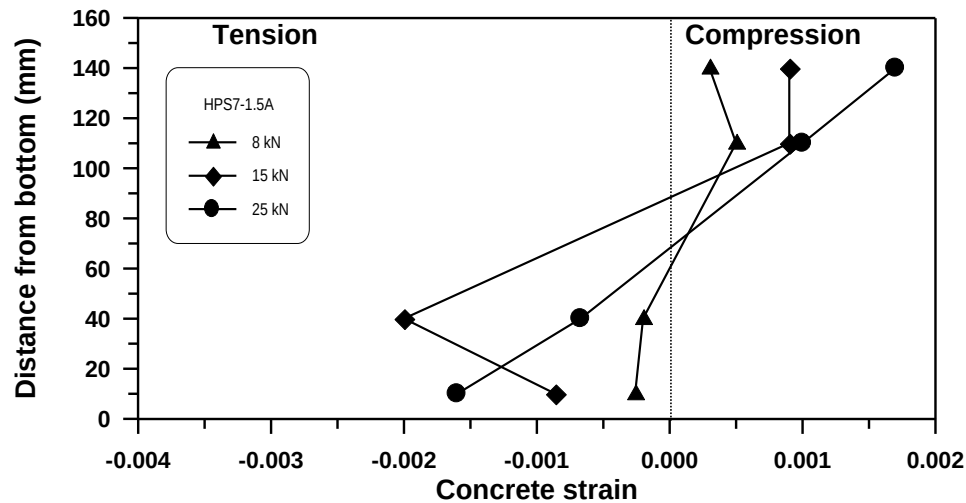


Figure (4-51) Concrete strain distribution at different load levels for deep beam (HPS4-1.5A)

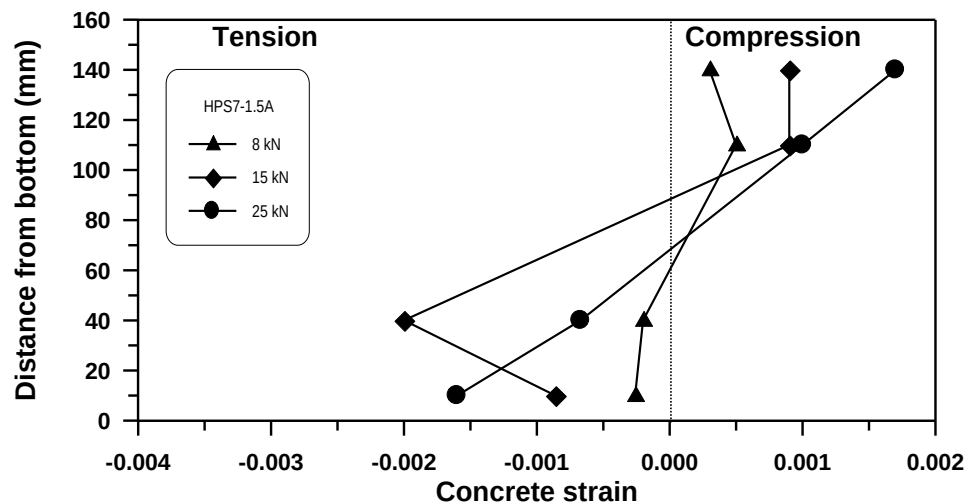


Figure (4-52) Concrete strain distribution at different load levels for deep beam (HPS7-1.5A)

4.14 The Effect of Fire Flame on the Reinforcing Steel Bars

The effect of fire flame on the properties of reinforcing steel bars is summarized in Table (4-11). At 400°C burning temperature, both burning and subsequent cooling did not affect significantly the mechanical properties of the reinforcing steel bars. But significant effect was observed

at 700°C burning temperature, the residual yield stress was (78.3%) and the residual ultimate stress was (81%) for the bar of 5.8 mm diameter, as for the bar of 8 mm diameter the residual yield stress was (80%) and the residual ultimate stress was (81.8%).

Table (4-11): The effect of fire flame on properties of steel bars

Bar diameter (mm)	Exposure Temp. (°C)	Yield stress N/mm ²	Residual Yield Stress %	Ultimate Stress N/mm ²	Residual Ultimate Stress %
5.8	25	650	100	744	100
	400	650	100	744	100
	700	509	78.3	603	81
8	25	540	100	670	100
	400	540	100	670	100
	700	432	80	548	81.8

4.15 Residual Load Bearing Capabilities of Fire-Exposed Reinforced Concrete Beams:

4-15-1 Flexural Capacity of RC Beams Exposed Fire.

The beam specimens reinforced with steel ratio of (0.0126) are designed to fail in flexure (yielding of steel reinforcement) for the two types of concrete (NSC and HPC).

From the test results shown in Table (4-12) and (4-13), it is clear that the values of bending moment capacity decrease when the beams were exposed to fire flame for the two fire temperatures 400 °C and 700 °C .

For NSC specimens at burning temperature (400 °C), the residual bending moment capacity was (86-91%), (84-88%), as for HPC specimens the residual bending moment capacity was (84-88%), (70-72%) for 1.0 and 1.5 hour exposure periods respectively.

For NSC specimens at burning temperature (700 °C), the residual bending moment capacity was (53-60%), (26-41%), as for HPC specimens the residual bending moment capacity was (50-58%), (37-38%) for 1.0 and 1.5 hour exposure periods respectively.

From these results, it can be seen that the bending moment capacity for HPC is more sensitive to fire flame temperature than NSC. This can be attributed to the higher density and lesser voids, most of the water is adsorbed causing a higher loss in bending moment for HPC. This finding must be taken into account when HPC structure is exposed to fire.

4-15-2 Shear Capacity of RC Beams Exposed to Fire.

The beam specimens reinforced with steel ratio of (0.004) are designed to fail in shear for the two types of concrete (NSC and HPC).

The value of shear capacity is related to compressive strength, which decreases when the concrete beam is exposed to high temperature. The shear capacity test results are given in Tables (4-14) and (4-15).

At burning temperature (400 °C), for NSC specimens the residual shear capacity was (78-93%), (75-77%) for 1.0 and 1.5 hour periods respectively. For HPC beam specimens the residual shear capacity was (90-98%), (89-92%) for 1.0 and 1.5 hour exposure periods respectively.

It can be indicated that HPC shear strength has an obvious resistance to high fire temperatures more than of NSC.

At burning temperature (700 °C), for NSC specimens the residual shear capacity was (54-57%), (38-46%) for 1.0 and 1.5 hour exposure periods respectively. For HPC specimens the residual shear capacity was (50-55%), (40-45%) for 1.0 and 1.5 hour exposure periods respectively.

From the test results it can be indicated that the residual shear capacity for HPC and NSC beam specimens become closer at fire temperature 700 °C.

4.16 Verification of ACI Building Code Provisions.

The test results were used to verify the recommendations and design simplifications of the ACI Building Code Pertaining to flexural strength design and shear strength design, specifically, comments are made on accuracy of strength Predictions.

These results were summarized in Tables (4-12) to (4-15) and the relationship between fire temperature and residual bending moment and shear strength are illustrated in Figures (4-53) to (4-57).

The results are compared with design provisions of ACI 318/2005 code.

4-16-1 Ultimate Bending Moment Capacity.

At burning temperature (400 °C), for NSC and HPC specimens, the ACI Building code gave conservative results to predict bending moment capacity. The ratio between the measured and ACI-predicted values were in the range from (0.99-1.05), (1.02-1.24) for NSC and HPC respectively. In general, using the ACI-equivalent rectangular stress block provides good estimates of the flexural capacity of the HPC beams. These ratios agreed with that obtained by **(Ali and Abul Hasnat, 1986)**, **(Shuaib and Roy, 1991)**, **(Moetaz et al, 1996)**.

At burning temperature (700 °C), for NSC and HPC specimens, the ACI Building Code become unable to predict bending moment capacity. The ratio between the measured and ACI-Predicted values were in the range from (0.48-0.88), (0.67-1.04) for NSC and HPC respectively.

From the results, it is clear that the predicted ultimate bending moment capacity obtained from ACI code provisions is higher than that obtained in the experimental work. This can be attributed to the precracking which happens upon burning and increase in intensity when the fire temperature increases.

4-16-2 Ultimate Shear Strength.

The shear strength prediction of reinforced concrete beams without web reinforcement has been an important piece of information in the design process of concrete beams. Especially, with the development of concrete technology, high performance concrete has been increasingly used in practice. There is, therefore, an urgent need to take effect of fire flame temperature on shear strength of beams made with high-and normal-strength concrete (**Chao and Tsong, 2004**).

At burning temperature (400 °C), for NSC and HPC specimens, the ACI Building code gave conservative results to predict shear strength capacity after exposure to fire flame temperature.

The ratio between the measured and ACI-predicted values were in the range from (0.96-1.18). (1.55-1.81) for NSC and HPC respectively.

At burning temperature (700 °C), for NSC and HPC specimens, the ACI Building Code becomes unable to predict shear strength capacity after exposure to high temperature, the ratio between the measured and ACI-predicted values were in the range of (0.75-0.93) (0.96-1.16) for NSC and HPC respectively.

Table (4-12): Comparison of the flexural test results with that obtained from ACI

318 code for NSC flexure beam specimens

Specimen Identi- fication	Compressive Strength of cube (MPa)	Steel Yield Stress (MPa)	Ultimate Load (kN)	Mu(test) (kN.m)	Percentage Residual Moment capacity (%)	Mu(ACI) (kN.m)	$\frac{\text{Mu (test)}}{\text{Mu (ACI)}}$
NFB	42	540	29.100	4.37	100	3.86	1.13
NF4-1A	35.5	540	26.5	3.98	91	3.78	1.05
NF4-1W	33.5	540	25.8	3.78	86	3.74	1.01
Nf4-1.5A	35	540	25.6	3.84	88	3.76	1.02
NF4-1.5W	31.5	540	24.5	3.675	84	3.7	0.99
NF7-1A	27	432	17.6	2.64	60	2.99	0.88
NF7-1W	19	432	15.4	2.31	53	2.79	0.83
NF7-1.5A	22	432	12.2	1.8	41	2.88	0.63
NF7-1.5W	12	432	7.6	1.14	26	2.38	0.48

Table (4-13): Comparison of the flexural test results with that obtained from ACI

318 code for HPC flexure beam specimens.

Specimen Identi- fication	Compressive Strength of cube (MPa)	Steel Yield Stress (MPa)	Ultimate Load (kN)	Mu(test) (kN.m)	Percentage Residual Moment capacity (%)	Mu(ACI) (kN.m)	$\frac{\text{Mu (test)}}{\text{Mu (ACI)}}$
HPFB	89	540	38.100	5.72	100	4.14	1.38
HPF4-1A	65	540	33.50	5.03	88	4.06	1.24
HPF4-1W	52.5	540	32.20	4.83	84	3.98	1.21
HPF4-1.5A	52.5	540	27.50	4.13	72	3.97	1.04
HPF4-1.5W	45.5	540	26.80	4.02	70	3.93	1.02
HPF7-1A	45	432	22.20	3.33	58	3.21	1.04
HPF7-1W	38	432	19.00	2.85	50	3.16	0.9
HPF7-1.5A	42.5	432	14.600	2.19	38	3.19	0.67
HPF71.5W	35.5	432	14.300	2.15	37.5	3.13	0.67

Table (4-14): Comparison of the shear test results with that obtained from ACI 318 code for NSC deep beam specimens

Specimen Identification	Compressive Strength of cube (MPa)	Compressive Strength of cylinder (MPa)	v_u (test) (MPa)	Percentage Residual shear strength	v_u (ACI) (MPa)	$\frac{v_u \text{ (test)}}{v_u \text{ (ACI)}}$
NSB	42	35.7	1.18	100	1.02	1.17
NS4-1A	35.5	30.18	1.10	93	0.93	1.18
NS4-1W	33.5	28.48	0.92	78	0.91	1.01
NS4-1.5A	35	29.75	0.89	75	0.93	0.96
NS4-1.5W	31.5	26.78	0.92	77	0.88	1.04
NS7-1A	27	22.95	0.68	57	0.81	0.83
NS7-1W	19	16.15	0.64	54	0.68	0.93
NS7-1.5A	22	18.7	0.55	46	0.74	0.75
NS7-1.5W	12	10.2	0.45	38	0.54	0.83

Table (4-15): Comparison of the shear test results with that obtained from ACI 318 code for HPC deep beam specimens

Specimen Identification	Compressive Strength of cube (MPa)	Compressive Strength of cylinder (MPa)	v_u (test) (MPa)	Percentage Residual Shear strength	v_u (ACI) (MPa)	$\frac{v_u \text{ (test)}}{v_u \text{ (ACI)}}$
HPSB	89	80.1	2.28	100	1.52	1.49
HPS4-1A	65	58.5	2.23	98	1.3	1.72
HPS4-1W	52.5	47.25	2.05	90	1.17	1.75
HPS4-1.5A	52	46	2.09	92	1.15	1.81
HPS4-1.5W	45.5	40.95	2.03	89	1.09	1.55
HPS7-1A	45	40.5	1.26	55	1.08	1.16
HPS7-1W	38	34.2	1.15	50	0.99	1.156
HPS7-1.5A	42.5	38.25	1.03	45	1.05	0.98
HPS7-1.5W	35.5	31.95	0.92	40	0.96	0.96

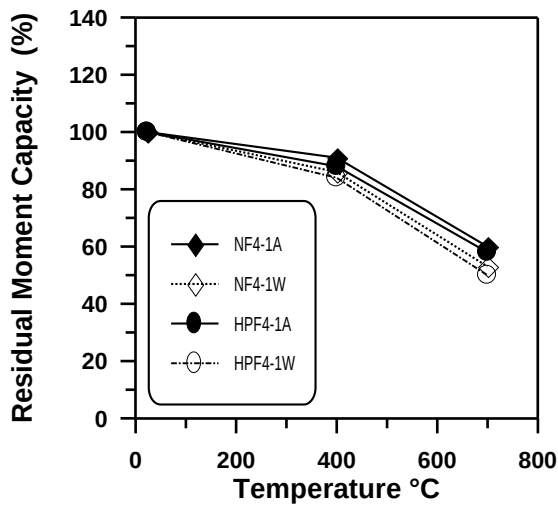


Figure (4-53): Effect of fire temperature on the residual moment capacity of flexure beam specimens for (1) hour period of exposure.

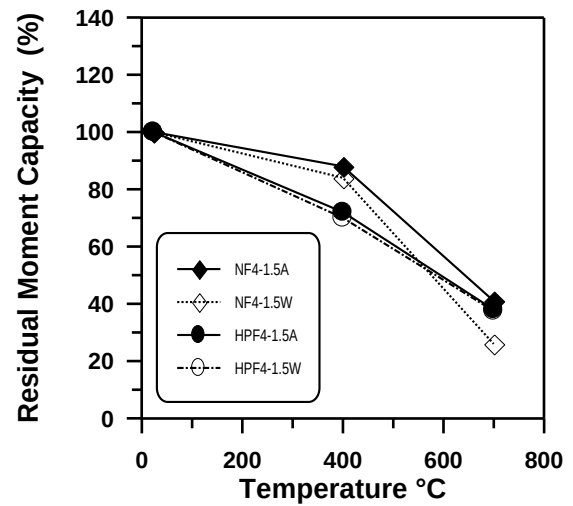


Figure (4-54): Effect of fire temperature on the residual moment capacity of flexure beam specimens for (1.5) hour period of exposure.

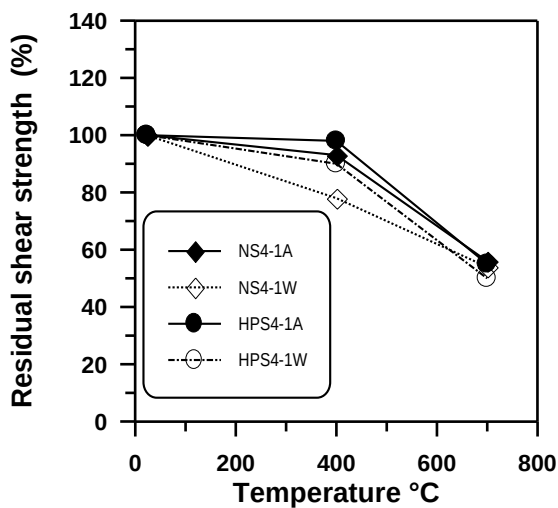


Figure (4-55): Effect of fire temperature on the residual shear strength of deep beam specimens for (1) hour period of exposure

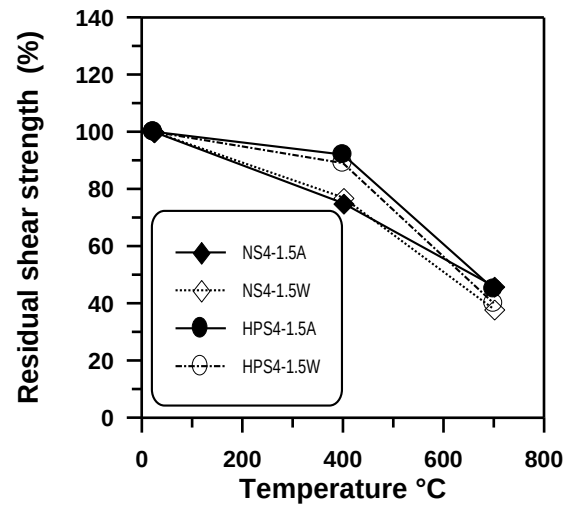


Figure (4-56): Effect of fire temperature on the residual shear strength of deep beam specimens for (1.5) hour period of exposure

Several existing equations are available to predict the shear strength of RC slender beams. Which are selected and used in this study for comparison of the results from experimental work, are outlined in the Table (4-16).

Table. (4-16) Summary of methods for calculating shear strength of beams without web reinforcement.

<i>METHOD</i>	<i>EQUATION</i>	<i>EQ. NO.</i>
ACI.318M-02.Code	$V_c = 0.17 \sqrt{f_c'} b_w d \leq 0.3 \sqrt{f_c'} b_w d$	4-5
Canadian Code 1984	$V_c = 0.2 \sqrt{f_c'} b_w d$	4-6
New Zealand. Code	$V_c = (0.07 + 10 \rho_w) \sqrt{f_c'} b_w d$	4-7
Zsutty. Method	$V_c = 2.2 (f_c' \rho_w d / a)^{1/3} b_w d$	4-8
Rebeiz. Method	$V_c = [0.4 + 1.7 (f_c' \rho_w d / a)^{1/2}] b_w d$	4-9

Table (4-17) and (4-18) present the comparison between the experimental results with (ACI, Canadian, New Zealand) Codes, Zsutty equation and Rebeiz equation. To test equations after exposure to fire flame temperature the relative shear strength values RSSV (V_{test} / V_u calculated) were found for NSC and HPC specimens

At burning temperature (400 °C), for NSC specimens the RSSV were less than (1) (0.96-1.18), (0.82-1), (0.74-0.91), (0.82-1.01) for (ACI, Canadian, Zsutty, Rebeiz) respectively. For HPC specimens the RSSV were greater than (1) for all equations. For NSC it was found that shear capacity drops significantly when $f_c' < 40\text{MPa}$, therefore, the equations

were unconservative to predict shear strength after exposure to (400 °C) fire temperature.

At burning temperature (700 °C), for NSC specimens the RSSV were less than (1) (0.75-0.93), (0.64-0.79), (0.54-0.65), (0.56-0.71), while HPC specimens the RSSV were (0.96-1.16), (0.82-0.99), (0.71-0.94), (0.83-1.05) for (ACI, Canadian, Zsutty, Rebeiz) respectively. The results indicated that (ACI, Canadian, Zsutty, Rebeiz) become unable to predict shear strength after exposure to fire flame, whereas New Zealand code predicts well the shear strength of NSC and HPC beam specimens. Also, the three codes, Zsutty and Rebeiz equation are conservative to predict HPC shear strength of beam specimens after exposure to fire flame. This indicates that the shear strength of HPC has a superior resistance to high temperatures greater than that of NSC with the same steel ratio in spite of the sensitivity of HPC to high temperature from the material point of view.

This finding is justifiable as the splitting tensile strength, flexural strength and modulus of elasticity of concrete subjected to fire flame are more severely affected. Hence, flexural negatively affected by exposure to fire flame because they depend on the above strengths rather than on compressive strength. This may explain why the three codes (ACI, Canadian, New Zealand), Zsutty and Rebeiz equations predicts higher values of flexural and shear strengths than the experimental results as these equations depend on compressive strength to predict flexural and shear strength.

Table (4-17): Comparison of the test results with (ACI, Canadian, New Zealand) codes, Zsutty and Rebeiz values for NSC shear beam specimens

Specimen Identi- fication	Compressive Strength of cylinder (MPa)	$\frac{V_c \text{ (test)}}{V_c \text{ (ACI)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (Canad)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (New Z)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (Zsutty)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (Rebeiz)}}$
NSB	35.7	1.17	0.99	1.8	0.93	1.03
NS4-1A	30.18	1.18	1.00	1.82	0.91	1.01
NS4-1W	28.48	1.01	0.86	1.57	0.77	0.86
NS4-1.5A	29.75	0.96	0.82	1.48	0.74	0.82
NS4-1.5W	26.78	1.04	0.86	1.53	0.79	0.87
NS7-1A	22.95	0.83	0.7	1.28	0.61	0.67
NS7-1W	16.15	0.93	0.79	1.44	0.65	0.71
NS7-1.5A	18.7	0.75	0.64	1.16	0.54	0.58
NS7-1.5W	10.2	0.83	0.7	1.29	0.54	0.56

Table (4-18): Comparison of the test results with (ACI, Canadian, New Zealand) codes, Zsutty and Rebeiz values for HPC shear beam specimens

Specimen Identi- fication	Compressive Strength of cylinder (MPa)	$\frac{V_c \text{ (test)}}{V_c \text{ (ACI)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (Canad)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (New Z)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (Zsutty)}}$	$\frac{V_c \text{ (test)}}{V_c \text{ (Rebeiz)}}$
HPSB	80.1	1.49	1.27	2.32	1.36	1.49
HPS4-1A	58.5	1.72	1.46	2.65	1.48	1.64
HPS4-1W	47.25	1.75	1.93	2.71	1.46	1.62
HPS4-1.5A	46	1.81	1.54	2.8	1.50	1.67
HPS4-1.5W	40.95	1.55	1.58	2.88	1.52	1.68
HPS7-1A	40.5	1.16	0.99	1.79	0.94	1.05
HPS7-1W	34.2	1.156	0.98	1.79	0.91	1.01
HPS7-1.5A	38.25	0.98	0.83	1.51	0.71	0.87
HPS7-1.5W	31.95	0.96	0.82	1.48	0.75	0.83

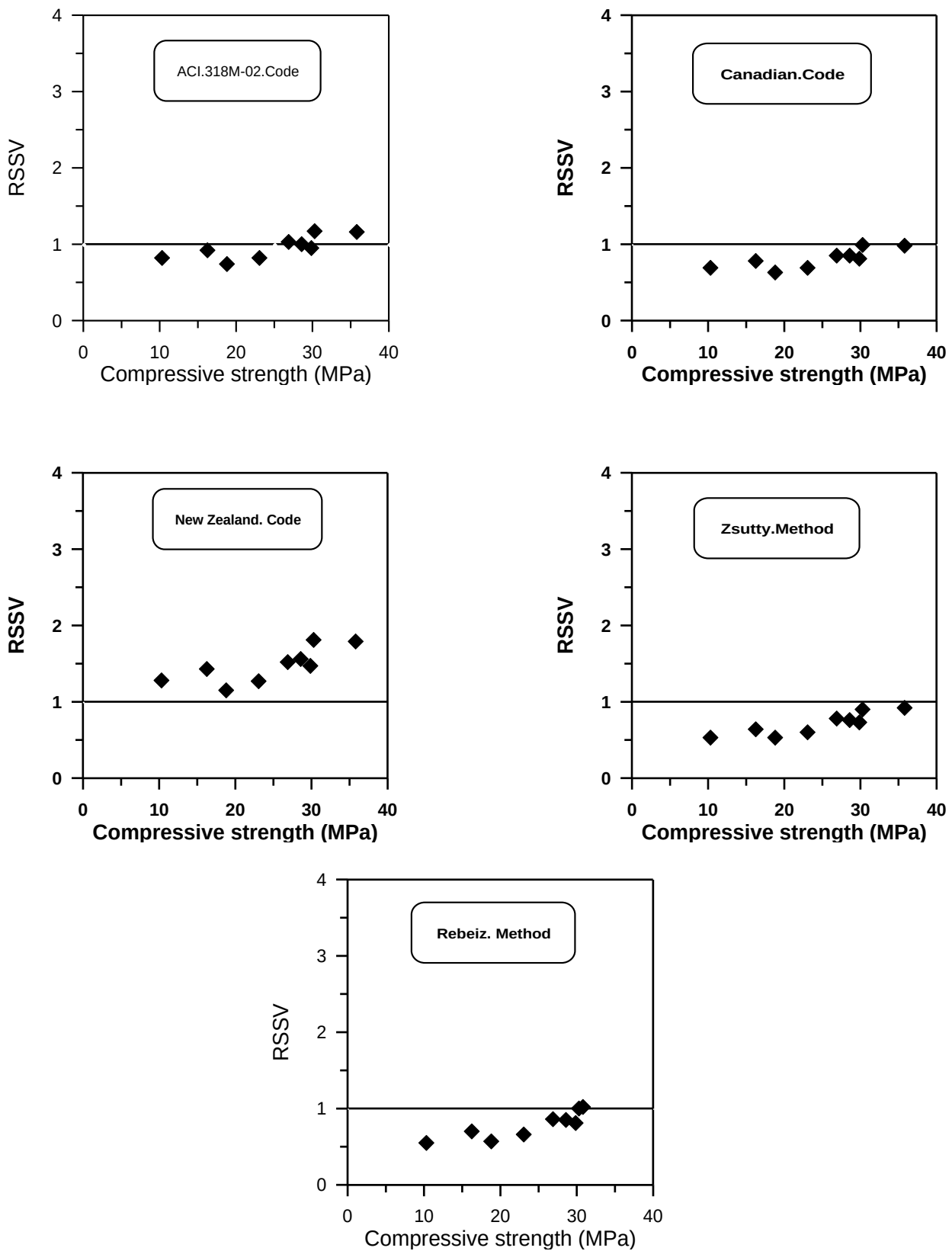


Figure (4-57): Compressive strength after exposure to fire flame temperature versus relative shear strength values RSSV for NSC.

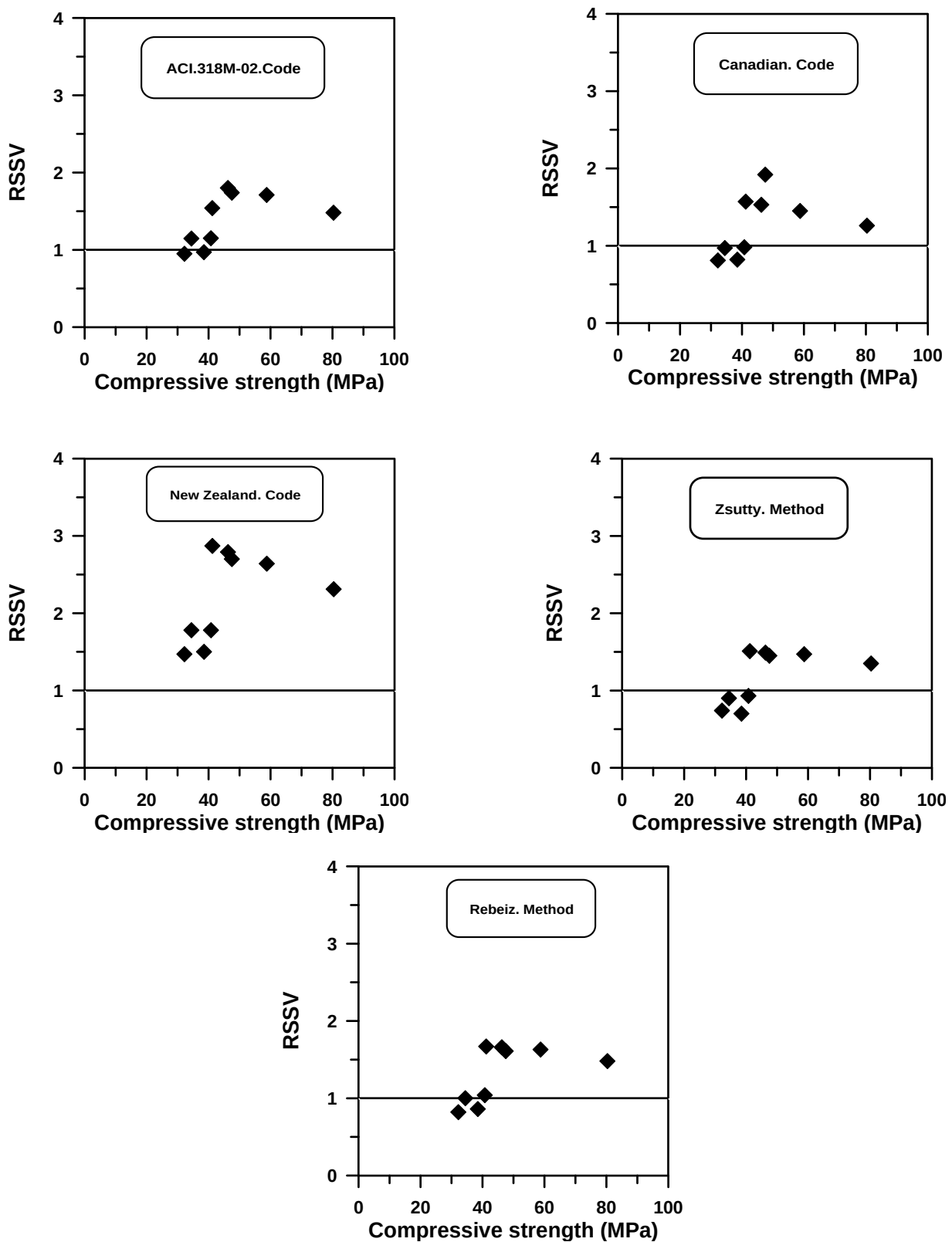


Figure (4-58): Compressive strength after exposure to fire flame temperature versus relative strength values RSSV for HPC.

4.17 Cracking Pattern

When the applied load progresses, cracks initiate, propagate and can be detected at early loading stages. The cracks were marked with a red marking pen, then photographs were taken to the crack pattern.

4-17-1 Flexural Beam Specimens :

Specimens (NFB), (HPFB) were a control specimens before exposure to fire flame.

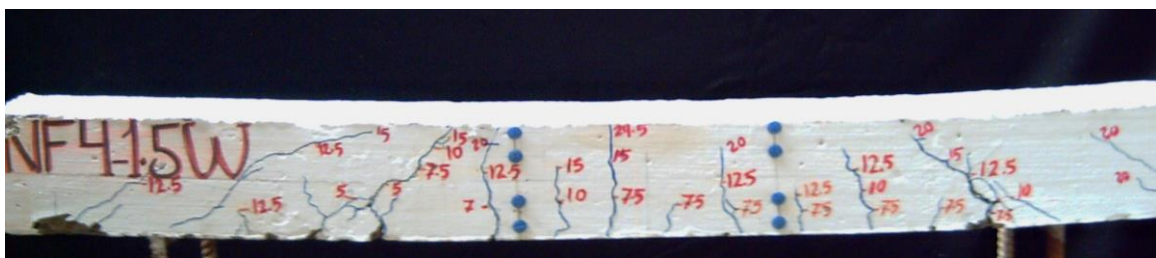
The beams were loaded gradually until the first cracks take place early in the test for both NSC and HPC depending on the temperature level. The flexural cracks appear initially in the constant moment region. Further flexural cracks are formed progressively and widened as the loading increased. For specimen burnt at (400 °C) flexural cracks were wide speared along the beam outside the pure bending moment region. However inclined cracks are formed due to the presence of increasing shear stresses as the load and temperature increase. For specimen burnt at (700 °C), additional vertical cracks appeared on the beam surface, followed by formation of diagonal cracks, the failure began outside the mid span of beams. Failure of the test specimens occurred by yielding of steel reinforcement followed by crushing of the concrete near the supports. The cracking patterns of the tested beams at failure are shown in Plate (4-2) .

4-17-2 Deep Beam Specimens:

Specimens (NSB), (HPSB) were control deep beam specimens without subjecting to fire flame.

When the load was increased, cracks appeared on the beam surface near the supports followed by the formation of diagonal cracks. A diagonal tension crack is generally defined as an inclined crack in the shear span that extends from the tensile reinforcement towards the nearer concentrated load that intersects the tensile reinforcement at an angle of approximately

The typical cracking patterns of the beams are shown in Plate (4-3).



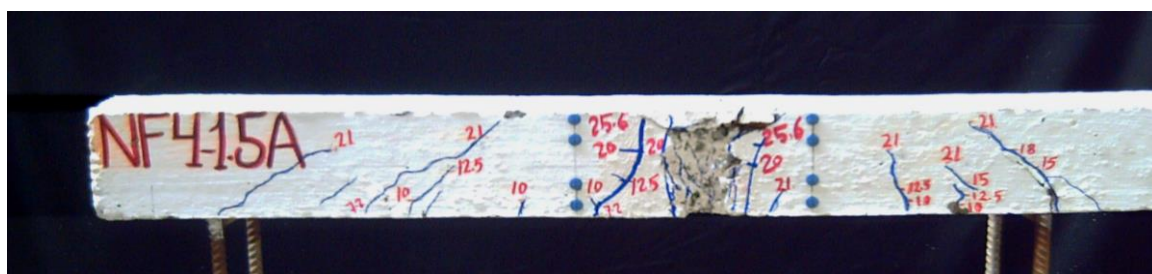
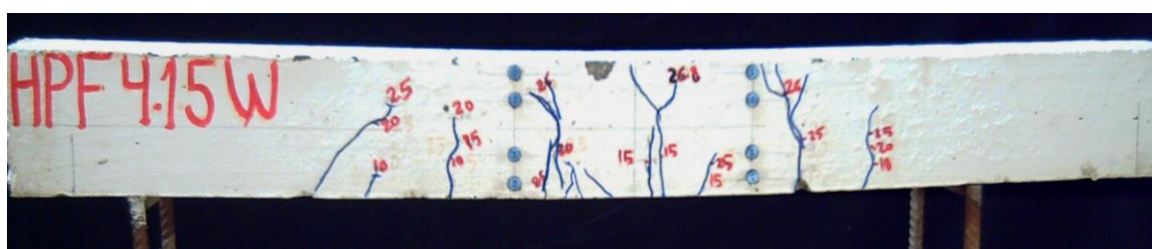
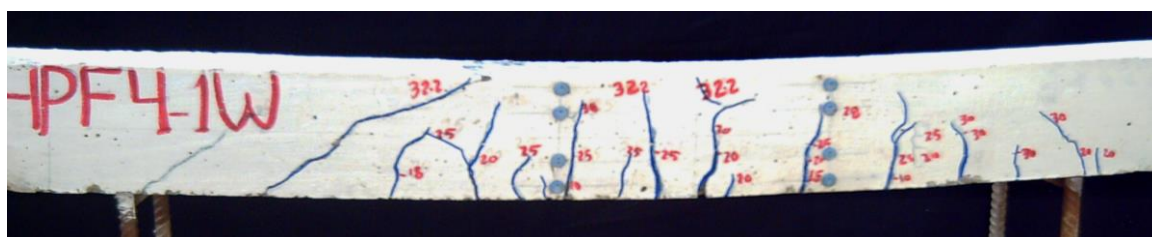
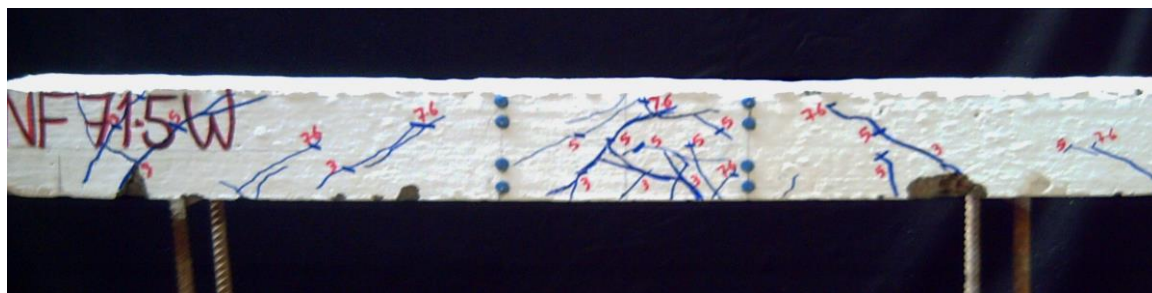
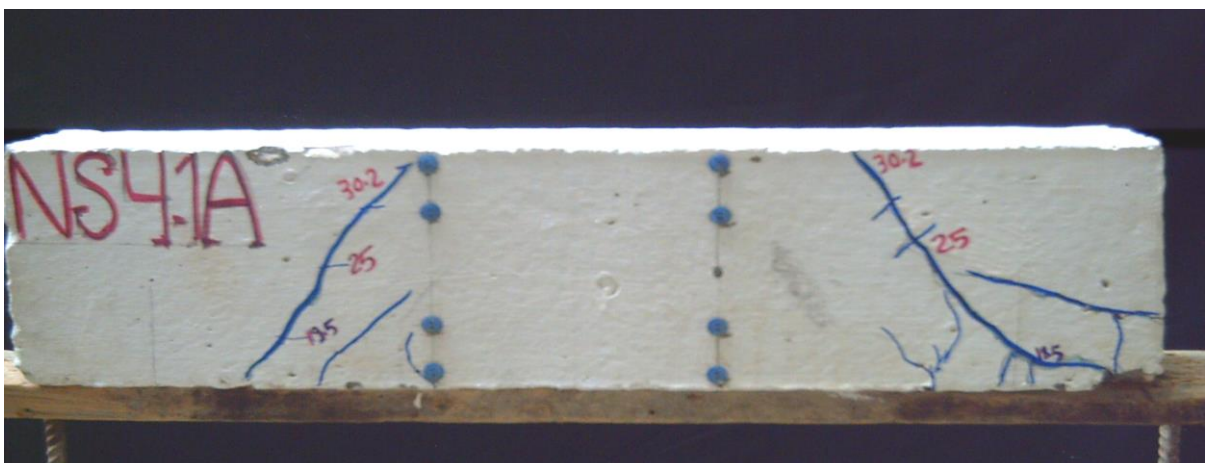
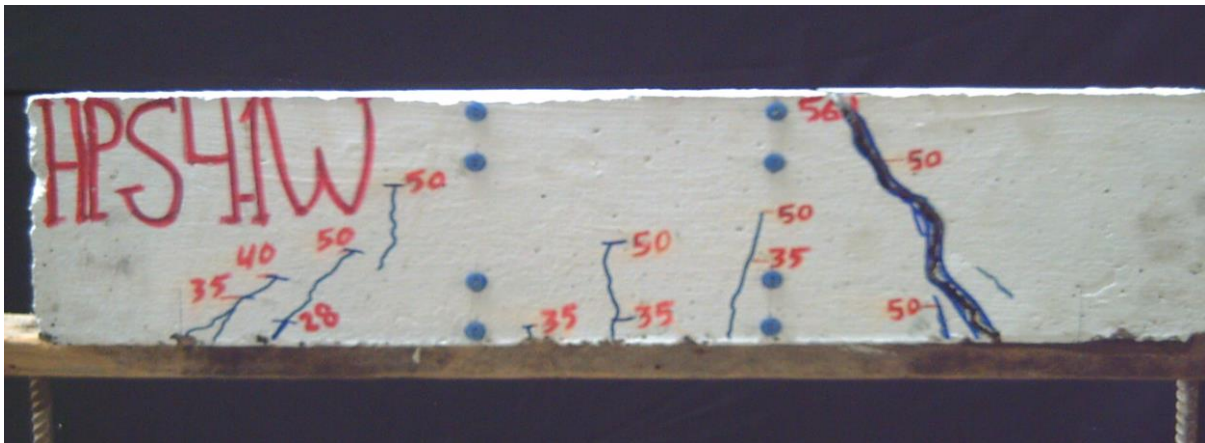
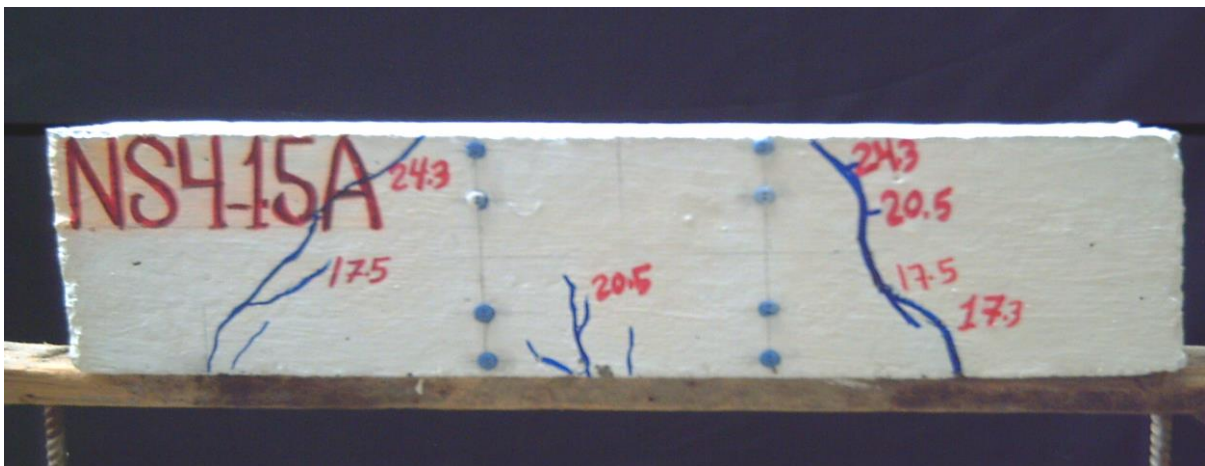


Plate (4-2) : Some of flexural beam specimens after failure



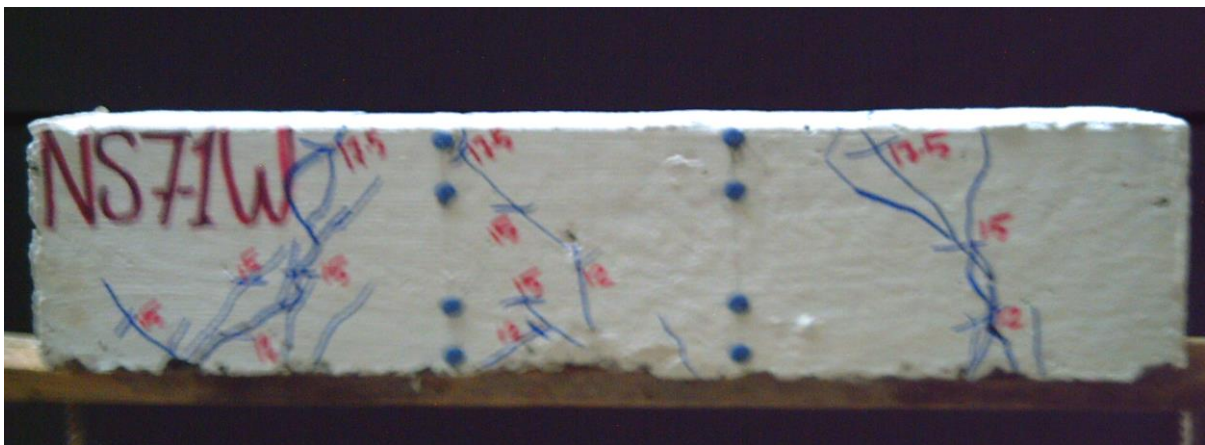
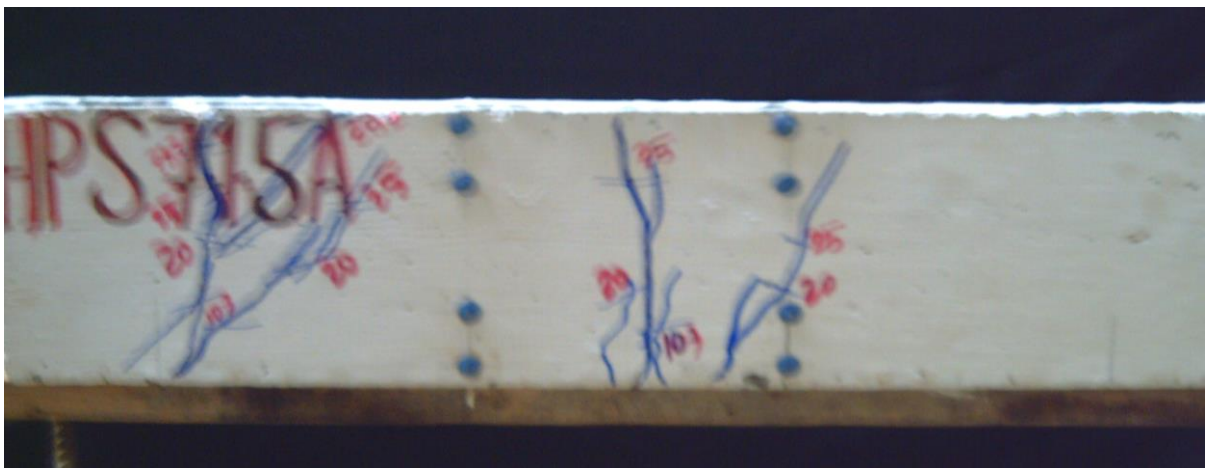
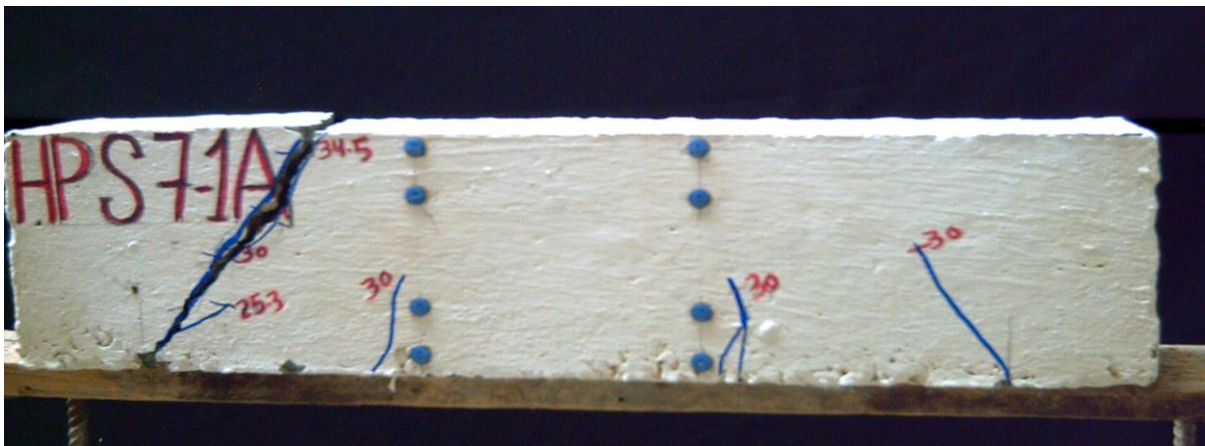
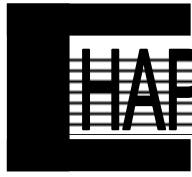


Plate (4-3) : Some of deep beam specimens after failure



CONCLUSIONS AND FUTURE WORK

5.1 Conclusions

Based on the test results of the present study, the following conclusions can be drawn.

1- The compressive strength of concrete decreases with fire flame temperature by an amount depending on the exposure temperature.

◀ The percentage residual in compressive strength for HPC after exposure to (400°C and 700°C) is about (51-73%) and (40-50%) respectively.

◀ The percentage residual in compressive strength for NSC after exposure to (400°C and 700°C) is about (77-84%) and (29-65%) respectively.

2- The splitting tensile strength is more sensitive to fire flame exposure than the compressive strength. The percentage residual for HPC at (400 °C and 700 °C) is about (61-78%) and (37-40%) respectively, while the percentage residual for NSC at (400 °C and 700 °C) is about (50-80%) and (41-51%) respectively.

3- The flexural strength is more sensitive to fire flame temperatures than compressive strength and splitting tensile strength. The percentage residual flexural strength for HPC after exposure to (400 °C and 700 °C) is about

(34-40%) and (13-21%) respectively, while the percentage residual flexural strength for NSC at (400 °C and 700 °C) is about (37-42%) and (16-27%) respectively.

4- Modulus of elasticity of concrete is the most affected factor by fire flame rather than compressive strength. The percentage residual modulus of elasticity for HPC after exposure to fire temperature (400 °C and 700 °C) is about (41-47%) and (15-18%) respectively, while the percentage residual for NSC at (400°C and 700°C) is about (59-71%) and (20-27%) respectively.

5- For HPC beam specimens designed to fail in flexure. The first crack load and ultimate load decrease when subjected to fire flame.

◀ After exposure to fire temperature (400 °C), the percentage residual first crack load and ultimate load are (69-77%), (85-88%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (57-61%), (70-71%) respectively.

◀ After exposure to fire temperature (700 °C), the percentage residual first crack load and ultimate load are (27-53%), (50-58%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (21-30%), (37-38%) respectively.

6- For NSC beam specimens designed to fail in flexure. The first crack load and ultimate load decrease when subjected to fire flame.

◀ After exposure to fire temperature (400 °C), the percentage residual first crack load and ultimate load are (68-88%), (87-91%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (59-85%), (84-88%) respectively.

◀ After exposure to fire temperature (700 °C), the percentage residual first crack load and ultimate load are (47-55%), (52-61%) for (1) hour

exposure period respectively, while for (1.5) hour exposure period are (35-41%), (26-42%) respectively.

7- For HPC beam specimens designed to fail in shear, it was found that:

◀ After exposure to fire temperature (400 °C), the percentage residual first crack load and ultimate load are (82-100%), (90-98%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (69-76%), (88-92%) respectively.

◀ After exposure to fire temperature (700 °C), the percentage residual first crack load and ultimate load are (71-72%), (50-55%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (26-29%), (40-45%) respectively.

8- For NSC beam specimens designed to fail in shear, it was found that:

◀ After exposure to fire temperature (400 °C), the percentage residual first crack load and ultimate load are (89-99%), (78-93%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (91-89%), (75-77%) respectively.

◀ After exposure to fire temperature (700 °C), the percentage residual first crack load and ultimate load are (63%), (51-57%) for (1) hour exposure period respectively, while for (1.5) hour exposure period are (42-43%), (38-47%) respectively.

9- The bending moment capacity of HPC specimens is more sensitive to fire flame exposure than that of NSC specimens.

◀ After exposure to fire temperature (400 °C), for HPC the residual bending moment capacity is (84-88%), (70-72%), as for NSC (86-91%), (84-88%) for (1.0) and (1.5) hour exposure periods respectively.

◀ After exposure to fire temperature (700 °C), for HPC the residual bending moment capacity is (50-58%), (37-38%), as for NSC (53-60%), (26-41%) for (1.0) and (1.5) hour exposure periods respectively.

10- The shear strength capacity of HPC specimens is lesser negatively affected by fire flame temperature than NSC as follows:

◀ After exposure to fire temperature (400 °C), for HPC the residual shear strength capacity is (90-98%), (78-93%), as for NSC (78-93%), (75-77%) for (1.0) and (1.5) hour exposure periods respectively.

◀ After exposure to fire temperature (700 °C), for HPC the residual shear strength capacity is (50-55%), (40-45%), as for NSC (54-57%), (38-46%) for (1.0) and (1.5) hour exposure periods respectively.

11- Crack width in deep beams that are subjected to fire flame are higher than the beams that are not subjected to fire.

12- With regard to the residual ultimate load bearing capacity, the shear strength for NSC beam specimens is found more sensitive to fire flame temperature than HPC, whereas, bending moment capacity for HPC is more sensitive to fire flame exposure than NSC.

13- The maximum mid span deflection for the beam specimens designed to fail in flexure differs randomly with fire flame temperature increase. For NSC the maximum mid span deflection increases when fire flame temperature increases, but the reverse happens in HPC. Where max mid span deflection is higher for NSC beams compared with HPC.

14- Based on the available experimental results in this research work concerning the bending moment capacity and shear strength capacity for shallow and deep beams it can be concluded that:

◀ After exposure to fire temperature (400 °C), for HPC and NSC, the ACI 318/ 2002-code can be safely used to predict the residual bending moment capacity and shear strength capacity from the residual compressive strength after exposure to fire flame.

◀ After exposure to fire temperature (700 °C), for HPC and NSC, the ACI-Code become unable to predict bending moment capacity and shear strength capacity.

15- The ratio between the measured and ACI-predicted values for flexural strength was in the range (0.99-1.05), (1.02-1.21) for NSC and HPC respectively at temperature (400 °C), while the values was (0.48-0.88), (0.67-1.04) at temperature (700 °C).

16- The ratio between the measured and ACI-predicted values for shear strength was in the range (0.96-1.18), (1.55-1.81) for NSC and HPC respectively at temperature (400 °C), while the values was (0.75-0.93), (0.96-1.16) at temperature (700 °C).

17- For comparison purposes, the values of shear strength compared with three codes (ACI, Canadian, New Zealand), Zsutty and Rebeiz research, based on RSSV (Relative Shear Strength Value) when the beam specimens are exposed to fire flame as follows.

◀ At burning temperature (400 °C), for HPC beams, the RSSV is greater than (1) for all equations, while for NSC beams the RSSV is less than (1) which was (0.96-1.18), (0.82-1), (0.74-0.91), (0.82-1.01) for (ACI, Canadian, Zsutty, Rebeiz) respectively.

◀ At burning temperature (700 °C), for HPC beams, the RSSV was (0.96-1.16), (0.82-0.99), (0.71-0.94), (0.83-1.05), while for NSC beams was (0.75-0.93), (0.64-0.79), (0.54-0.65), (0.56-0.71) for (ACI, Canadian, Zsutty, Rebeiz) respectively.

18- The New Zealand Code predicts shear strength after exposure to (400°C and 700°C) fire flame temperature conservatively for HPC and NSC.

19- The three codes (ACI, Canadian, New Zealand), Zsutty and Rebeiz equation gave conservative results to predict shear strength for HPC after exposure to 400°C fire flame temperature.

20- After exposure to fire flame, the cracks in flexural failure speared along the beam until outside the pure bending moment region, while the diagonal cracks in shear failure became wider. Unlike the flexural cracks which formed gradually, the diagonal cracks formed suddenly.

21- At burning temperature (400 °C), for both NSC and HPC, the strain corresponding to the peak load increased slightly. However, at temperature (700 °C), the strain at peak load increased significantly.

5.2 Recommendations for Further Studies

- 1-** The effect of fire flame exposure on punching shear for NSC and HPC.
- 2-** The effect of fire flame exposure on flexural behavior and shear strength for light weight concrete.
- 3-** The effect of the steel reinforcement cover on the strength and behavior of deep beams and flexural failure after exposure to fire flame temperature.
- 4-** The effect of fire flame on deep beam specimens with stirrups and different shear span ratio (a/d).
- 5-** Studying the effect of fire flame temperature on the behavior of HPC slab, column, and frame system.

Acknowledgement

Praise be to ALLAH, most gracious, most merciful , who gave me the ability and desire to complete this research work.

First, I would like to express my sincere gratitude to my supervisors; Asst. Prof. Dr. Mahdi S.Essa and Ass.Prof.Mr.Samir A.Al-Mashhedi for their remarkable suggestions, encouragement and guidance through the research; I am really indebted to them.

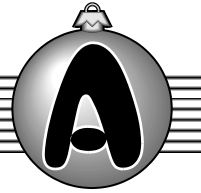
Thanks to all staff of Civil Engineering Department /College of Engineering /University of Babylon for their appreciable support. Special thanks go to Mohammed Jassam and Abbas Kadhim for their help in carrying out the experimental tests.

Thank also to the staff of construction material laboratory at Babylon Technical Institute for their help in using the various facilities.

Finally, to my family, I express my deepest appreciation for their love, help, understanding, and support.



APPENDIX



A-1 Flexural Analysis for NSC:

Section of Beam

$width = b = 100mm, depth = h = 100mm$

Reinforcement

Two $\phi 8mm$ $f_y = 540$ and stirrups $\phi 6mm$ $f_y = 650$.

Calculations

By the following calculations the moment capacity of the section is found:

$$A_s = \text{Area of flexural steel} = 2(8)^2 * \frac{\pi}{4} = 100.5mm^2$$

$$f_y = 540MPa, f_c = 25.5MPa$$

$$d = 80mm, bw = 100mm$$

$$\rho = \frac{A_s}{b_w d} = \frac{100.5}{100 * 80} = 0.0126$$

$$\rho_b = \beta_1 \frac{0.85 f_c}{f_y} \frac{600}{600 + f_y}, \beta_1 = 0.85$$

$$\rho_b = 0.02, \rho_{max} = 0.75 * \rho_b = 0.0156$$

$$\rho_{min} = \frac{1.4}{f_y} = \frac{1.4}{540} = 0.0026$$

$$\rho_{min} < \rho < \rho_{max}$$

$$\therefore \rho < \rho_{max}$$

Section is flexurally adequate. $\therefore$

Ultimate moment capacity-Mu-can be found according to the following ACI-318/2005code formula

$$Mu = A_s f_y d (1 - 0.59 \rho \frac{f_y}{f_c}).$$

$$Mu = 100.5 * 540 * 80 (1 - 0.59 * 0.0126 * \frac{540}{25.5})$$

$$Mu = 3.66 kN.m$$

$$(Total\ load) 0.15P = 3.66 \Rightarrow P = 24.4 kN$$

" Check Shear"

$$V_c = 0.17 \sqrt{f_c^-} * b_w d = 0.17 \sqrt{25.5} * 100 * 80$$

$$V_c = 6.87 kN$$

$$V_s = \frac{d}{s} A_v f_y, A_v = z \frac{\pi}{4} (36) = 56.55 mm^2$$

$$V_s = \frac{80}{100} * 56.55 * 650 = 29.4 kN$$

$$V_n = V_c + V_s = 6.87 + 29.4 = 36.27 kN$$

$$V_u = 0.75 * 36.27 = 27.2 kN$$

$$0.5p = V_c \Rightarrow p = 54.4 kN$$

Max shear force

Shear failure load > bending failure load.:

Bending failure controls.:

A-2 Flexural Analysis for HPC:

$$f_y = 540, f_c^- = 70.2 MPa$$

$$\rho = 0.0126, \rho_b = \beta_1 \frac{0.85 f_c^-}{f_y} \frac{600}{600 + f_y}$$

$$\beta_1 = 0.65$$

$$\rho_b = 0.65 \frac{0.85 * 70.2}{540} \frac{600}{600 + 540}$$

$$\rho_b = 0.038, \rho_{max} = 0.028$$

$$\rho_{min} = \frac{1.4}{f_y} = \frac{1.4}{540} = 0.0026$$

$$\rho_{min} < \rho < \rho_{max}$$

Section is flexurally adequate..:

$$Mu = 100.5 * 540 * 80 (1 - 0.59 * 0.0126 * \frac{540}{70.2})$$

$$Mu = 4.1 kN.m$$

$$0.15P = 4.1 \Rightarrow P = 27.3 kN (Total load)$$

(Check Shear)

$$V_c = 0.17 \sqrt{f_c} * b_w d$$

$$V_c = 0.17 \sqrt{70.2} * 100 * 80$$

$$V_c = 11.4 kN$$

$$V_s = \frac{80}{100} * 56.55 * 650$$

$$V_s = 29.4 kN$$

$$V_n = V_c + V_s = 11.4 + 29.4 = 40.8 kN$$

$$V_u = 0.75 * 40.8 = 30.6 kN$$

$$Total load = P = 61.2 kN$$

Shear failure load > bending failure load.:

Bending failure controls.:

A-3 Shear Analysis for NSC Deep Beam Specimens :

$$f_y = 650 MPa, f_c = 25.5 MPa$$

$$b_w = 100 mm, d = 137 mm$$

$$V_c = 0.17 \sqrt{f_c} * b_w d$$

$$= 0.17 \sqrt{25.5} * 100 * 137$$

$$V_c = 11.76 kN$$

$$V_n = V_c, V_u = \phi V_n$$

$$V_u = 0.75 * 11.76 = 8.82 kN$$

$$\frac{P}{2} = Vu \Rightarrow p = 2 * 8.82$$

$$p = 17.64 kN$$

(Check Flexure).

$$A_s = z \frac{\pi}{4} (36) = 56.55 mm^2$$

$$\rho = \frac{A_s}{bd} = \frac{56.55}{137 * 100} = 0.004$$

$$\rho_b = \beta_1 \frac{0.85 f_c^-}{f_y} \frac{600}{600 + f_y}, \beta_1 = 0.85$$

$$\rho_{\max} = 0.011$$

$$\rho_{\min} = \frac{1.4}{650} = 0.00154$$

$$0.00154 < \rho_{act} < 0.011$$

$$Mu = \phi A_s d f_y (1 - 0.59 \rho \frac{f_y}{f_c^-})$$

$$Mu = 4.1, Total Load = p = 82 kN$$

bending failure load $\therefore$ < Shear failure load

Shear failure controls $\therefore$

A-4 Shear Analysis for HPC Deep Beam Specimens:

$$f_{yb} = 650 MPa, f_{cb} = 70.2 MPa$$

$$b_w = 100 mm, d = 137 mm$$

$$V_c = 0.17 \sqrt{70.2} * 100 * 137 = 19.5 kN$$

$$V_u = 0.75 * 19.5 = 14.63 kN$$

$$\frac{P}{2} = Vu \Rightarrow \rho = 29.26 kN$$

(Check Flexure).

$$\beta_1 = 0.65, \rho_b = \beta_1 \frac{0.85 f_c^-}{f_y} \frac{600}{600 + f_y}$$

$$\rho_b = 0.028$$

$$\rho_{\max} = 0.75 * 0.028 = 0.02$$

$$\rho_{\min} = \frac{1.4}{f_y} = \frac{1.4}{650} = 0.0015$$

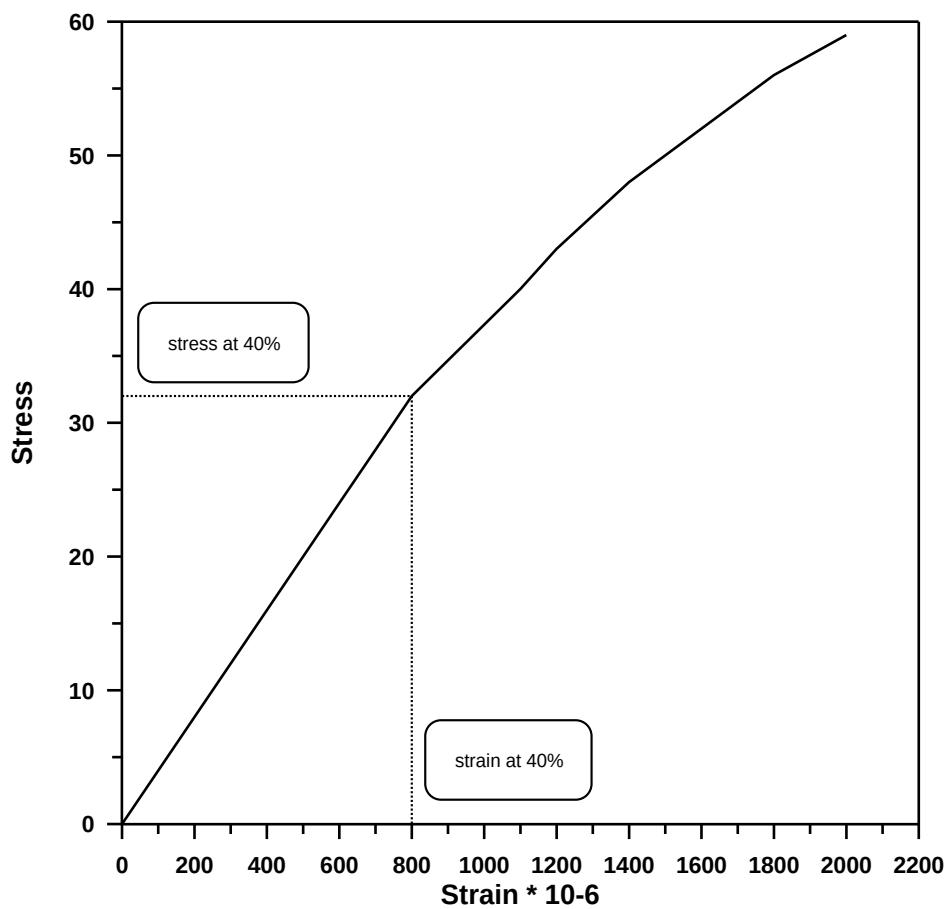
$$\rho_{\min} < \rho < \rho_{\max}$$

$$Mu = 4.53kN, Total p = 90.6kN$$

bending failure load $\therefore$ < Shear failure load

Shear failure controls $\therefore$

A-5 Method calculation of modulus of elasticity for HPC



(Example of stress – strain relation in compression for HPC)

$$E_c = \frac{0.032 - 0.001}{0.008 - 0.00005} = 41.33Gpa$$

CERTIFICATION

We certify that we have read this thesis, titled " ***Effect of Fire Flame Exposure on Flexural Behavior and Shear Strength of Reinforced NSC and HPC Beams*** ", and as an examining committee examined the student " **Awad Ali Saqier** " in its content and in what is connected with it, and that in our opinion it meet the standard of thesis for the degree of Master of Science in Civil Engineering (***Construction Materials***).

Signature

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Date : / / 2009

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Date : / / 2009

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Head of the Civil Engineering Department.

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Certification

We certify that this thesis titled “***Effect of Fire Flame Exposure on Flexural Behavior and Shear Strength of Reinforced NSC and HPC Beams***”, was prepared by (Awad Ali Saqier) under our supervision at Babylon University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

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NOTATIONS

Most commonly symbols used are listed below. These and others are defined where they appear in the research.

Symbol	Description
a	Shear span, distance between concentrated load and face of support
a/d	Shear span to depth ratio
A_s	Area of tension Reinforcement
b_w	Width of the beam
d	Effective depth of the beam
E_s	Modulus of elasticity of the steel.
E_{ca}	Modulus of elasticity of the concrete after exposure to fire flame temperature.
E_{cb}	Modulus of elasticity of the concrete before exposure to fire flame temperature
f_{cua}	Compressive strength of concrete after exposure to fire flame temperature.
f_{cub}	Compressive strength of concrete before exposure to fire flame temperature.
f_{ra}	Flexural strength after exposure to fire flame temperature.
f_{rb}	Flexural strength before exposure to fire flame temperature.
f_{sa}	Splitting tensile strength after exposure to fire flame temperature.
f_{sb}	Splitting tensile strength before exposure to fire flame temperature.
f_y	Yield stress of steel reinforcing bar.
f_u	Ultimate strength of steel reinforcing bar.
f_{cu}	Compressive strength of concrete based on B.S Specifications.
f'_c	Compressive strength of concrete based on ASTM Specifications.
h	Depth of the beam.
HPC	High performance concrete.
HSC	High strength concrete
HPF4-1A	High performance reinforced concrete flexural beam specimen subjected to 400 °C fire temperature for (1) hour exposure period and cooled by air.
HPS4-1.5W	High performance reinforced concrete shear beam specimen subjected to 400°C fire temperature for 1.5 hour exposure period and cooled by water.
HPF7-1A	High performance reinforced concrete flexural beam specimen subjected to 700 °C fire temperature for (1) hour exposure period and cooled by air.

Symbol	Description
HPS7-1.5W	High performance reinforced concrete shear beam specimen subjected to 700 °C fire temperature for 1.5 hour exposure period and cooled by water.
I_n/d	Clear span to effective depth ratio.
M_u	Ultimate moment at the section.
M_f	Ultimate moment of resistance with shear.
M_n	moment of resistance without shear.
NSC	Normal strength concrete
NF4-1A	Normal reinforced concrete flexural beam specimen subjected to 400 °C fire temperature for (1) hour exposure period and cooled by air.
NS4-1.5W	Normal reinforced concrete shear beam specimen subjected to 400 °C fire temperature for (1.5) hour exposure period and cooled by water.
NF7-1A	Normal reinforced concrete flexural beam specimen subjected to 700 °C fire temperature for (1) hour exposure period and cooled by air.
NS7-1.5W	Normal reinforced concrete shear beam specimen subjected to 700 °C fire temperature for (1.5) hour exposure period and cooled by water.
RSSV	Relative Shear Strength Value
S	Spacing of vertical and inclined shear reinforcement ,mm
SF	Silica fume.
V	Shear strength, kN
V_u	Ultimate shear strength of reinforced concrete beams,kN
V_{test}	Test shear strength kN
w/c	Water to cement ratio.
ρ_w	Reinforcement ratio of main steel equal to $(A_s/b_w d)$.
ρ_v	Reinforcement ratio of vertical stirrups steel equal to $(A_v/b_w S)$.

 **ACI-ASCE Committee 426, 1973**, "The Shear Strength of Reinforced Concrete Members" Journal of the Structural Division, Proceedings of ASCE, Vol .99, No. ST6, June, pp.1091-1187.

 ACI Committee 318, "Building Code Requirements for Structural Concrete and Commentary (**ACI 318M-02/ACI 318 RM-05**)", **2005**, American Concrete Institute, Detroit, pp109-157.

 **ACI committee 363, 1997**, "State-of-the-Art Report on High Strength Concrete", American Concrete Institute, Detroit, Michigan, pp. 1-55.

 **Aitcin, P.C., March 1980**, "How to Produce HSC", Concrete Construction, Vol.25,No3, pp. 222-236.

 **Alberto Meda, Pietro G, Gambarora, and Marco Bonomi, 2002**, "High-Performance Concrete in Fire-Exposed Reinforced Concrete Sections "ACI Structural Journal", May- June, pp277-287.

 **Al-Hassani, A., Ramadani, K.E., and Xue, H.Y., October 2002**, " Properties of HSC at Different Ages", the 6th International Conference of Concrete Technology for Developing Countries, Amman-Jordan,.pp.21-24.(Cited by Al-Hayderi,2003).

 **Al-Hayderi, Hawraa. Saeed., December, 2003**, " Correlation Between Strength of Different Size Shapes and Curing Condition for High Strength Concrete " M.Sc. Thesis College of Engineering, Department of Civil Engineering, Al-Mustansiria University, Baghdad, Iraq, 101 pp.

 **Ali Nadjai, Didier Talamon, Faris Ali, 2005**, "Fire Performance of Concrete Beams Reinforced with FRP Bars", University of Ulster, International Institution for FRP in Construction.

 **AL-Musawi, J-M.S., 1989** "Shear Capacity of High Strength Concrete Beams with Web Reinforcement", M.SC, Thesis, University of Technology, Baghdad, Iraq, 121 pp (in Arabic).

 **Asa'ad, B. Muhammad., 1987** "Response of Reinforced Concrete Subject to Fire ", M.Sc. thesis, University of Technology, Baghdad.

 **Ashraf H. Elzanaty, Arthur H. Nilson, and Floyd O. Slate, 1986**, "Shear Capacity of Reinforced Concrete Beams Using High-Strength Concrete", ACI Structural Journal, Vol. 83, No31, March-April, PP.295-290.

 **ASTM (C 496 - 86), 1989**, " Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens", American Society for Testing and Materials, Annual Book of ASTM Standards, V. 04. 02, PP. 32-34.

 **ASTM (C 469-87a), 1984**, "Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression ", American Society for Testing and Materials.

 **ASTM C 1240 – 2003**, "Standard Specification for Use of Silica Fume as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout", Vol. 04.02, 2003, pp.1-6.

 **ASTM C 494/C 494M – 1999a**, "Standard Specification for Chemical Admixtures for Concrete", Vol. 04.02, 1999, pp.1-9.

 **ASTM Designation C78-84, 1989**, "Flexural Strength of Concrete Using Simple Beam with-Third Point Loading", Annual Book of ASTM, Materials, Philadelphia, Pennsylvania, Vol.04-02, pp.32-34.

 **ASTM E05.11 Task Group. 1982**, "Repeatability and Reproducibility of Results of ASTM E 119 Fire Test", Research Report No. RR: E5-1003, ASTM, Philadelphia.

 **Aziz, O.Q., 1997**, "Shear Strength Prediction of Crushed Stone Reinforced Concrete Deep Beams", Ph.D. University of Technology , Baghdad, May, 195 pp.

 **Baalbaki, W., Benmokrane, B., Chaallal, O., and Aitcin, P.C., 1991**, "Influence of Coarse Aggregate on Elastic properties of High-performance Concrete", ACI Materials Journal, No. 88-M54, September-October, pp-499-503.

 **BS 1881:part 121, 1983**, "Method for Determination of the Static Modulus of Elasticity in Compression".

 **BS.5328, part 2, 1990**, "Guide to Specifying Concrete Mixes", British Standards Institution.

 **BS.1881, part 116, 1989**, "Method for Determination of Compressive Strength of Concrete Cubes", British Standards Institution.

 **BS 5328, part 4, 1990**, "Specification for the Procedures to be Used in Sampling, Testing and Assessing Compliance of Concrete".

 **BS 8110, Part 1, 1997**, "Code of Practice for Design and Construction ", British standards Institution, London, 1997.

 **Bunni, Z.K., 1994**, "The Effect of the Shear Span to Depth Ratio on Shear Strength of the High-Strength Concrete Beams" M.Sc. Thesis University of Technology , Baghdad, 120 pp.

 **Canadian Standards Association, 1984**, " Design of Concrete Structures for Buildings (CAN.3-A23.3-M84)", National Standard of Canada Rexdale, Canada,.

 **Carette, G., and Malhotra, V.M., 1983**, "Early-Age strength Development of Concrete Incorporating ACISP79-41", pp.765-784.

 **Carrasquillo, R. L., Nillson, A. H., and Slate, F. O., 1987**, " Properties of High Strength Concrete Subjected to short – term – loads", ACI Journal Proceeding 78 No .3, May – Jun, PP . 171 -178.

 **Castillo, C., and Durrani, A.J., 1990**, "Effect of Transient High Temperature on High-Strength Concrete", ACI Materials Journal, Vol.87, No.1, Jan-Feb, pp47-53.

 **Chan, S.Y.N., Peng, G., and Chan, J.K.W., 1996**, "Comparison between High-Strength Concrete and Normal-Strength Concrete Subjected to High Temperature "Materials and Structures, V.29, No.12, pp.616-619.

 **Chan YN, Lox. Sun W., 2000**, " Compressive Strength and Pore Structure of High – Performance Concrete After Exposure to High Temperature Up to 800 °C ", Cement and Concrete Research , Vol .30, No .2 , Feb, pp .247 – 251.

 **Chao-Wei Tang, Tsong Yen, 2004**, "Prediction of the Ultimate Shear Strength of High and Normal – Strength Concrete Beams Without Stirrups Using Neural Network" 6th International Symposium on High Strength/ High Performance Concrete Taiwan.

 **CEB-FIP, 1993**, Comite Euro-International du Beton-Federation International Precontrainte "Model Code for Concrete Structures", Mc-90, Thomas, London, (Cited by Ghali,1993).

 **Edward, W.T., and Gamble, W.L., 1986**, “Strength of Grade 60 Reinforcing Bars After Exposure to Fire Temperatures”, Concrete International, Oct, p.1.

 **Essa, M. S., 1999**, “ Effect of Burning by Fire Flame on Properties of Concrete”, Journal of Babylon University, Series E, Engineering Science, Vol.4, No.5, October, pp. 1192-1202.

 **Ghali, A., and Azarnejed A., 1999**, “ Deflection Prediction of any Concrete Strength ”, ACI Structural Journal, Vol.96, No.5, September-October, pp. 807-816. [Cited by Reference Ghali, 1993]

 **Habeeb, G. M., 2000**, "Residual Mechanical Properties of High Strength Concrete Subjected to Elevated Temperature", Ph.D, Thesis, College of Engineering, Department of Civil Engineering, Al- Mustansiria University, Baghdad, Iraq, November, 164 pp.

 **Hibbeler, R.C., 2005** "Mechanics of Materials" 6th Educational Publishers, Pearson Prentice Hall, 873pp.

 **Holmes, M., and Anchor, R.D. 1982**, “The Effects of Elevated Temperatures on the strength Properties of Reinforcing and Prestressing Steels”, Structural Engineering, V.60, No.1, March, pp.7-13.

 **Hotton, R., 1984**, "Permeability and Pore Structure of Cement Paste Containing Fly Ash, Slag, and Silica Fume", ASTM Committee C-I on cement, June, pp. 128-143.

 **Hsu, H.J, and Lin,C.S, 2006**, "Residual Bearing Capabilities of Fire-Exposed Reinforced Concrete Beams", International Journal of Applied Science and Engineering, Chaoyang University of Technology, ISSN 1727-239.

 **Iraqi Organization of Standards , IOS 5 : 1984** , for Portland Cement.

 **Iraqi Organization of Standards, IOS 45 : 1984**, for Aggregate.

 **Iravani, S., 1996**, " Mechanical Properties of High performance Concrete ", ACI Materials Journal, No.93-M47, September-October, pp.416-425.

 **Jahren, P.A., 1989**, "Fire Resistance of High-Strength/Dense Concrete with Particular Reference to the Use of Condensed Silica Fume-A Review", SP-114, American Concrete Institute, Farmington Hills, Mich, pp.1013-1049.

 **Jensen, J.J., Hammer, T.A., and Hansen, P.A, 1997**, "Fire Resistance and Residual Strength of HPC Exposed to Hydrocarbon Fire," Proceedings of International Workshop on Fire Performance of High-Strength Concrete, NIST SP 919, NIST, Gaithersburg, Feb. 13-14, pp.59-68.

 **Karem, S. Sahib., 2006**, "Flexural Behavior of Hybrid Reinforced Concrete Beams" M.Sc. Thesis, College of Engineering, Department of Civil Engineering, Al-Mustansiria University, Baghdad, Iraq, July.

 **Kong, F.G, Robins, P.J., and Cole, D.F., 1970**, "Web Reinforcement Effects on Deep Beams", ACI Structural Journal, Dec, pp.1010-1017.

 **Khan, M. R., and Royles, R., 1986**, " Post –Heat Exposure Behavior of Reinforced Concrete Beams ", Magazine of Concrete Research, Vol.38, No.135, June, pp.59-66.

 **Lin, I. J., Chen, S. T., and Lin, C. J., 1999**, "The Shear Strength of Reinforcing Concrete Beam after Fire Damage". "Structure Safety Evaluation after Fire Damage", Scientific & Technical Publishing Co., Ltd., Taiwan, 117-136.

 **Lin,W.M., Lin, T.D., and Powers-Couche, L.J., 1996**, "Micro structures of Fire-Damaged Concrete ", ACI Materials Journal, V.93, No.3, May-June, pp.199-205.

 **Lox, Sun W ., and Chan YN . 2000**, "Effect of Heating and Cooling Regimes on Residual Strength and Microstructure of Normal Strength and High-Performance Concrete " , Cement and Concrete Research , Vol .30 , No . 3 , Mar, pp. 379- 383.

 **Mehta, P.K., and Aitcin, P.-C., 1990**, " Principles Underlying Production of High-Performance Concrete", Concrete, Cement, and Aggregates, V.12, Winter, pp. 70-78.

 **Moetaz, M. E., Ahmed, M. R., and Shadia, E. 1996**, " Effect of Fire on Flexural Behavior of RC Beams", Construction and Building Materials, 10, 2:147-150. [Cited by Reference Hsu. 2006].

 **Morley, P.D and Royels, R., 1983**, "Response of the Bond in Reinforced Concrete to High Temperature", Magazine of Concrete Research, Vol.38, No.123, Jun, pp.67-75.

 **Muhammed, Nura. J., 2007**, "Residual Strength of Reinforced Normal Strength, High Strength and Light Weight Concrete Panels Exposed to High

References

Temperatures", M.Sc.Thesis, College of Engineering, Department of Civil Engineering, Al- Mustansiria University, Baghdad, Iraq, May.

 **National Building Code of Canada, 1995**, National Research Council of Canada, Ottawa, Ontario Canada.

 **National Ready Mixed Concrete Association, 2000**, "What, Why and How? Supplementary Cementitious Materials", Concrete in Practice, CIP 30, USA, pp1-2.

 **Neville, A.M., 1995**, "Properties of Concrete", Longman Group, Ltd., 4th and Final Edition, pp.(329-397), (674-687).

 **New Zealand Standards Association, 1982**, "Code of Practice for Commentary on: The Design of Concrete Structures (NZS 3101:1982, Part 1 and 2)", Standard Council, New Zealand.

 **NISTIR 6295, 1999**, "Curing of High Performance Concrete" report of the state –of the building and fire research laboratory, Gaithersburg, Maryland 20899, March.

 **Nilson, A.H., and Darwin D., 1997**, "Design of Concrete Structures" 12th Edition, 780 pp.

 **Park, R., and Paulay, T., 1975**, "Reinforced Concrete Structure". Jon Wiley and Sons, 769 pp.

 **Paulson, K. A., Nilson, A. H., and Hover, K. C., 1991**, "Long-Term Deflection of High-Strength Concrete Beams", ACI Materials Journal, Vol.88, No.2, March-April, pp.197-206.

 **Rebeiz, K. S., 1999**, "Shear Strength Prediction for Concrete Members", ACI Journal, Vol.125, No.3, March, , pp.301-308.

 **Shuaib H. Ahmad and D.M.Lue.,1987**, "Flexural- Shear Interaction of Reinforced High- Strength Concrete Beams "ACI Structural Journal, V.84, NO.4, PP 330-341.

 **Shuaib H. Ahmad and Roy Barker, 1991**, " Flexural Behavior of Reinforced High- Strength Light weight Concrete Beams", ACI Structural Journal, V.88, NO.1, PP 69-77.

 **Takeuchi, M., Hiramoto, M., Kumagai, N., Yamazaki, N., Kodaira, A., and Sugiyama, K. 1993**, "Material Properties of Concrete and Steel Bars at Elevated Temperatures", Trans of the 12th Int. Conf. On SMIRT, Vol.H, pp.133-138.

 **Tan, K.H., Kong, F.K., Teng, S., and Guan L., 1995**, "High –Strength Concrete Deep Beams with Effective Span and Shear Span Variations", ACI Structure Journal, July –August, pp.395-405.

 **The Benefits of High Performance Concrete, 1998**, contractors news June-July, p.p26-34.

 **Umran, M. K., 2002**, "Fire Flame Exposure Effect on Some Mechanical Properties of Concrete", M.Sc.Thesis, College of Engineering, University of Babylon, October, 103 pp.

 **U.S. Department of Transportation, 2005**, "Silica Fume User's Manual", Federal Highway Administration, April, p. 183.

 **Xie, J.,G., 1995**, "Mechanical Properties of Three High-Strength Concrete Containing Silica Fume", ACI Materials Journal, V.92, No.2, March-April, pp.135-145.

 **Wen, H.X., Bagheri, A.R., and Plum, D.R., 1989**, "The Strength Development of Silica Fume Concrete and its Prediction under Varying Temperature Conditions", Magazine of Concrete Research, Vol.41, No.149, pp.199-204.

 **Washa, G. W., and Fluck, P. G., 1952** "Effect of Compressive Reinforcement on the Plastic Flow of Reinforced Concrete Beams", ACI Journal, Vol. 49, No. 2, pp.89-108.[Cited by Paulson, K. A, 1991].

 **Zhang, B., Bicanic, N., Pearce, G.J., and Balabanic, G., 2000**, "Residual Fracture Properties of Normal-and High-Strength Concrete Subject to Elevated Temperature" Magazine of Concrete Research, V.52, No.2, pp.123-135.

 **Zsutty, T. C., 1971**, "Shear Strength Prediction for Separate Categories of Simple Beam Test ", ACI Journal, Vol.68, No.2, Feb, pp.138-143.