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BEHAVIOR OF HIGH STRENGTH SHEAR CONNECTORS EMBEDDED IN CONCRETE

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

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ABSTRACT

BEHAVIOR OF HIGH STRENGTH SHEAR CONNECTORS EMBEDDED IN CONCRETE

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The push-out sample consists of a small part of steel column. It is linked to a concrete wall through its both sides by connectors resisting shear force. The slip between the wall and the steel is measured at changing load increments or displacements. In this thesis, the structural behavior of three different types of shear connectors (Headed stud, L-shaped, and Channel) with ordinary and high strength steel properties embedded in different types of concrete (ordinary concrete, high strength concrete, and reactive powder concrete) was studied. The objective of this study was to investigate the behavior of selected shear connectors to determine the mechanical properties of different types of concrete and study the shear stiffness of shear connectors by conducting a total of 36 push-out tests specimens. In these test specimens, three types of shear connectors, headed stud, L – shaped and channel types of connectors, with two types of bond, welded and epoxy, were used. The results of this study indicated that reactive powder concrete increased mechanical properties of concrete as the concrete age increased. The specimens, OCHW, HCHW and RCHW generated the best shear force values for normal weight, high strength, and reactive powder concrete, respectively (The first letter is used to define the type of concrete: “O” is for ordinary, “H” is for high strength and “R” is for reactive powder concrete. The second letter is used to describe the type of shear connector, “C” is for channel connecter. The third letter is used to define the steel type for shear

connector, “H” is for high strength steel. The fourth and the last one is used for the type of connections between the studs and the steel plate, “W” is for Welding).

Keywords: Push out Test, Shear Connectors, Reactive Powder Concrete, High Strength Concrete

ÖZ

BETONA GÖMÜLÜ YÜKSEK MUKAVEMETLİ KAYMA BAĞLANTI ELEMANLARININ DAVRANIŞI

ADIL HADI WARDI WARDI
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İtme deneyine maruz kayma elemanı, küçük bir çelik kolon bölümünden oluşan düzenek içerisine yerleştirilir. Çelik kolonun bağlantısı her iki taraftan bir beton duvara kesme kuvvetine direnç gösteren bağlantı elemanları ile sağlanır. Beton duvar ile çelik arasındaki kayma miktarı, değişen yük artışlarında veya yer değiştirmelerinde ölçülerek değerlendirmeye alınır. Bu tez çalışmasında, farklı tiplerde betona (normal beton, yüksek mukavemetli beton ve reaktif pudra betonu) gömülü normal ve yüksek mukavemetli çelik özelliklere sahip üç farklı tipte kayma bağlantısının (başlıklı kayma elamanı, L-şekilli ve C ya da U profilli) yapısal davranışı çalışılmıştır. Bu çalışmanın amacı, farklı beton türlerinin mekanik özelliklerini belirlemek için seçilen kayma bağlantı elemanlarının yapısal davranışını araştırmak ve toplam 36 adet itmeye maruz test numunesi oluşturarak kayma bağlantı elemanlarının kesme kuvvetine karşı rijitliğini incelemektir. Deneysel çalışmada toplam üç tip kayma bağlantı elemanı (başlıklı, L şekilli ve C tipi) ile iki tip bağlantı türü (kaynaklı ve epoksi) kullanılmıştır. Bu çalışmanın sonuçlarına göre betonun yaşı arttıkça reaktif pudra betonunun mekanik özelliklerinin de arttığı görülmüştür. En iyi kesme kuvveti değerinin; OCHW, HCHW ve RCHW deney çalışmalarında kullanılan sırasıyla normal ağırlık, yüksek mukavemet ve reaktif pudra betonu için

elde edildiği tespit edilmiştir (Deneylere ait adlandırmada, ilk harf beton tipini tanımlamak için kullanılmıştır: “O” normal, “H” yüksek mukavemetli ve “R” ise reaktif pudra betonu için kullanılan kısaltmalardır. İkinci harf kayma bağlantı elemanı tipini tanımlamak için kullanılmıştır: “C” C ya da U profilli bağlantılar için kullanılan bir kısaltmadır. Üçüncü harf, kayma bağlantı elemanının çelik tipini tanımlamak için kullanılmıştır: “H” yüksek mukavemetli çelik için kullanılan bir kısaltmadır. Dördüncü ve sonuncusu, kayma elemanları ile çelik levha arasındaki bağlantıların türü için kullanılmıştır: “W” kaynaklı bağlantı türü için kullanılan bir kısaltmadır).

Anahtar Kelimeler: İtme Testi, Kayma Bağlantı Elemanları, Reaktif Pudra Beton, Yüksek Mukavemetli Beton

DEDICATION

I would like to dedicate my thesis to my father for being encouragement and supporting to me through my life. To whom words cannot describe her my lovely mother, she has actively supported me in my determination to find and realize my potential and to make this contribution to our world. It is also dedicated to whom companion my life, my wife. Also dedicated this to all my brothers and my sisters who are also my friends and supporters in this life. I would also like to thank all my family and friends for helping me during my study journey. Finally, with your love and support, I was able to reach this goal.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS/ABBREVIATIONS

FEA	Finite Element Analysis
FRC	Fiber Reinforced Concrete
HSC	High Strength Concrete
MP	Macro-Polymeric
NC	Normal Concrete
NDTs	Nondestructive Tests
NFC	No Fine Concrete
NWC	Normal Weight Concrete
OPC	Ordinary Portland Cement
PP	Polypropylene
RCC	Reinforced Cement Concrete
RF	Resonance Frequency
RPC	Reactive Powder Concrete
SCC	Self Compacting Concrete
SCMs	Supplementary Cementitious Materials
UHPC	Ultra-High Performance Concrete
UPV	Ultrasonic Pulse Velocity

CHAPTER 1
INTRODUCTION

1.1 General

Composite structures term means that structural element with others are connected by any connections type. Composite structure analysis was discussed by many researchers, through experimental and theoretical researches depending on degree and type of connections. Many theories take into account by applying equilibrium and compatibility equations to derive the relationships between all parameters of composite structures.

In recent years, the world has used composite materials in composite structures instead of traditional raw materials and used some materials that give more enhance of mechanical properties to resists various types of loading in short and long time periods. The goal of using composite structure is to increase strength of structural members, and design economic cross sections.

1.2 Shear Connectors

There are many types of shear connectors that are classified according to the behavior of shear connectors (rigid and flexible). A rigid shear connector is a stiff one and exhibits little or no deformation. The common types of rigid connectors are the channel and stud shear connectors. Flexible shear connectors, such as headed studs and channels, deforms more than rigid types before their failures. The benefits of headed shear connectors are to resists shear that is developed between two materials, concrete and steel, and the head prevents pull – out failure.

1.3 Composite column

A composite column consists of deck and corrugated sheet or steel plate connected by shear connectors. The composite column may contain more than one layer so it is called multilayer composite column. The presence of sheet or straight plate (fillet) not work as permanent frame work but also work as additional reinforcement at the bottom so that the resistance of concrete in tension become more ductile.

Concrete columns are connected to steel sections through shear connectors and the assembly makes a composite structure. These composite structural elements have been used in buildings of residential or office, and in industrial facilities due to their fast construction capability allowing savings in overall cost of a construction. Composite columns can rest on steel columns, concrete or masonry walls.

Composite columns are designed in two stages, construction stage and service stage (composite stage). Shoring and un-shoring are the two most important stages that must be taken into account in designing composite columns and beams. In shoring, steel plate or steel beam is supported to carry the wet concrete and self-weight of the steel section. After the concrete is dry and gains most of the strength, supports are removed and then the composite action begins. However, in an un-shoring stage, the steel section alone carries all the weight without any types of support.

The analysis of composite columns depend on the type of theory that is adopted for the different types of connections. In the first type, there are no connections between concrete column and steel plate. In this case, concrete column rests on steel plate or corrugated sheet, and there are no shear connectors between them so that they work independently. The ultimate load is at its minimum and almost no composite action is observed. The failure in this case is progressive.

If a concrete column is connected infinitely stiff to a steel plate by means of stud connectors in each direction so that the slip is around zero, then in this case is assumed that there is full interaction between two different materials that differ by modulus of elasticity and poisson's ratio. In this case, the strength and stiffness of composite column are increased while there is decrease in stresses and deflections. The failure can be brittle if it occurs suddenly or ductile if it happens progressively.

1.4 Push Out Test

The push-out sample consists of a small part of steel column. It is linked on both sides to a concrete wall by connectors resisting shear as shown in Fig. 1.1. The walls are bedded with mortar connected directly to the reaction floor with load being applied to the higher end of the member. The slip between the two walls and the steels is measured at load increments or displacement. The average value of the slip usually graphed against the load per connector.

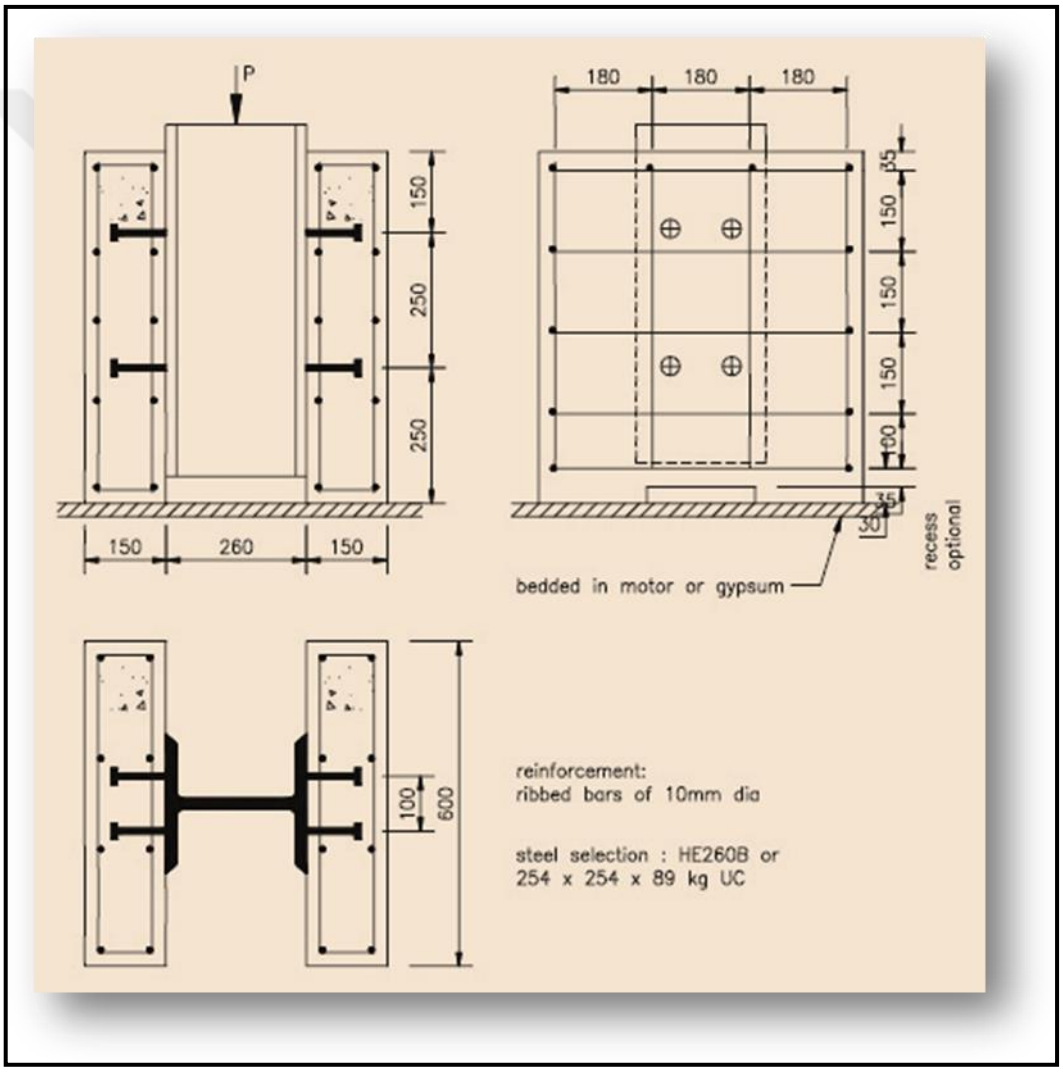


Figure 1.1 Push – out test according to Euro code 4.[84]

1.5 No-Fine concrete

No-fine concrete is a special type of conrete obtained by eliminating the fine material (fine sand) from the normal concrete mix. The single-sized coarse aggregates are surrounded and held together by a thin layer of cement paste giving strength of concrete.

The advantages of this type of concrete are:

1. Lower density compared with other types of concrete..
2. Lower cost due to lower cement content.
3. Lower thermal conductivity relatively low drying shrinkage.
4. No segregation and capillary movement of water.
5. Better insulating characteristics than conventional concrete because of the presence of large voids.

1.6 Objective of the Study

The main purpose of the study is to investigate the structural behavior of different types of shear connectors (at least 3 types) with ordinary and high strength material properties embedded in different types of concrete (ordinary concrete as reference, high strength concrete, reactive powder concrete).

The details of the experimental program for this study are presented below, including material testing, which are used during, specimen's preparation and test procedures (Table 1.1). The objective of this study then can be summarized as follows:

1. Investigate the mechanical properties of different types of concrete including traditional tests for raw materials, compressive strength, tensile strength, and flexural strength.
2. Study the shear stiffness for shear connectors by conducting a total of 36 push-out tests specimens including three types of shear connectors (Headed stud, Channel, and L – shaped) with two types of bond; welded and epoxy.

Table 1.1 Test parameters

Type of concrete	Type of shear connectors	Type of steel	Type of connection
Normal weight concrete, High strength concrete, Reactive powder concrete and No fine concrete	Headed stud , L-shaped and Channel	Ordinary strength and High strength	Welding and epoxy

CHAPTER 2

LITERATURE REVIEW

2.1 Introductions

Push-out testing is frequently used in experiments to assess the interfacial shear strength developed at a composite interface. Behavior of shear connections is of a great importance in steel-concrete composite structures. This behavior is generally recognized by the load-slip curve obtained from the test of experimental push-out, although it is expensive and time consuming. The most important characteristic relations for the design of a composite structure are the load-slip and the shear capacity. These relations can be determined from a full-scale composite beam test but it might be costly and a time-consuming, therefore push-out tests are selected.

2.2 Composite Structure

Analysis of composite structure was discussed by many researchers based on the experimental tests that depended on different types of connections. Following is a summary of some published articles.

Many theories take equilibrium and compatibility equations into account to derive relationships between all parameters of a composite beam. The oldest theory for composite beams is adopted by Newmark [1] in 1951 that was based on elastic analysis and linear materials for concrete, steel and shears stud connectors. The axial force that was developed between the interface of steel beam and composite slab was obtained by:

$$F_{xx} - \alpha_2^2 F = -\alpha_2^2 \beta_2 M \tag{2.1}$$

Where,

$$\alpha_2^2 = \frac{k}{s}\left(\frac{1}{A_c E_c} + \frac{1}{A_s E_s} + \frac{d_c^2}{E_c I_c + E_s I_s}\right) \tag{2.1.1}$$

$$\sum E I = E_c I_c + E_s I_s \tag{2.1.2}$$

$$\overline{EI} = \sum EI + \overline{EI}\, d_c^2 \tag{2.1.3}$$

$$\beta_2 = \frac{\overline{EA}}{EI} d_c \tag{2.1.4}$$

Where,

α and β: constants and they are functions of mechanical properties and geometry.

dc: distance between the centroids of concrete and steel layers.

M: applied external moment.

F_c and F_s: axial forces in concrete and steel layers, respectively.

I_c , I_s: moment of inertia for concrete and steel layers, respectively.

A_c, A_s: cross sectional area for concrete and steel layers, respectively.

The solution of the basic differential equation in 2.1 led to determine the axial forces in concrete and steel and then slip at the interface between concrete and steel sections.

In 1968, Adekola [2] derived equations that represent uplift forces T and axial forces F as follows:

$$T_{xxxx} + k_n\left[\frac{1}{E_c I_c} + \frac{1}{E_s I_s}\right]T + \frac{k_n \rho}{E_c I_c} - k_n\left[\frac{Z_{si}}{E_c I_c} + \frac{Z_{ci}}{E_s I_s}\right]F_{xx} = 0 \tag{2.2}$$

Where,

$$F_{xx} - k_s\left[\frac{1}{E_s I_s} + \frac{1}{E_c I_c} + \frac{d_c^2}{E_s I_s + E_c I_c}\right] F + \frac{k_s}{k_n}\left[\frac{E_c I_c Z_{si} - E_s I_s Z_{ci}}{E_s I_s + E_c I_c}\right] T_{xx} = \frac{-k_s d_c}{E_s I_s + E_c I_c} M \quad (2.3)$$

ρ: represent load per unit length.

kn: is the normal shear stiffness for connectors.

In 1975, Johnson [3] derived a second order differential equation taking slip between concrete and steel into account for a simply supported beam but without the presence of uplift forces. The derived equation was as follows:

$$U_{cs,xx} - \alpha_1^2 U_{cs} = -\alpha_1^2 \beta_1 N \quad (2.4)$$

Where,

U, is the slip at the interface between steel and concrete. The other parameters in this equation are as follows:

$$\beta_1 = \frac{A_1 S d_c}{k} \quad (2.4.1)$$

$$\alpha_1^2 = \frac{k}{S E_s I_1 A_1} \quad (2.4.2)$$

$$I_1 = \frac{I_c}{m} + I_s \quad (2.4.3)$$

$$\frac{1}{A_o} = \frac{m}{A_c} + \frac{1}{A_s} \quad (2.4.4)$$

$$A_1 = d_1^2 + \frac{I_1}{A_o} \quad (2.4.5)$$

Where,

N: is the vertical shear at a certain section.

S: spacing between center to center of shear stud connectors.

K: is the shear stiffness of shear stud connector.

m = Es/Ec.

In 1985, Roberts [4] suggested four ordinary differential equations including slip and uplift forces as follows:

$$M_{c,xx} + M_{s,x} - F_{s,xx}\, d_1 = \rho \qquad F_{c,x} + F_{s,x} = 0 \tag{2.5}$$

$$F_{c,x} - k_s\,[(U_{ci} - Z_i\, W_{c,x}) - (U_s - Z_{ci}\, W_{s,x})] = 0 \tag{2.6}$$

$$M_{c,xx} + F_{c,xx}\, Z_{ci} - k_n\,(W_s - W_c) = \rho_s + \rho_c \tag{2.7}$$

In 1989, Narayanan et al. [5] investigated shear strength capacity of composite plate girders in presence of web cut - outs. Test results showed that the shear strength capacity of composite girder was greater than steel girder.

In 1990, T. M. Robert and Al-Amery [6] derived sets of equations by applying equilibrium and compatibility equations to a composite beam taking the slip and uplift effect into account by adopting partial interaction between steel beam and reinforced concrete slab. The sets of equations were solved by the method of finite difference and all unknowns like slips, strain, stresses and forces were also determined.

In 1995, Mohammad Makki Abbass [7] investigated the behavior and strength of a composite plate girder with reinforced web perforation. Twelve composite plate girders that contained reinforced circular web perforation were tested and compared with the reference specimen without web opening. Sets of equations were derived and solved analytically to calculate the shear strength of composite plate girder with and without web openings.

In 1997, Al-Amery and Mohammad Makki Abbass [8] suggested a set of analytical equations to analyze the system, and found out the angle of failure under shear for composite plate girders with and without web cut outs. The derived

equations dealed with the ultimate strength capacity of composite plate girders with and without web circular cut – out as functions of girder geometry and parameters.

Dezi, et al. 2001 [9] suggested an algebraic method which was used for non–complex composite structures. For this purpose, the headed studs shear connectors were known to behave in a complex manner, so the pseudo–elastic analysis was used. In this study a modified Young's Modulus for concrete was used. The effect of shrinkage was modified by creep, and was estimated by means of a pseudo–elastic analysis as shown below:

$$\overline{E}_e(t,t_0) = \frac{E_C}{1+\varphi(t,t_0)} \tag{2.8}$$

$$E_{C,MS}(t,t_s) = \frac{E_C}{1+0.5\varphi(t,t_s)} \tag{2.9}$$

Where:

E_c = elastic modulus of the concrete

$\varphi(t,t0)$ = creep coefficient

t = loading time

t_0 = final time

t_s = age of concrete at the beginning of shrinkage

According to Gilbert [10] the creep coefficient changes with time, as shown in Figure 2.1. The values of the creep coefficient calculated by using Eq. (2 .10) below, the value of creep coefficient reduces the elastic modulus of concrete.

$$\overline{E}_e = \frac{E_c}{(1+\psi\varphi)} \tag{2.10}$$

where,

φ = creep coefficient,

ψ = aging coefficient. (The value of 0.5 is most appropriate as used in the German design code (DIN 1045–1) to allow time-induced stress).

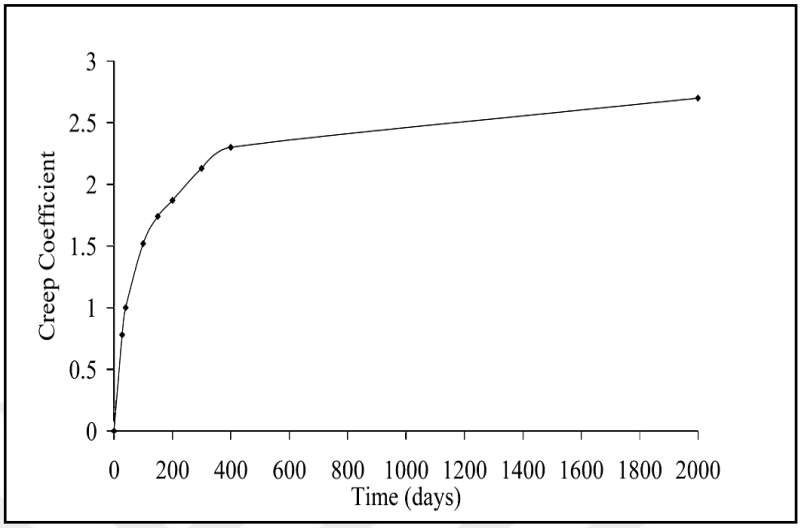


Figure 2.1 Creep Coefficient of Concrete with Respect to Time, Gilbert [10]

In 2004, Mohammad Makki Abbass [11] investigated the full behavior and strength of composite beams subjected to cyclic loading. The solution included linear and nonlinear materials of concrete, steel and stud shear connectors under static and cyclic loadings. The mechanical properties of all composite material as function of number of cyclic. Set of ordinary differential equations was derived including the friction effect between the steel beams and concrete slab.

In 2004, Khalil [12] studied short and long term effects for 180 days on the mechanical properties of reinforced concrete, RC slabs. Total of eight RC slabs with main parameters considered the polymer ratio of 3, 5 and 7 percent. Deflections were compared with respect to reference specimen and showed that they decreased as the polymer ratios increased for short and long term periods.

In 2005, Aziz, K. I. [13] suggested a new model that explained the performance of multilayer composite slab using partial interaction theory subjected to static loading. Differential equations were derived by applied equilibrium and compatibility equations and they were solved by using computer codes. Linear and nonlinear analysis of concrete, steel plates and shear stud connectors were used taking slip and uplift into account in simulations. The unknown variables such as

slip, deflection, stresses and strains were solved at each incremental loading. Specimens of push-out tests were tested to determine the load-slip relationship, maximum load capacity and maximum slip value. Validation and verifications to evaluate the actual behavior of multilayered composite slab were checked with respect to the experimental results.

In 2006, Amit Pashan [14] studied the experimental study that included the testing of push-out samples with channel shear connectors. The test program Included the testing of 78 push-out samples, and its objective was to develop new equations for channel shear connectors in solid concrete slabs and panels with a large ribbed metal surface placed parallel to the beam. Six series of push out samples were tested in two phases. The height of the channel connector was the primary parameter studied difference between the two phases. On average, when the channel length was increased from 50 mm to 100 mm, the increase was about 39%. A further increase of 24% was recorded as the channel length was increased from 100 mm to 150 mm when the failure occurred due to channel web fracture. The influence of web thickness of channel connector was significant but was minimal for a concrete crushing-splitting type of failure.

In 2010, Mark A. Bradford [15] studied the composite slab behavior due shrinkage and creep effects. Due to long-term effects, shrinkage, creep, axial force and axial strain values were recorded during tests. There was deformation through slab thickness due to shrinkage and also an increase in deflections and strain were recorded.

In 2010, Dipl J. [16] investigated the time dependent behavior of composite slab under service and ultimate loadings. Deflections and strain were measured during tests to evaluate the real behavior of composite slab under long term effects. The results showed that deflections strain and slip were affected by the connections of composite slab components and that the amount of creep varied according to connection type.

In 2011, Stefano De Vittorio [17] examined the long term effects on composite slab. The experimental tests concerned on cracked cross section of slab

and evaluated the effects of shrinkage and creep on the behavior of slab under ultimate loading. Ten slabs were casted and tested by taking into account the environmental effects. The results showed that there was a difference in deflection. The deflections between the slabs were subjected to shrinkage and creep after the partial cracking of cross section and that after several weeks when the cross section not cracked.

In 2012, R.I. Gilbert et al. [18] investigated the effects of time – depend of composite slabs under service loading. Shrinkage through thickness of slab was attended and showed that the long term effects on deflections and dry shrinkage of slab.

In 2013, G. Ramakrishna and T. Sundararajan [19] investigated the long term effects on the mechanical properties of composite mortar. The compressive strength, flexural, durability and other properties was evaluated under the effects of long varying from 28 to 120 days. The results indicated that all mechanical properties and long- term strength were improved.

In 2013, M. Fragiacomo and E. Lukaszewska [20] examined by experimental tests the long term effects of composite floor. The composite systems subjected to sustained loading and keep it one year period. During the applied loading, deflection, slip and strain at different positions was recorded. There were a little variation of deflection, slip and strain with time and small increased was recorded.

In 2013, Al-Ranzi Gianluca et al. [21], presented state of art and discussed time - depend behavior of composite structural elements (concrete and steel) that composite beams, composite columns and composite slab subjected to service and ultimate loading. In case of composite slab, the creep and shrinkage was studied and there were little difference and changing in results of deflection and slip but there were in ultimate loading.

In 2013, Aida Mazoz1, et al [22] investigate the behviour of I-shape with a new shear connector by Push-out tests. The study summarizes 24 results of push-out test samples with I-shape shear connector. The test samples were prepared to study of the action of I-shape connector height, the I-shape connector length, strength of

concrete and the number of cross bars reinforcing on the ultimate load capacity. The experimental outcome are offered and argued, focusing on failure form and load-slip method. Finally, the experimental outcome are compared and predicted the ultimate load capacity for this connectors.

In 2015, Safat Al-deen et al. [23] investigated by experimental tests to evaluate the long term behavior of composite steel concrete beams with solid slab that design by partial interaction theory. The composite floor slab was rested on secondary composite beams that tested. During tests, shrinkage and creep was measured by applied sustained distributed loading. Push – out tests was adopted in case of short and long term to study the relationship between shear and slip for both cases. There was difference in tests result between short and long term deflections and strain.

In 2015, Chengcheng Du et al [24] studied the time depend strain of concrete by applied step by step history loading. The experimental work adopted was to apply loading during the construction stages of high rise building and monitoring the strain progress. The authors noticed that the strain developments of early age affected by loading age and stress range.

In 2015, Huiyong Ban et al. [25] studied the time depend of composite beams subjected to sustain loading. A total four full scales of composite beams with simply supported boundary conditions at the ends was tested and subjected to long term loading during nine months. Push – out tests to evaluate the behavior of stud shear connectors were also carried out for load – slip relationship. The concept that to use blind bolts was successful to get lower values of creep, shrinkage and deflection for long-term effects.

In 2016, Jing Zhou, et al. [26] developed and explained a pull-out test for an improved fiber-bundle. It was to be a practical plan out of the monitoring of the surfaces fracture with ocular microscopes, and another comparison of data results between this test and several single-fiber tests. The heating of MW could be sufficiently reinforced the interaction of carbon fiber/epoxy composites, this process

increased of about 52.8% in interfacial shear strength. At the same time, the decrease of a cycle time was 54.2% compared with thermal curing.

Xing, et al. (2016), [27] examined the shear behavior of shear connections through tests of push-out tests as a solution of FRP-concrete hybrid beam. Two utilities that (FRPs) provide its compared with classic steel bolts, regarding to at least 2.5 times the shear capacity and about 10 times the slip modulus. Equations were adopted for calculations to facilitate the design of shear capacity and slip modulus of FRP.

Sameera, et al. (2016), [28] investigated the feasibility of using smooth bolts as shear connectors to develop demountable steel–concrete beams. By using full-scale beam specimens, the flexural behaviour of composite beams with types of smooth bolt and welded stud connectors was tested. The FEMs were constructed by using the load–mid span deflection curves obtained from the experiments.

In 2017, Jun-Seong Park, et al. **[**29] examined the bond strength of fiber-reinforced concrete for the push-out tests with cement layers of different thickness. The specimens were made by horizontally sectioning them into 1-mm-thick slices. Then, the push-out were evaluated the bond strengths of the fragments.

Jun-Yan Wang, et al. (2017), [30] presented a series of tests on headed stud shear connectors in composite structures through push-out tests Many testing parameter investigated such as effects of initial stiffness, ultimate strength, the shank diameter and ductility of the shear connectors. Five groups of the shear connectors each with three specimens were tested. Design recommendations and formula are presented for the shear strength based on the experimental results.

Qinghua Han, et al. (2017), [31] investigated single-headed stud connection tests for concrete slabs with fragment rubber and steel beams. A commercially available software of ABAQUS used to develop as an accurate numerical analysis to derive damage plasticity models and nonlinear material properties of concrete with four different rubber contents and reasonable contact algorithms. The results show that the most dangerous one is bottom welding defect of the stud , especially on the ductility and ultimate slip of the stud.

Pankaj Kumar, et al. (2017) [32] observed and provided a qualitative comparison under impact load of the behavior of steel-concrete composite connections bonded and mechanically connected. It was found that the tests showed ductile failure nearby the studs due to concrete crushing in composite specimen in the the mechanically connected specimen. Whereas, the connection of adhesively bonded can resist higher number of blows (twice) for crack initiation but, the required number of blows were relatively less for final failure. In addition, the failure in concrete near the adhesive concrete interface was brittle.

Chihiro Hirama, et al. (2017), [33] showed the similarity tendency in the shear strength between headed studs in slabs with a steel deck cut on the flange and headed studs in a solid slabs. The shear strength of headed studs decreased gradually with the increasing diameter in both failure modes with a diameter of more than 25 mm in solid slabs gradually. Furthermore, the shear strength was studied by slab type and failure mode of the headed studs with a diameter of more than 25 mm.

In 2018, Vinicius et al. [34] evaluated the influence of sample arrangement, zirconia surface treatment, and bonding area using finite element analysis (FEA). The adhesion increased significantly by surface treatment, except for 'resin cement macro-test' ($p=0.00$). FEA proved that the stresses distributed were more homogeneously at the interface of micro-specimens.

Jin Dia (2018), [35] showed that the increased magnitude of connectors with large-size will reduce the number of connections as an excellent alternative. Additionally, the flow of coarse aggregate concrete can be improved by an increased its diameter. For this purpose, 13 groups of 39 push-out specimens were tested to verify. The results indicated that the bearing capacity would be improved by increases the geometrical size of the hole under a high level of a concrete dowel confinement. Finally, an empirical formulation for the shear capacity was proposed considering taking account multiple influencing factors.

In 2018, Dominic Kruszewski et al. [36] developed novel repair method for the corrosion damage at the steel bridge girders. The idea embraced welding headed studs and covering it in ultra-high performance concrete (UHPC). Also, presented a

series of experimental results of push-out specimens with studs welded to a plate with 9.5 mm thick, which was extracted from an old steel bridge. A formulation was refined based on obtained results of headed studs embedded in UHPC.

Jiaxin Chen, and Nawawi Chouw (2018), [37] conducted an experimental study on the bond of reinforced concrete composite coconut fiber tied with flax fibre reinforced polymer. The interface between the flax fiber and coconut fibre were considered by four different conditions. Twelve cylindrical specimens were performed by push-out tests. In the results, the bond between the concrete core and polymer were highly reinforced in mechanically bonded interface.

2.3 High Strength Concrete (HSC)

In terms of a universally applicable numerical value, the definition of "high strength" is not possible. Term "High strength" is dependent on many things, such as the availability of good quality local materials and construction practices.

Strength is a relative property, which depends on any factors. Many factors influencing the measured strength of concrete include specimen geometry such as age, curing history, and size; testing equipment parameters, such as loading capacity, the loading rate and uniformity of load distribution, and lateral and longitudinal stiffness. Also, there are geographic considerations. For example, in regions where compressive strength of 60 MPa (9000 psi) is produced commercially on daily basis, concrete might not be considered "high strength" until it gets a strength in the range of 70 or 80 MPa (10,000 or 12,000 psi).

Conversely, in regions where the upper limit on commercially available concrete is 30 MPa (4000 psi), the concrete a achieving a design requirement of 40 MPa (6000 psi) might be considered high strength. The reason for such diversity can be attributed to: need and ability. Although the both are relative, need to the type of construction and the lead of the designer, and ability to the obligation of the concrete producer and quality of the the availability locally materials (Michael A. Caldarone 2008) [38].

2.3.1 Previous Studies on HSC

In 2010, H.B. Shim et al. [39] studied A lot of push-out tests of shear studs embedded in normal strength concrete and high strength concrete were conducted. Experimental push out tests were used to evaluate both the load-slip curve of the connector and the shear stud capacity. push-out tests results and the values given in current codes of practice were compared to the results of the finite element model.

K. Holschemacher et al (2010), [40] conducted tests with a task of steel fibres which have different groups with steel bar reinforcement. Experiments show that for all selected fibre contents were obtained a more ductile behaviour and higher load levels in the post-cracking range. The study creates a rule for selection of suitable fibre contents and types for reinforcement.

In 2011, M.A. Megat Johari et al. [41] studied the influence of supplementary cementitious materials (SCMs) on the engineering properties of high strength concrete (HSC), namely silica fume, fly ash, metakaolin, and ground granulated blast-furnace slag. The results show that the inclusion of the different SCMs had considerable influence on the workability of HSC. Silica fume and metakaolin significantly enhanced the strength of HSC. Fly ash and ground granulated blast-furnace slag reduced the early-age strength but, it enhanced the long-term strength of the HSC.

Zhu Ming-qiao et al (2012), [42] comparing the reinforced steel fiber high-strength concrete thin-walled box girder with pure torsion, with the data of experiment, the full ranger of torsional of the specimen has been studied by ANSYS program. The results show that both of them were in harmony and verified that ANSYS program can figure out the full range of torsional behavior of the reinforced steel fiber high-strength concrete thin-walled box girder better, which offer reliable data in further research. The linear torsion angle and distortion angle of specimen tested under different torsion levels, which is in different sections and along the span of the girder is calculated by ANSYS program. The results demonstrate the rule that the linear torsion angle and distortion angle along the span of the girder is varying.

In 2012, Hama Sheelan M., [43] investigated time depend of composite beams in case of using high strength lightweight concrete. The experimental tests divided into three parts, first, the cube specimen to specify the mechanical properties of porcelinate aggregate. Second, tests the full composite beams by cast the slab by porcelinate reinforced concrete and tested for short term. Third, tests composite beam for long term effect. The results indicated that there were improvements in the lightweight composite beams timed dpendant behavior using admixture (silica fume and metakaolian).

P. Dinakar (2013), [44] presented the effect of merging metakaolian (MK) on the durability and mechanical properties of high strength concrete for a constant water/binder ratio of 0.3. This research showed the potential local MK to produce high strength and high performance concretes.

Saber Fallah (2017), [45] produced 280 concrete specimens in 28 different test groups, for which the engineering were evaluated, In this study Investigated the properties of HSC containing silica fume and nano-silica that affected by different amounts of polypropylene (PP) and macro-polymeric (MP) fibers. The mechanical properties of high-strength concrete was improved incorporating macro-polymeric fibers in the concrete mixture. Moreover, negative effects on the physico-mechanical properties of the HSC when high volume fractions of polypropylene fibers in the concrete mixture was used.

Shaohua He, et al (2017), [46] investigated twenty-four push-out specimen for Examined the structural behavior of perfobond strip (PBL) connectors for steel-concrete joint of hybrid girders with UHPC. Effects of different parameters discussed in depth. Furthermore, the author developed an analytical model for PBL/UHPC and obtained an appropriate parameters from present data, this parameters used in the hybrid girders' steel-concrete joints to provide reliable prediction of ultimate resistant-capacity of PBL.

Baek et al (2018), [47] investigated the effectiveness of steel fiber inclusion on the structural performance of HSC columns with test of 7 specimens with and without steel fiber. In this study, all test specimens were subjected to axial load,

Therefor, anew developed estimation equation was suggested for the safe design of steel fiber reinforced high strength concrete columns.

2.4 Reactive Powder Concrete (RPC)

Reactive Powder Concrete (RPC) is a developing composite material that will allow the concrete industry (a) to help improve its use, (b) to build material use, build a strong structure, and (c) to durable, economic benefits, and sensitive to environment. The results show that RPC give high strength (both compressive and flexural) and lower permeability compared to HPC in comparison of the physical and mechanical properties and durability. The available literature on RPC will be presented, including laboratory investigations of both RPC and HPC. Specific benefits and potential applications of RPC have also been described. In fact, there are unlimited studies on RPC, and this section presents a review of some important parts of these studies.

In 2005, Lee and Chisholm [48] observed that poor tensile strength of composite cementations materials are improved by additng of steel fibers to the RPC mix. However, observed that such materials improve measured compressive strength of otherwise equivalent mixes to which they were added. The contribution of steel fibers is likely to reflect an increase in tensile strength due to improved compressive strength RPC, supposed to the accepted view that concrete under a uniaxial compressive load fails because of lateral strain caused by Poisson's ratio effects. However, it was also noted that the presence of the fiber typically produced a denser and more readily compacted mortar with less entrapped air, attributes that are also beneficial to the material's mechanical properties.

Kaseer (2007) [49] Investigated the Reactive Powder Self Compacting Concrete (RP-SCC) by using additive materials to enhance the mechanical properties of concrete like compressive strength, modulus of elasticity, and split tensile. According to the test results, the newly improved concrete reached a compressive strength up to 200 MPa and modulus of rupture around 27 MPa. Test results are recorded for 3, 7, 28, 60 and 90 days.

In 2008, Ibraheem [50] investigated the stress – strain relationship of RPC. The experimental investigations consisted types of fibers, volume fraction, and pozzolanic admixture of fibers. Based on the test results, a mathematical formula was proposed for stress – strain relationship indicating that RPC- silica fume mixes gave the highest values of compressive strength.

Hannawayya (2010), [51] studied the mechanical properties of RPC and also the flexural behavior of beams. The main parameters of the study were on fiber steel ratio and silica fume content. The flexural behavior was also checked using FEA models through using two methods the results were promoting implying that they were closes to the test results.

In 2011, Al-Hassani and Ibraheem [52] proposed an equivalent bi-linear compressive stress block for RPC sections under pure bending moment based on experimental stress-strain curves conducted by Ibraheem in 2008 [50] The proposed bi-linear compression block was used to derive an equation for calculating the nominal ultimate bending moment capacity (M_n) of rectangular singly reinforced RPC sections. The accuracy of the derived M_n equation was examined by comparing to the results of the experimental test conducted by Ibraheem.

Danha (2012) [53] investigated the tensile strength of RPC. The experimental and theoretical approaches were used to find the tensile strength including the enhancent tensile strength by using RPC. Different percentages of silica fume up to 30% and steel fiber up to 3% were used. Analytical models were proposed based on the experimental results, and also were checked by analytical solution. The results indicated that, using additives enhance the mechanical properties of concrete.

In 2012, Mahdi [54] investigated reinforced concrete columns by two approaches, first experimental tests and second analytical approach. The study focused on the strength of concrete in the presence of steel fibers and polypropylene fibers and then the results were compared with respect to normal concrete. The master variables were the compressive strength, lateral and longitudinal reinforcement, type and ratio of fiber. The results presented that the addition of steel fibers enhanced the mechanical properties of concrete.

In 2013, Ghailan [55] studied reactive powder concrete RPC in both experimental and theoretical works. Twenty-four tests including twelve RPC and another twelve modified RPC "MRPC" were subjected to shear. The results from the practical and analytical approach adopted by the author indicated that the use of RPC and MRPC compared to normal concrete showed a great sign of behavior. There was also an increase in shear strength.

Al-shafi'I (2013), [56] investigated the behavior of (RPC) of T section under shear. The beams were subjected to four point loads, a series of T section tests and rectangular beams were tested too, and compared with each other. The tests focused on recording cracked load, ultimate shear, mid deflection and crack width. The researcher concluded that increasing the percentage of steel fiber to (2%), increases the cracked load (60%) compared to T section constructed from normal concrete.

In 2013, Mohammed, [57] Tested RPC and normal concrete (NC) with twenty-four specimens by all failed in flexural. The main variables adopted by the researcher were RPC ratio, volumetric steel fiber, and longitudinal steel ratio. By experimental results, RPC gives higher capacity in loading and decreases in deflection.

Ismael (2013), [58] investigated experimental and theoretical works of (RPC) of T-section beam under flexural loading. A total of fifteen beams were tested taking into account various parameters like steel fiber, silica fume, tensile steel, and geometry section. Crack loads, strength capacity, maximum deflection, and failure mode were the main parameters studied by the author. By using block stress distribution, the author derived the equation to determine bending moment capacity. The tests result indicated that, increasing the RP, then the capacity loading increases.

Hmeed (2014), [59] Studied I-section of RPC in both experimental and theoretical works. The main parameters adopted were the concrete type and longitudinal steel ratio. The author noted that the behavior of beams under flexural depended on the type of concrete used in the flange. The change in concrete type in the top and bottom flanges maked changes in deflection and crack loading around hindered percent. Tests results focused on crack load, load capacity, mid deflection,

and failure mode. Tests results indicated that increasing ratio of RPC makes the beams increase in load capacity and decrease in deflections.

In 2015, Al-Hassani et al. [60] examined the behavior of T-beams in the case of RPC and normal concrete. The flexural behavior was investigated experimentally through Reinforced Cement Concrete RCC beams to understand the use of normal strength concrete with RPC. The authors concluded that using normal concrete in flange and RPC in a web of T-beam enhanced the behavior of T-beams under flexural effects.

In 2016, Masdar Helmi et al. [61] presented the effect of treatments on microstructure formation (8MPa static pressure and 240°C heat curing for 48h). Results of pressure showed that the total pore volume decreased, and the capillary pore volume increased due to the movement of grains. While, increased volume of the air and pressure within entrapped voids due to thermal expansion of the solid phases, heat treatment accelerated the propagation of microcracks.

In 2017, Xiaomeng Hou et al. [62] classified Reactive powder concrete (RPC) as ultra-high performance concrete, owing to its superior strength, excellent durability and high fracture energy. Also nondestructive tests (NDTs) was carried out to specimens. The resonance frequency (RF) method and ultrasonic pulse velocity (UPV) method were used. The samples were subjected to the different temperature of 120℃,to 900℃. a comparisons have been made between RF and UPV measurements versus residual mechanical strength, the result shows that the residual strength increases from room temperature to 300℃. However, above 300℃, the residual decreasing gradually. Relationships among residual mechanical properties versus NDTs values proposed.

M. Vigneshwari et al. [63] used an agricultural waste Rice Husk Ash (RHA) as a partial substitute for Silica fume SF with different percent from 10%, to 50% , under normal curing and steam curing. Test results showed that Rice Husk Ash can be used as an alternative material for Silica fume to produce RPC without compromising the required qualities, because of RPC containing RHA has satisfactory mechanical and durability performance under both curing.

Yanfeng Ruan et al (2018), [64] compared the properties of nano-zirconia (NZ) reinforced reactive powder concrete (RPC) with water curing and heat curing. also studied the method of curing mechanisms to RPC through X-ray powder diffraction (XRD) analysis, the mechanical behaviors were investigated for NZ reinforced RPC with heat curing . Experimental results showed that 3% NZ cured in hot water at 90 °C for two days presents a 15.0%, and 17.0%, and 35.0% of compressive, splitting tensile, flexural strengths, of RPC respectively increased compare with that of RPC with 3% NZ cured in water for 28 days. Therefore, It can be concluded that the stiffness of RPC can be reduced and the compressive/bending toughness of RPC can be improved when NZ is introduced.

2.5 Summary

Numerous researches concerning production of a new concrete in the presence of RPC was allude to above. Most of the results indicated that RPC make increased the compressive strength, modulus of elasticity, modulus of rupture, tensile strength, and workability. Shrinkage, creep and cracks decreased in RPC. Deflection and slip reduced in the composite structures.

The main objective of the current research work is to investigate the structural behavior of different types of shear connectors embedded in different types of concrete. For this purpose, high strength concrete, reactive powder concrete, and no fine aggregate concrete were used with normal weight concrete, which was as a reference. Also, different types of shear connectors including headed stud, L-shaped, Channel shear connectors were tested. Welding and epoxy were used as a bond between the steel plate and shear connectors.

CHAPTER 3
EXPERIMENTAL INVESTIGATIONS

3.1 Introduction

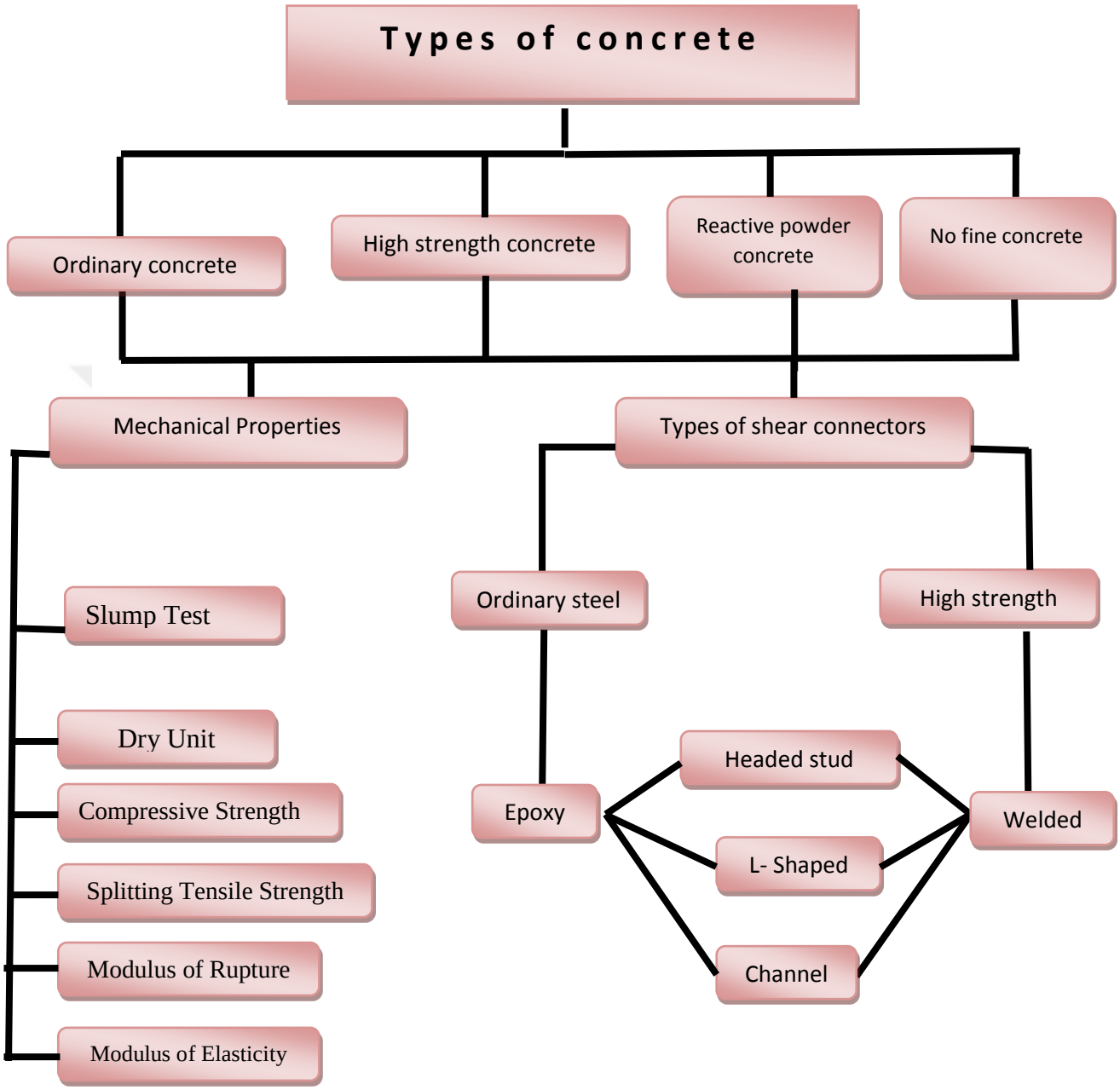
In this chapter, the details of the experimental program for this work are presented including the materials testing, which are used, specimen's preparation and test procedures.

The experimental program consist of two main parts: the first one was the study of the mechanical properties of different types of concrete including normal weight concrete (NWC), high strength concrete (HSC) and Reactive powder concrete (RPC). The second one was casting and testing 36 push-out test specimens including three types of shear connectors (Headed stud, Channel shape, and L – section) with two types of bond (welded and epoxy bond) between the shear connectors and the flange in steel section. The mechanical properties included in this study are as follows:

1. Slump Test
2. Dry Unit Weight
3. Compressive Strength
4. Splitting Tensile Strength
5. Modulus of Rupture
6. Modulus of Elasticity

The flow chart shown in Figure 3.1 describes the full layout of experimental program that considered in this work

Experimental Program



Four types of concrete x three types of shear connectors bond x two types of x two types of steel = total 48 specimens

Figure 3.1 Full layout of experimental program

3.2 Materials

3.2.1 Cement

Ordinary Portland cement was used to cast all specimens (OPC - Type I). Tables 3.1 and 3.2 show its physical properties and chemical composition, respectively. Test results indicate that the adopted cement conforms to the Iraqi specifications No.5/ 1984 [65].

Table 3.1 Physical properties of cement*

Physical properties	Test result	Limits of Iraqi specifications No.5/1984
Specific surface area, Blaine Method, (m^2/kg).	300	> 230
Setting time : -Initial setting (min.) -Final setting (min.)	 110 229	 ≥ 45 min. ≤ 600min
Compressive strength of mortar (MPa): 3-days 7-days	 21 30	 ≥ 15 ≥ 23
Soundness % (Autoclave)	0.02	≤ 0.8

* Physical properties were carried out in the Engineering Syndicate of Iraq , Branch of AL-Anbar – Construction laboratory

Table 3.2 Chemical composition of cement *

Oxide composition	Abbreviation	by weight%	Limits of Iraqi specifications No.5/1984
Lime	CaO	61	-
Silica	SiO2	19.84	-
Alumina	Al2O3	5.28	-
Iron oxide	Fe2O3	4.2	-
Sulphate	SO3	2.49	≤ 5%
Magnesia	MgO	2.48	≤ 2.8%
Loss on Ignition	L.O.I.	3.8	≤ 4%
Lime saturation Factor	L.S.F.	0.92	0.66-1.02
Insoluble residue	I.R.	1.13	≤ 1.5%
Main compounds (Bogues eq.)		by weight of cement%	
Tricalcium silicate (C3S)		46.95	
Dicalcium silicate (C2S)		24.52	
Tricalcium aluminate (C3A)		8.05	
Tetracalcium aluminoferrite (C4AF)		10.39	

* Physical properties were carried out in the Engineering Syndicate of Iraq, Branch of AL-Anbar – Construction laboratory

3.2.2 Fine Aggregates

Natural sand has been used as fine aggregate in concrete mixes. Table 3.3 shows the grading of fine aggregate used in this work, while Table 3.4 shows its physical and chemical properties. All tests of fine aggregate have been conducted in The Engineering Syndicate of Iraq, Branch of AL-Anbar – Construction laboratory. The results indicate that the grading and the sulfate content of fine aggregate are within the requirements of the Iraqi Specification No.45/1984 [65] Zone2 and Figure 3.2 shows the full sieve analysis behavior with lower and upper limits.

Table 3.3 Sieve analysis of fine aggregate*

Sieve Size (mm)	Passing %	Limits of the Iraqi specifications No. 45/1984 % Passing (Zone 2)
10	100	100
4.75	95	90-100
2.36	80	75-100
1.18	67	55-90
0.6	42	35-59
0.3	15	8-30
0.15	2	0-10
Pan	----	---

* Physical properties were carried out in the Engineering Syndicate of Iraq, Branch of AL-Anbar – Construction laboratory

Table 3.4 Physical and chemical properties of fine aggregate*

Physical properties	Test result	Limit of Iraqi Specifications No.45/1984
Sulfate content (SO_3%)	0.22%	0.5% (max)
Absorption%	0.64%	-
Fineness modulus	2.97	-

* Physical properties were carried out in the Engineering Syndicate of Iraq, Branch of AL-Anbar – Construction laboratory

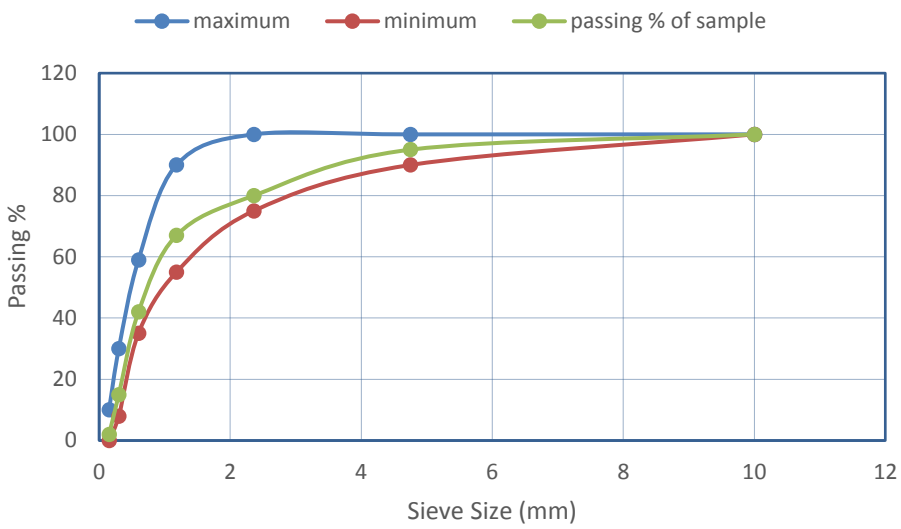


Figure 3.2 Sieve analysis for fine aggregate

3.2.3 Coarse Aggregate

Maximum size of using crushed coarse aggregate was 10 mm, so that the concrete could slip smoothly through molding, and there was no intersection with reinforcement and full cover will work. Crushed natural gravel was washed and submerged in to water for 24 hours and dried in air to get Saturated Surface Dry(SSD). Table 3.5 and Table 3.6 listed the sieve analysis of coarse aggregate, sulfate content, physical properties of coarse aggregate grading of this aggregate, and the limit specified by Iraqi Specification No. 45/1984. [66] The coarse aggregate was tested in the Engineering Syndicate of Iraq, Branch of AL-Anbar – Construction laboratory, Figure 3.3 shows the full sieve analysis behavior with lower and upper limits.

Table 3.5 Sieve analysis of coarse aggregate

Sieve Size (mm)	% Passing	Limit of Iraqi specification No. 45/1984
12.5	100	100
9.5	100	85-100
4.75	9.5	0-25
2.36	1	0-5

Table 3.6 Physical and chemical properties of coarse aggregate

Properties	Test results	Limits of (I.O.S.)No.45/1984
Specific gravity	2.65	-
Sulfate content	0.08	≤0.1
Absorption %	0.54%	-

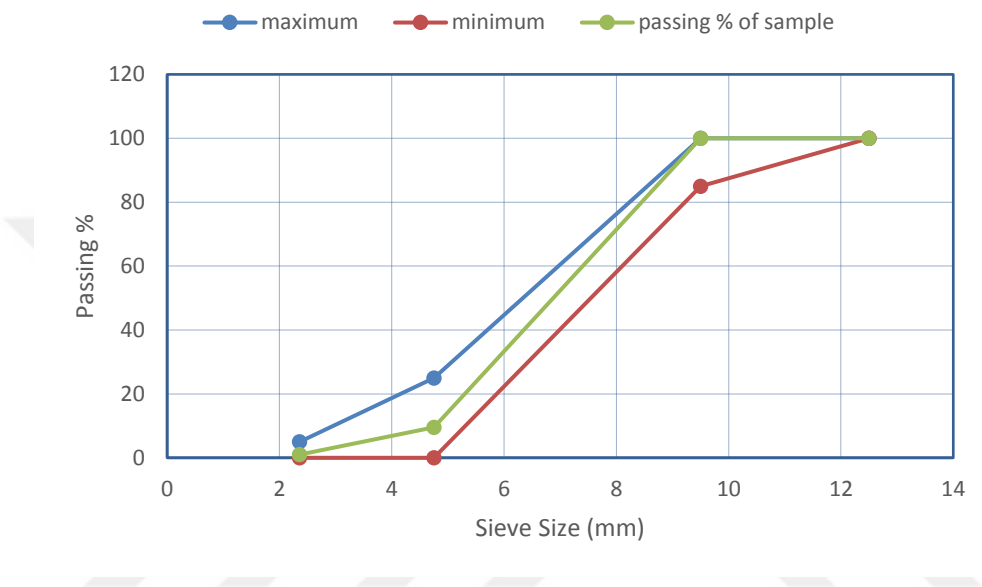


Figure 3.3 Sieve Analysis for Coarse Aggregate

3.2.4 Mixing and Curing Water

Ordinary tap water was used in this work in mixing and curing concrete without any additives. The water satisfied the requirements of Iraqi Specification No. 1703/ 1992. [67].

3.2.5 Epoxy

The epoxy was used to connect the stud to the steel plate. The amount of epoxy to connect each part was based on the instructions of suppliers.

3.2.6 Stud Connector

Three types of shear connectors were used, headed stud, channel and L-section. The headed stud and L-type connectors are commonly used in composite structure. The diameter of the stud is 6 mm with a height of 35 mm and a head

diameter of 10 mm as shown in Fig. 3.4.a. The mechanical properties of concrete are listed in Table 3.7 according to ASTM A416A [68] and ASTM A615-14 [69].

The dimensions of channel connector are shown in Fig. 3.4.c. The mechanical properties for the used steel were listed in Table 3.8 based on DIN 17100 ST44-2d [70].

Table 3.7 Mechanical properties of headed stud and L- shaped

Type	Yield Stress (MPa)	Ultimate strength (MPa)	Min. Elongation (%)
Ordinary	295	430	12.33
High strength	1160	1270	6.5

Table 3.8 Mechanical properties of Channel connector

Type	Thickness (mm)	Ultimate Strength (MPa)	Yield Stress (MPa)	Elongation (%)
Steel plate		433	495	16.95
DIN 17100 ST44-2 steel plate - mechanical properties				
t ≤ 16 mm		275	580 – 430	10
t > 16 mm		265	560 – 410	16

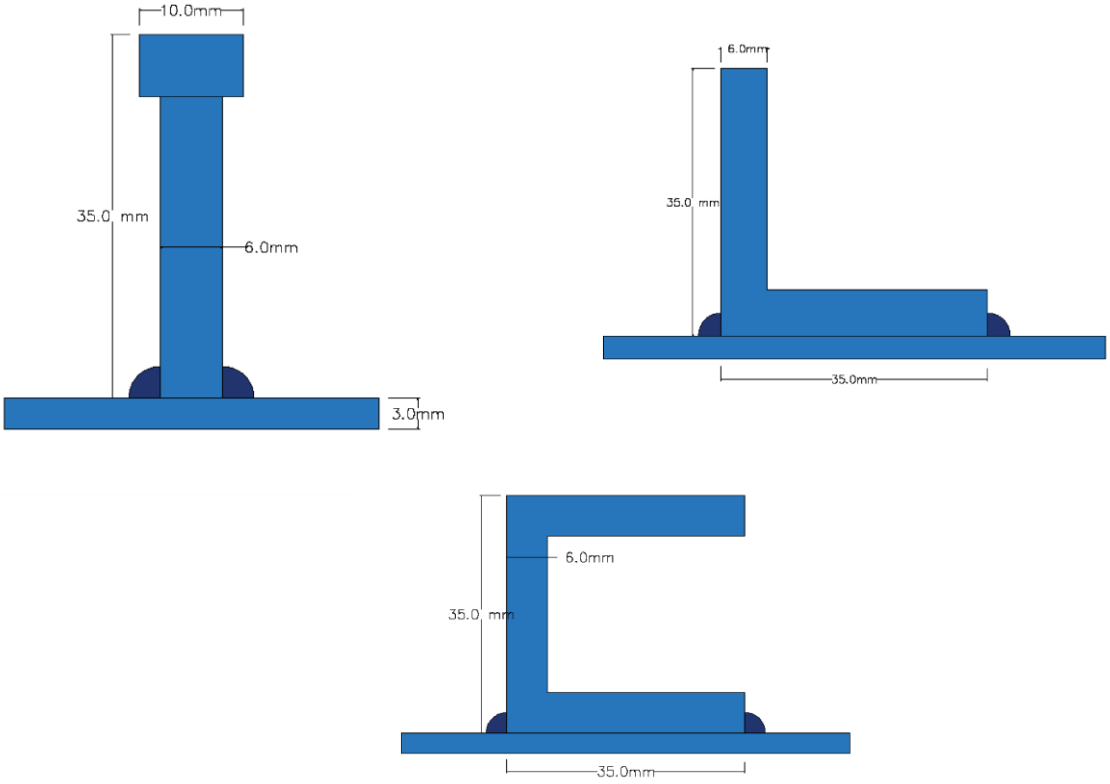


Figure 3.4 Shear Stud Connector Dimension

Figure 3.5 Manufacturing of shear connectors

3.2.7 Steel Reinforcement

BRC (British Reinforcement Company) is the type of reinforcement that was used to reinforce the RC composite slabs. The mechanical properties and Its geometry are listed in Table 3.9. The BRC was placed 20 mm away from the top. The test results show that steel satisfy the specifications ASTMA185-7A [71].

Table 3.9 BRC Geometry and mechanical properties*

Square opening (mm)	Average of Yield Tensile Strength (MPa)	Average of Ultimate Tensile Strength (MPa)	Average of the Tensile Modulus of Elasticity (MPa)	Elongation at Ultimate Stress (%)
90*90	520	725	210000	11
ASTMA185-7A	min450	min520	200000	min10

* Test carried out in Engineering Syndicate of Iraq, Branch of AL-Anbar – Construction laboratory

3.3 Materials for Reactive Powder Concrete

In recent years, reactive powder concrete (RPC) has provided a good use as composite materials in various structural applications such as, bridges, high rise structures, and nuclear power plants., which is also called ultra-high performance concrete (UHPC).

In general, the term RPC is used for a type of new cementitious material with the use of a very fine quartz sand (less than 0.6 mm). In this type of concrete, coarse aggregate is removed adding a very effective pozzolanic material is added including a particular quantity of micro steel fibers in the cementitious matrix to be able exhibit good tensile strength and performance.

Mechanical and physical properties are improved in RPC since the RPC is mainly based on a less defect of voids and micro-cracks by creating a homogeneous material. RPC enhance the ultimate load capacity of the construction members with a superior ductility, energy absorption, tensile strain-hardening behavior, crack control capability and durability that provide a greater structural reliability in compared to traditional concrete (RC) or fiber reinforced concrete (FRC) structures, Maha et al 2018. [72].

3.3.1 Material

The concrete mixes used in the study is Reactive Powder Concrete (RPC). Usually, chemical admixtures are employed to obtain a low water–cement (w/c) ratio with acceptable workability. The following is a brief description of the materials used in this concrete.

3.3.2 Fine Aggregate

Very fine sand with maximum size 600μm is used for RPC Mixes. This type of sand comes from the Sika company in Ireland. The grading of used fine sand zone 4 conforms to the Iraqi specification (IQS) No.45/1984 [66]. Table 3.10.

Table 3.10 Grading of the fine sand compared with the requirements of iraqi specification no.45/1984 [66]

Sieve Size (mm)	Cumulative Passing%	Limits of Iraqi specification No.45/1984
10	100	100
4.75	100	95-100
2.36	100	95-100
1.18	100	90-100
0.60	100	80-100
0.30	47	15-50
0.15	12	0-15

3.3.3 Silica fume

Silica fume is a highly reactive material that is used in relatively small amounts to enhance the properties of concrete. This type of silica fume is produced by the Sika company.

The American Concrete Institute, ACI 234R-06 [73] defined silica fume as "very fine non-crystalline silica produced in electric arc furnaces for silicon or alloys containing silicon". It is usually a grey coloured powder, somewhat similar to powder cement or some fly ashes. Silica fume is available as a dandified powder or in a water slurry form. It is generally used at a percentage by mass of cementations materials.

Adding to concrete mixture, the silica fume brings millions of very small particles and it fills in the spaces between cement grains. This is referred to as particle packing or micro-filling. The micro-filler effect would bring significant improvements in the nature of concrete even if silica fume does not react chemically. The Portland cement in concrete releases calcium hydroxide, which begins to react chemically. The silica fume (SiO2) forms additional binder material calcium silicate hydrate (C-S-H), due to reaction with this calcium hydroxide, which is very similar to the calcium silicate hydrate formed from the Portland cement. In this study, silica fume has 20% cement mass for (RPC). The chemical composition of this silica fume conforms to the ASTM C 1240-04 [74]. (Table 3.11).

Table 3.11 Chemical composition of silica fume

Compound Composition	Chemical Composition	Oxide Content (%)	Limit of Specification Requirement (ASTM C 1240)
Lime	CaO	0.5	——
Iron oxide	Fe2O3	1.4	——
Alumina	Al2O3	0.5	——
Silica	SiO2	92.1	85(min)
Magnesia	MgO	0.3	——
Sulphate	SO3	0.1	——
Potassium oxide	K2O	0.7	——
Sodium oxide	Na2O	0.3	——
Loss on ignition	L.O.I	2.8	6 (max)

*Manufacturer Properties

3.3.4 High Range Water Reducing Admixture (HRWR)

For this study, superplasticizer (high range water reducing agent HRWRA) based on polycarboxylic ether is used. One of the new generation of polymer-based superplasticizer Glenium 51 is used. The normal amount for Glenium 51 is (0.5 to 0.8) L/100kg of cement mass. BASF Construction Chemicals, manufactured Glenium 51. It is free from chlorides and complies with ASTM C494/C494M-01 (75). Table 3.12 shows the properties of Glenium 51. The use of Glenium 51 is necessary especially in cold weather because it works as an accelerator for hydration of cement.

Table 3.12 Typical properties of Glenium 51

No.	Property action	Concrete Superplasticizer
1	Color	Light brown
2	PH. Value	6.6
3	Form	Viscous liquid
4	Chlorides	Free of chlorides
5	Relative density	1.08 – 1.15 gm/cm3@ 25°C
6	Viscosity	128 ± 30 cps @ 20°C

3.3.5 Steel Fibers

Hooked ends steel fibers were used in this work with volume fractions of 1%. Sika company manufactured this type of steel fibers. The properties of the used steel fibers are given in Table 3.13.

Figure 3.6 Hooked ends steel fibers

Table 3.13 Properties of steel fibers

Property	Specifications
Relative density	7850 kg /m^3
Yield strength	1150 MPa
Modulus of elasticity	200x103 MPa
Strain at portion limit	5640x10^{-6}
Poisson's ratio	0.27
Average length	30 mm
Nominal diameter	0.4 mm
Aspect ratio	75

*Manufacturer Properties

3.4 Mix Design

Three types of concrete were used. Normal weight strength concrete with normal compressive strength of 25 MPa, high strength concrete with 85 MPa compressive strength and reactive powder concrete with 120 MPa compressive strength, all at 28 days. The details were listed in Table 3.14, for ordinary and high strength concrete

In the most basic form, reactive powder concrete contains a high content of Portland cement as main cementations material about (1000 kg/m3 of concrete), besides silica fume as a second supplementary cementations component. The superplasticizer has been used to give flowable concrete in an appropriate ratio. Also, steel fibers are added to improve its properties.

The mixture was designed based on the results obtained in the previous researchs. Maintain the same mixing ratios (see table 3.14).

Table 3.14 Concrete mix

Type of concrete	Cement kg/m^3	kg/m^3		Gravel kg/m^3	Silica fume* %	Silica fume kg/m^3	w/c	RWR** (L/m3)		Steel fiber content *** %	Steel fiber content kg/m^3
		Normal sand	Glass sand					G.	S.P		
Normal weight concrete, NWC	300	880	---	1100	---	---	0.4	---	----	----	----
High strength concrete, HSC	475	753	---	1025	----	----	0.3	---	----	----	---
PRC	1000	----	1000	----	20	200	0.2	---	5	1	78
No fine concrete	300	-----	-----	850	----	-----	0.4	---	----	------	-----

*Percent of weight cement.
** Superplasticizer.
*** Percent of mix volume.

3.5 Concrete Mixing Procedure

The preparation of specimen preparation and mixing procedure are described below:

i. Before mixing, all quantities were weighed and packed in a clean container and the moulds were oiled one day before casting.

ii. For ordinary and high strength concrete, the aggregates were prepared and immerged in water for 24 hours. After this stage, the aggregates were dried to form aggregate with saturated surface dry. Then, the materials were mixed in suitable mixers to obtain the required concrete.

iii. For all mixing, the silica fume and cement were mixed in dry state for about 5 minutes to disperse the silica fume particles throughout the cement particles, then the sand was added and the mixing continued for another (10 minutes). After this stage, the superplasticizer was dissolved in the water and the solution of water and superplasticizer was added to the rotary mixer and the whole mix ingredients were mixed for a sufficient time.

For the portions not reached by the blades of the mixer especially, the mixer was stopped and mixing was continued manually. Then, the mixer was operated for 5 minutes to gain reasonable fluidity. Fibers were uniformly distributed into the mix slowly in 5 minutes during mixing process, and then the mixing process continued for additional 2 to 3 minutes. In total, the time mixing requires 25 to 30 minutes of one batch. The mix details are given in Table 3.15.

Table 3.15 Mix details

Sample	Type of concrete	Type of shear connectors	Type of steel	Type of connection
OHOW	Ordinary	Headed stud	Ordinary strength	Welding
OHOE				Epoxy
OHHW			High strength	Welding
OHHE				Epoxy
OLOW		L- shaped	Ordinary strength	Welding
OLOE				Epoxy
OLHW			High strength	Welding
OLHE				Epoxy
OCOW		Channel	Ordinary strength	Welding
OCOE				Epoxy
OCHW			High strength	Welding
OCHE				Epoxy
HHOW	High strength	Headed stud	Ordinary strength	Welding

HHOE				Epoxy
HHHW			High strength	Welding
HHHE				Epoxy
HLOW		L- shaped	Ordinary strength	Welding
HLOE				Epoxy
HLHW			High strength	Welding
HLHE				Epoxy
HCOW		Channel	Ordinary strength	Welding
HCOE				Epoxy
HCHW			High strength	Welding
HCHE				Epoxy
RHOW	Reactive powder concrete	Headed stud	Ordinary strength	Welding
RHOE				Epoxy
RHHW			High strength	Welding
RHHE				Epoxy
RLOW		L- shaped	Ordinary strength	Welding
RLOE				Epoxy
RLHW			High strength	Welding
RLHE				Epoxy
RCOW		Channel	Ordinary strength	Welding
RCOE				Epoxy
RCHW			High strength	Welding
RCHE				Epoxy

In Table 3.15, the following symbols are used for simplification: The first letter ”O”, “H”, “R” are Types of concrete, the second letter “H”, “L”, “C”, are Types of shear connectors, the third letter “O”, “H” are types of steel for shear connectors, and the fourth letter “W”, “E” are types of connectors (welded or epoxy).

3.6 Casting and Compaction

Wood frames were fixed on to the steel plate. Compactions were made using mechanical vibrator to get more uniform mixing and to prevent any voids inside concrete. Steel trowel was used to level the top surface of slabs also to smooth it. Fig. (3.7).

Figure 3.7 Specimens were casted to mechanical properties testing.

3.7 Curing

All specimens prior to their testing were cured in water tank that contained fresh water. The specimens were kept under water to a specific and required time to make the concrete complete the hydration processes (see Fig. 3.8). The tank was under temperature control maintaining a specific value of 25±5 C^{o}.

Figure 3.8 Curing of the specimens in water tank

3.8 Fresh and Harden Concrete Specimens Tests

3.8.1 Slump Test

The slump tests were completed before any other tests were conducted such as; compressive strength, tensile strength or slab specimens to ensure the workability due to the varying percentages of material that was added to the concrete according to ASTM C143 / C143M - 5a [76]. Table 3.16 shows the slump test results, and Fig. 3.9 shows the photos from the slump test method.

Table 3.16 Slump test results

Type of concrete	Slump (mm)
Normal weight concrete (NWC)	50
High strength concrete (HSC)	70
Reactive powder concrete (RPC)	Collapse

Figure 3.9 Slump Test Method

3.8.2 Unit Weight (Density)

Density was found based on the weight of specimen that was divided to its volume. The density ranged according to the type of concrete according to ASTM C138 / C138M – 01a [77]. Table 3.17 has the list of values for density of all tested specimens according to the different types of concrete.

Table 3.17 Density for all tested specimens at 28 days

Concrete type	Density (kg/m3)
Normal weight concrete (NWC)	2430
High strength concrete (HSC)	2524
Reactive powder concrete (RPC)	2550
No Fine Concrete (NFC)	2000

3.8.3 Compressive Strength

The molds that were used to determine the compressive strength were cube with a dimension of 150x150x150 mm, which complies with the requirements of 1881 - 116- 1983 [78], and cylinder with 300 mm in height and 150 mm in diameter according to ASTM C39-86 [79] . The machine used in tests with capacity 2000 kN for compression, as shown in Figure 3.10. Three specimens were tested for each type of mix for cube and cylinder average value was taken.

Figure 3.10 Test specimen for cube and cylinder mixes

3.8.4 Splitting Tensile Strength

Splitting tensile strength was tested using traditional testing methods, which complies with according to ASTM C496-04 [80]. The calculation for all specimens to determine the splitting tensile strength depends on the breaking load, diameter of specimen and its length.

Based on ACI – 318- 2014 Code [81], the splitting tensile strength is calculated by applying the formula in equation 3.1.

$$f_t = \frac{2P}{\pi dL} \tag{3.1}$$

Where,

ft : Splitting tensile strength (MPa)

P : Applied load (N)

d: Diameter of the specimen (mm)

L : Length of the specimen (mm)

3.8.5 Flexural Strength Test

Flexural strength or moduli of rupture test that define the stress in a concrete before yielding in flexure was conducted according to ASTM C 293 [82] (center-point loading). The dimensions of prism were 400x100x100 mm. The testing machine had a capacity of 100 KN, and it was used to determine the flexural strength of concrete in different type of concrete with different ages. Fig. 3.11 shows the specimens prior to their testing. Three specimens were tested for each type of mix. The flexural strength (modulus of rupture) is calculated using the following equation:

$$f_r = \frac{3PL}{2bd^2} \tag{3.2}$$

Where:

f_r: Flexural strength (MPa)

P: Maximum applied load through testing machine (N)

L: Span length of specimen (mm)

b: Average width of specimen (mm)

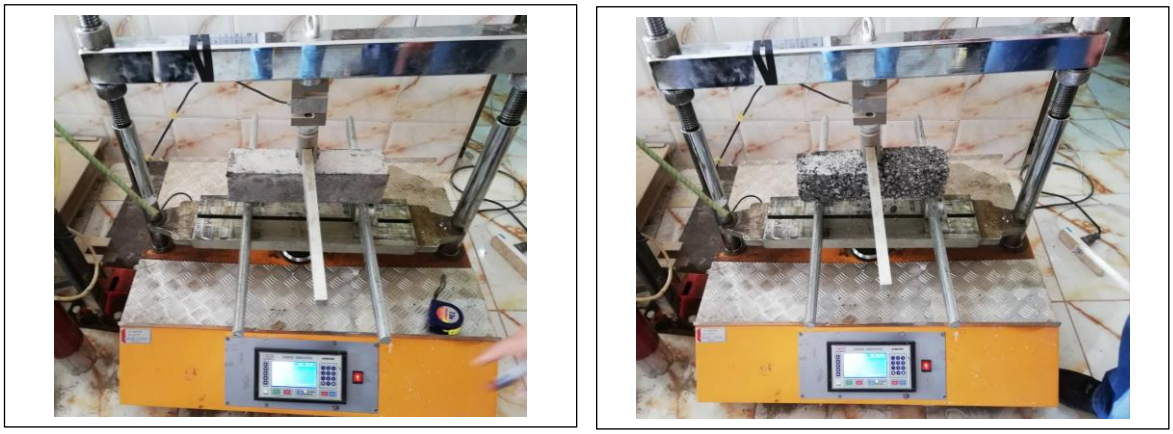


Figure 3.11 Flexural strength test

3.8.6 Modulus of Elasticity

The testing machine with a 2000kN capacity was used to determine the static modulus of elasticity see Figure 3.12. Modulus of elasticity was determined according to ASTM C469–02 [83] at 40% of ultimate load. Three specimens were tested for each type of mix. This test was conducted in the laboratory of the Engineering Syndicate of Iraq , Branch of AL-Anbar.

Figure 3.12 Static modulus of elasticity test

3.8.7 Push – Out Tests

Push-out test specimens were generated according to Euro code 4 [84]. simple push-out tests were used to determined the shear connectors properties, where two small walls were linked to the flanges of an I section by four studs located on each side. The walls were bedded onto the lower part of a compression-testing

machine and the load was applied gradually to the top end of the steel section. Then obtained a load-slip curve and the stiffness of shear connector were obtained.

The push-out part was fabricated according to the dimensions of the used studs as shown in Figure 3.13, then cast and cured in water for 28 days. Hydraulic machine with 2000 KN capacity was used to test the push out specimens at the laboratory of the Engineering Syndicate of Iraq, Branch of AL-Anbar (see Figure 3.14). The results of load-slip relationship were recorded and are presented in chapter four.

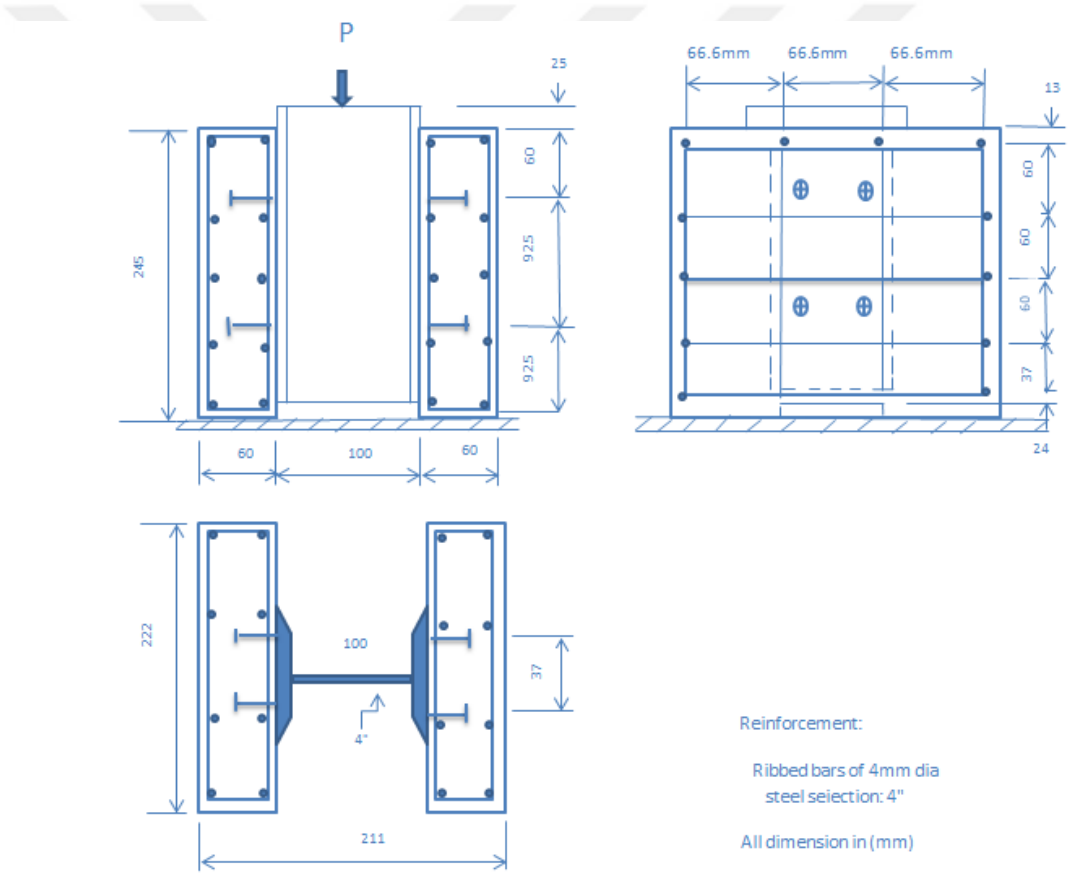


Figure 3.13 Push- out test specimen

Figure 3.14 Fabrication, casting and testing of push-out segments

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSION

4.1. Introduction

The main objective of the current research work is to investigate the structural behavior of different types of shear connectors embedded in different types of concrete. For this purpose, high strength concrete, reactive powder concrete, and no fine aggregate concrete were used with normal weight concrete, which was as a reference. Also, different types of shear connectors including headed stud, L-shaped, Channel shear connectors were tested. Welding and epoxy were used as a bond between the steel plate and shear connectors.

The mechanical properties of different types of concrete were investigated in this chapter, including compressive strength, splitting tensile strength, modulus of rapture, modulus of elasticity and density. Total number of 30 push-out specimens were tested to investigate the behavior of shear connectors mainly the load-slip relations.

The results of tests which described in chapter three are presented and discussed. In this chapter the results are divided into the following sections:

A. Mechanical Properties of Different Types of Concrete

1. Cube Compressive Strength (fcu)
2. Splitting Tensile Strength (ft)
3. Modulus of rapture (fr)
4. Modulus of Elasticity (Ec)
5. Density

B. Tested Push-Out Specimens

1. Load –slip relations
2. Crack pattern

4.2 Mechanical Properties of Different Types of Concrete

The properties of different types of concrete used in the present work are obtained from the tests as described in the following sections.

4.2.1 Compressive Strength

The percentage difference in compressive strength of different types of concrete compared to normal concrete is given in Table 4.1. Figures 4.1, 4.2 , 4.3 and 4.4 shows the variation of compressive strength developed at different ages.

Table 4.1 Compressive strength for different types of concrete with time

Type of concrete	Age (Days)	Compressive strength (MPa)	Change in Compressive strength Cylinder with respect to reference mix(%)
M1 (NWC) Reference	7	17.76	0
	14	22.96	0
	21	23.99	0
	28	26.77	0
	56	26.31	0
M2 (HSC)	7	55.44	212.16%
	14	66.33	188.89%
	21	82.00	241.82%
	28	85	217.51%
	56	88.54	236.52%
M3 (RPC)	7	80.76	354.72%
	14	97.76	325.78%
	21	110	358.52%
	28	120	348.26%
	56	122.65	366.17%
M4 (NFC)	7	11.01	-38.00%
	14	14.87	-35.23%
	21	16.54	-31.05%
	28	19.65	-26.59%
	56	19.87	-24.47%

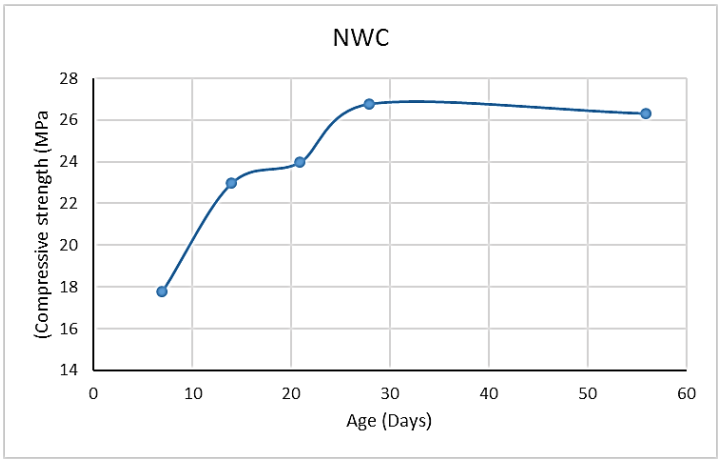


Figure 4.1 Compressive Strength for Normal Weight Concrete, NWC

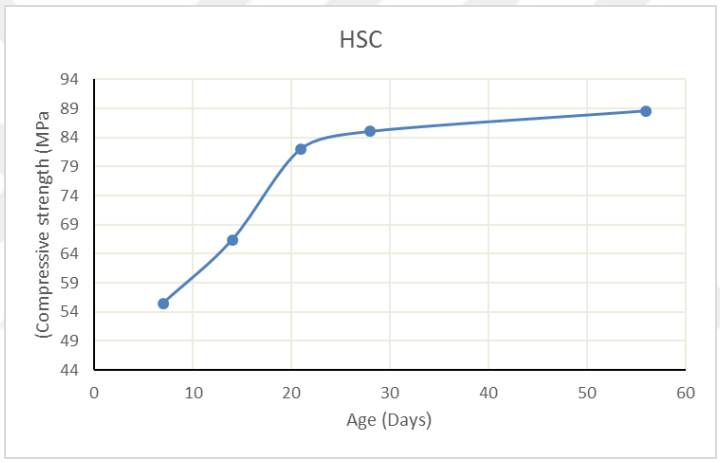


Figure 4.2 Compressive strength for high strength concrete, HSC

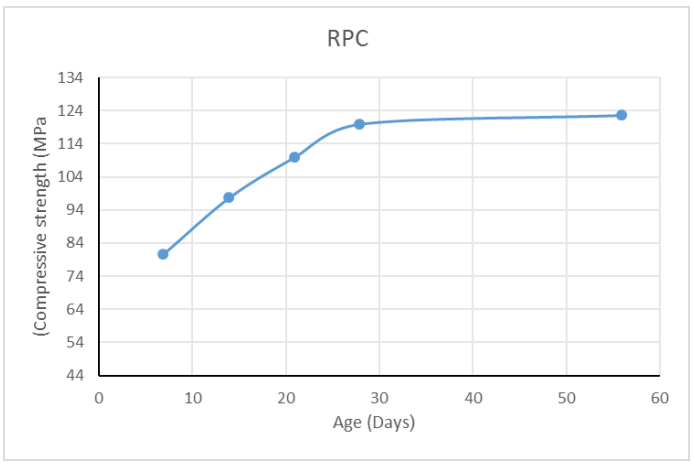


Figure 4.3 Compressive strength for rpc concrete, RPC

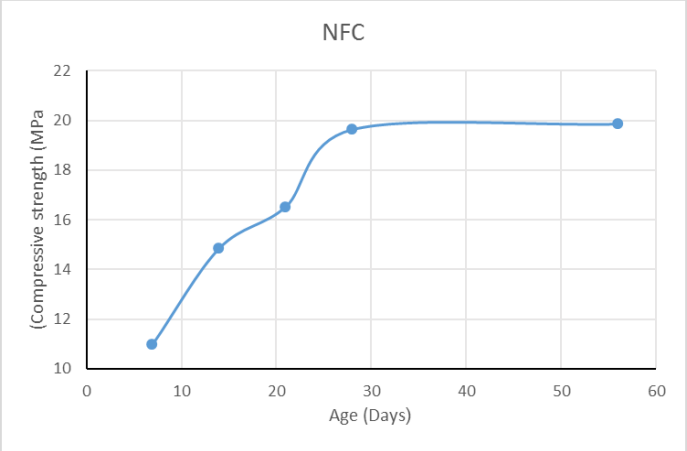


Figure 4.4 Compressive strength for no fine concrete, NFC

Based on the results above, one can see that in Figure 4.1. The highest concrete resistance was 27 MPa at 28 days. Fig. 4.2 shows that concrete strength increases with age and reaches the highest resistance of 88 MPa. Figure 4.3 shows that the concrete resistance is very high as it increases with increasing age, also the results show that high strength can be obtained at early age. In Fig. 4.4, the strength does not even exceed 20 MPa.

4.2.2 Splitting Tensile Strength

The percentage difference in splitting tensile strength of different types of concrete compared with normal concrete as shown in Table 4.2. Figures 4.5, 4.6, 4.7 and 4.8 show the splitting tensile strength developed at different ages.

Table 4.2 Splitting tensile strength for different types of concrete with time

Type of concrete	Age (Days)	Splitting tensile strength (MPa)	Change in splitting tensile strength cylinder with respect to reference mix(%)
M1 (NWC) Reference	7	1.64	0
	14	1.96	0
	21	2.65	0
	28	3.1	0
	56	3.13	0
M2 (HSC)	7	5.46	232.93%
	14	7.45	280.10%
	21	9.55	260.37%

	28	11.6	274.19%
	56	11.75	275.39%
M3 (RPC)	7	7.54	359.75%
	14	9.65	392.34%
	21	11.6	337.73%
	28	14.21	358.38%
	56	14.23	354.63%
M4 (NFC)	7	1.36	-17.07%
	14	1.77	-9.69%
	21	1.98	-25.28%
	28	2.01	-35.16%
	56	2.02	-35.46%

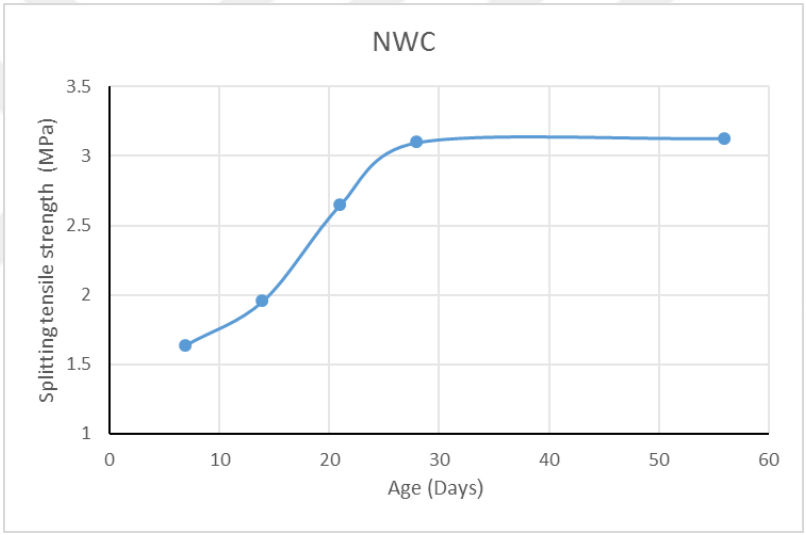


Figure 4.5 Splitting tensile strength for normal weight concrete, NWC

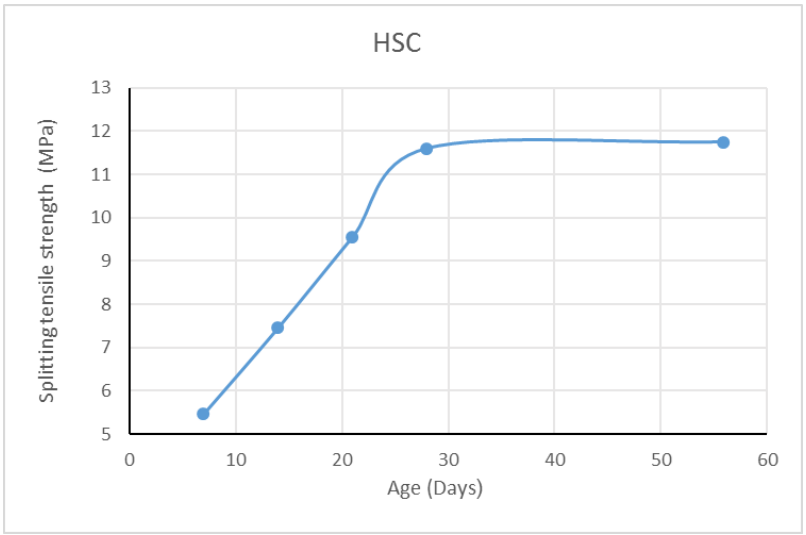


Figure 4.6 Splitting tensile strength for high strength concrete, HSC

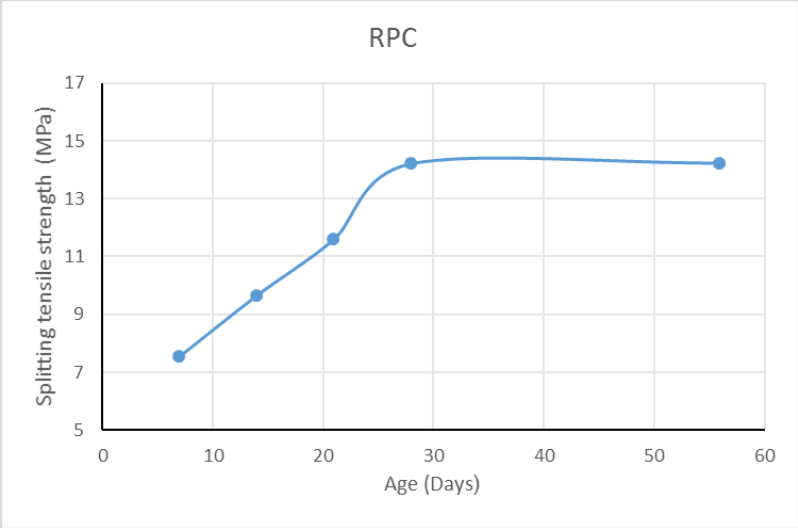


Figure 4.7 Splitting tensile strength for rpc concrete, RPC

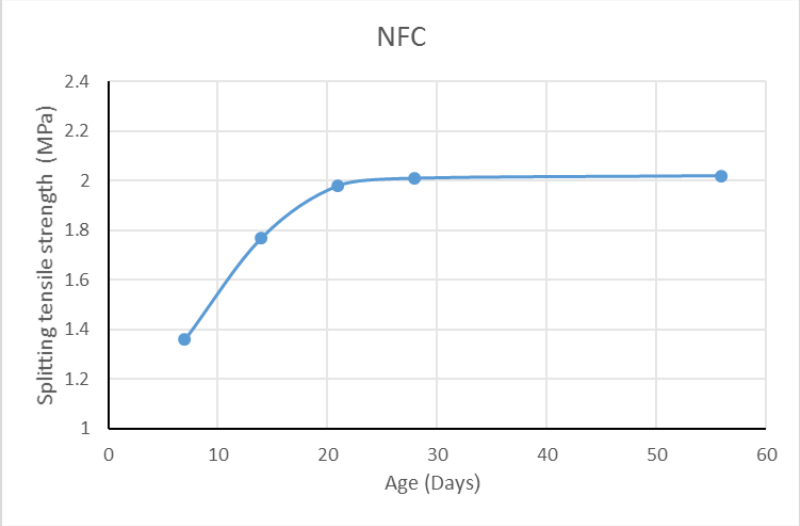


Figure 4.8 Splitting tensile strength for no fine concrete, NFC

Based on the values given in Table 4.2, the maximum change in splitting tensile strength at 14 days with respect to NWC for HSC and RPC was recorded as 280% and 392%, respectively, at 14 days. Also the results showed that high splitting tensile strength can be obtained by RPC. In Fig. 4.8, the splitting tensile strength did not exceed 2.1 MPa.

4.2.3 Modulus of Rupture

The percentage difference in modulus of rupture for different types of concrete compared to normal concrete as shown in Table 4.3. Figures 4.9, 4.10, 4.11 and 4.12 shows the modulus of rupture developed at different ages.

Table 4.3 Modulus of rupture strength for different types of concrete with time

Type of concrete	Age (Days)	Modulus of Rupture (MPa)	Change in Modulus of Rupture with respect to reference mix(%)
M1 (NWC) Reference	7	1.79	0
	14	2.12	0
	21	2.85	0
	28	3.4	0
	56	3.41	0
M2 (HSC)	7	5.88	228.49%
	14	7.85	270.28%
	21	9.98	250.17%
	28	11.9	250
	56	12.25	259.23%
M3 (RPC)	7	10.87	507.26%
	14	12.67	497.64%
	21	14.88	422.10%
	28	16.00	370.58%
	56	16.22	375.65%
M4 (NFC)	7	1.97	10.05%
	14	2.01	-5.18%
	21	2.51	-11.92%
	28	2.54	-25.29%
	56	2.66	-21.99%

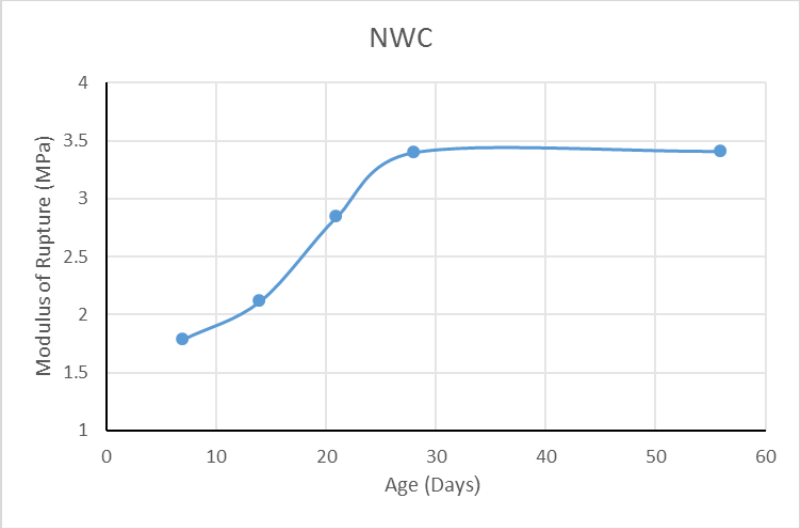


Figure 4.9 Modulus of rupture for normal weight concrete, NWC

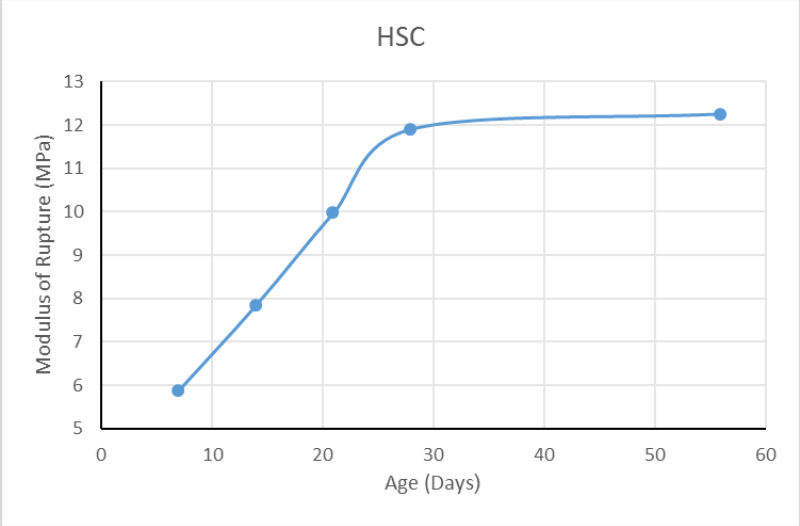


Figure 4.10 Modulus of rupture for high strength concrete, HSC

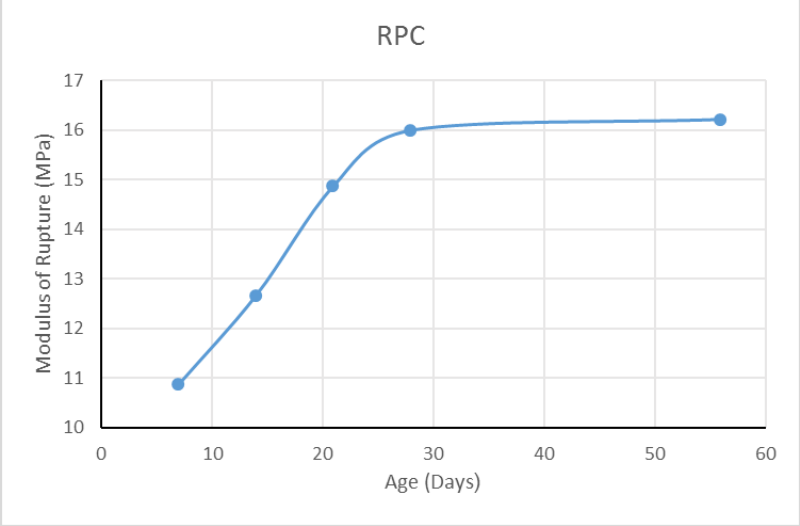


Figure 4.11 Modulus of rupture for RPC concrete

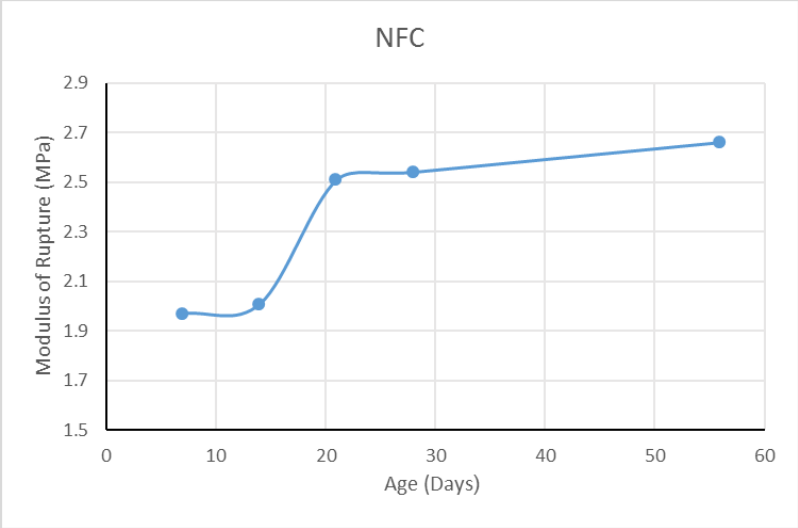


Figure 4.12 Modulus of rupture for no fine concrete, NFC

Based on the results listed in Table 4.3, the maximum change in modulus of rupture strength at 14 and 7 days with respect to NWC for HSC and RPC were 270% and 507%, respectively. Also, the results showed that high modulus of rupture strength can be obtained by RPC. In Fig. 4.12, low modulus of rupture strength was obtained by NFC, and it did not exceed 2.7 MPa

4.2.4 Static Modulus of Elasticity

The percentage difference in static modulus of elasticity for different types of concrete was compared to that of normal concrete shown in (see Table 4.4). Figures 4.13, 4.14, 4.15, and 4.16 show the static modulus of elasticity developed at different ages.

Table 4.4 Static modulus of elasticity for different types of concrete with time

Type of concrete	Age (Days)	Static Modulus of Elasticity (GPa)	Change in Static Modulus of Elasticity with respect to reference mix (%)
M1 (NWC) Reference	7	15.78	0
	14	21.66	0
	21	23.07	0
	28	24.65	0
	56	24.76	0
M2 (HSC)	7	20.47	29.72%
	14	22.41	3.46%

	21	30.33	31.47%
	28	33.22	34.77%
	56	33.42	34.98%
M3 (RPC)	7	22.33	41.51%
	14	27.54	27.15%
	21	35.33	53.14%
	28	43.44	76.23%
	56	43.46	75.53%
M4 (NFC)	7	12.44	-21.17%
	14	15.32	-29.27%
	21	16.65	-27.83%
	28	20.98	-14.89%
	56	20.99	-15.23%

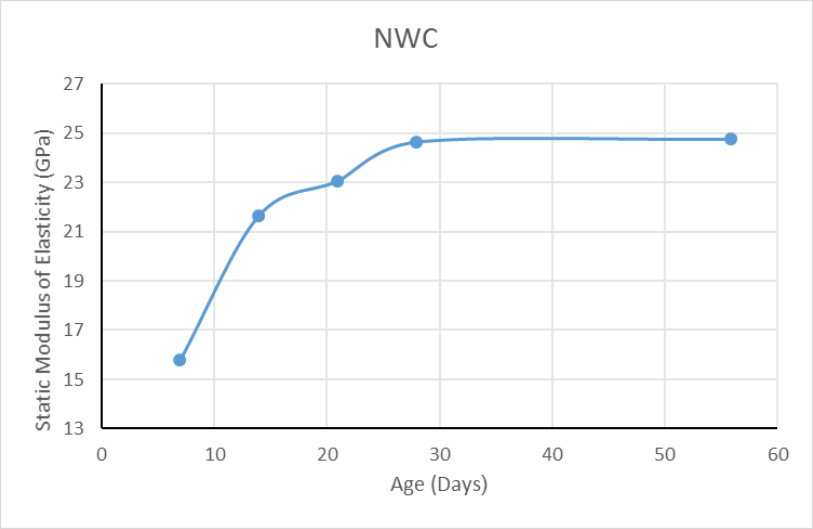


Figure 4.13 Static modulus of elasticity for normal weight concrete, NWC

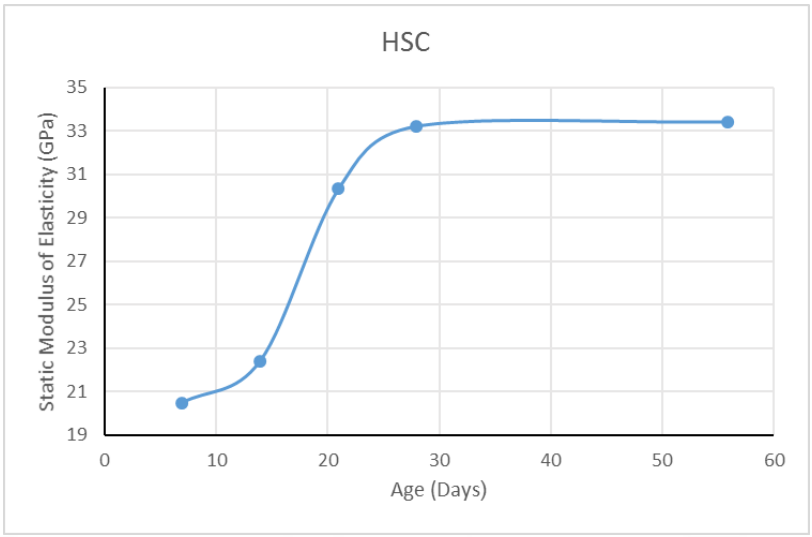


Figure 4.14 Static modulus of elasticity for high strength concrete, HSC

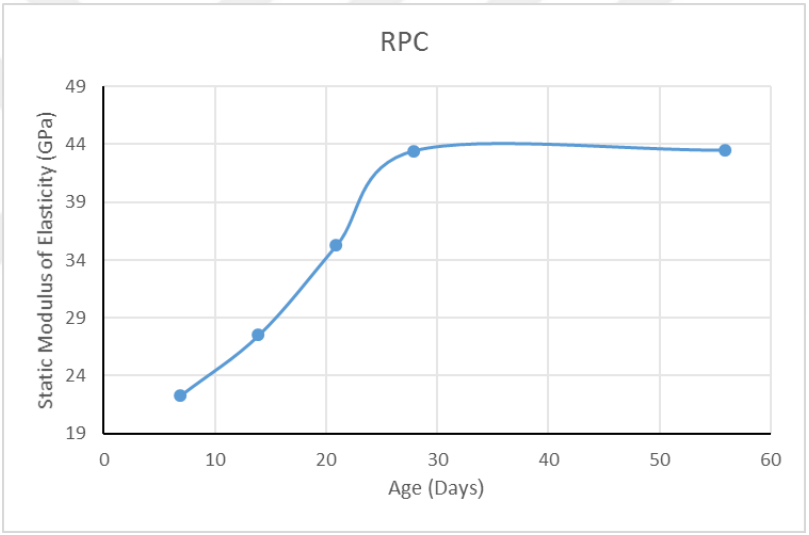


Figure 4.15 Static modulus of elasticity for rpc concrete, RPC

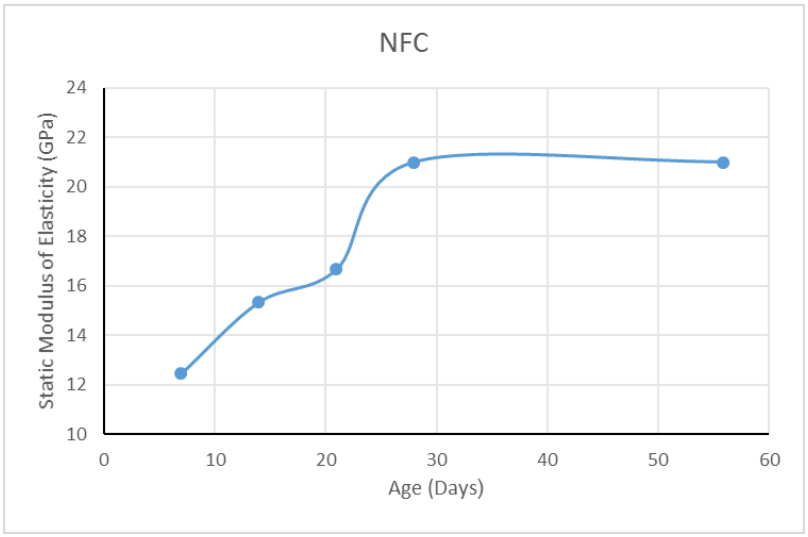


Figure 4.16 Static modulus of elasticity for no fine concrete, NFC

According to the results listed in Table 4.4, the maximum changes in static modulus of elasticity after 28 days with respect to NWC for HSC and RPC were around 35% and 76%, respectively. In addition, the results showed that high static modulus of elasticity was obtained with RPC. while the law one was obtained with NFC. In Fig. 4.16. The static modulus of elasticity for NFC did not exceed 21 MPa.

4.3 Push – Out Tests

Push-out test specimen was produced according to the requirements in Euro code 4 [90]. The shear connectors properties were determined experimentally through simple push-out tests where two small slabs were linked to the flanges of an I section by four studs on each side. The slabs were bedded onto the lower part of a compression-testing machine, and the load was applied gradually to the top end of the steel section. A load-slip curve and then after the stiffness of shear connector obtained.

The push-out segments were fabricated according to the dimensions of the used studs as shown previously in Figure 3.4. Then they were cast and cured in water for 28 days. Hydraulic testing machine with a capacity of 2000 kN was used to test the push out segments at the Construction Laboratory of the Engineering Syndicate of Iraq Branch of Al-Anbar shown (see Figure 3.14. The results of load-slip relationship were recorded and presented in this chapter.

4.3.1 Effect of Shear Stiffness

4.3.1.1 Normal Weight Concrete

The test results for normal weight concrete and different types of connectors are shown in Table 4.1. Figures 4.17, 4.18, and 4.19 illustrate the variation shear forces versus slip different shear connectors which have different type of steel properties and bonds in an ordinary strength concrete.

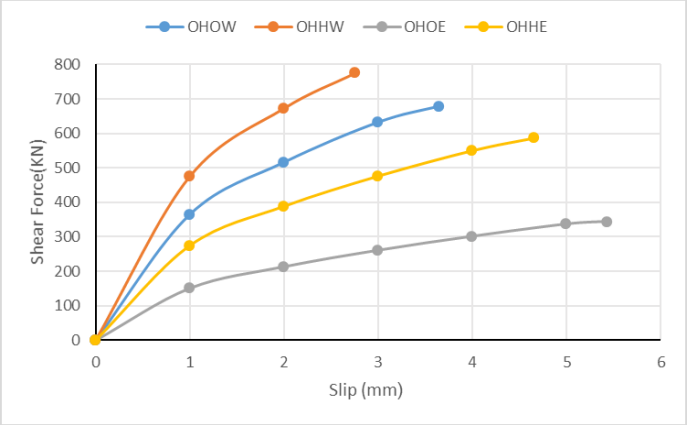


Figure 4.17 Headed stud shear connector with different type of steel properties and bonds in ordinary strength concrete

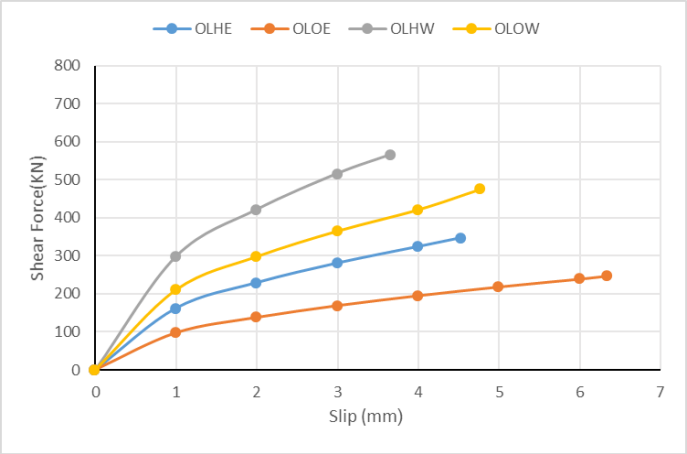


Figure 4.18 L- shaped shear connector with different type of steel properties and bonds in ordinary strength concrete

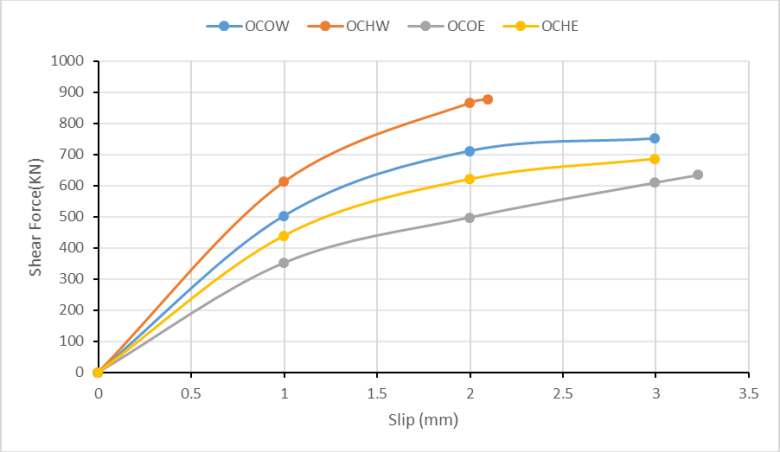


Figure 4.19 C- shaped shear connector with different type of steel properties and bonds for ordinary strength concrete

Table 4.5 The test results for push-out test in normal weight concrete

Symbol*	Type of shear connector	Type of steel properties	Type of bond	Ultimate shear force kN	Max. slip mm	Shear stiffness kN/mm	Percentage of difference %	Concrete crack width mm
OHOW	Headed	Ordinary	Welding	679	3.65	93.01	0	0.1
OHHW	Headed	High	Welding	776	2.76	140.7	0	0.04
OHOE	Headed	Ordinary	Epoxy	345	5.43	31.76	0	0.32
OHHE	Headed	High	Epoxy	587	4.66	62.98	0	0.19
OLOW	L- shaped	Ordinary	Welding	476	4.77	49.89	-29.9	0.11
OLHW	L- shaped	High	Welding	566	3.66	77.34	-27.06	0.09
OLOE	L- shaped	Ordinary	Epoxy	248	6.34	19.56	-28.12	0.39
OLHE	L- shaped	High	Epoxy	347	4.53	38.30	-40.89	0.22
OCOW	channel	Ordinary	Welding	751	2.99	125.58	10.6	0.08
OCHW	channel	High	Welding	876	2.09	209.56	12.89	0.03
OCOE	channel	Ordinary	Epoxy	634	3.23	98.14	83.77	0.11
OCHE	channel	High	Epoxy	686	3.01	113.95	16.87	0.09

*The symbols I Table 4.5 are used for simplification: The first letter ”O”, is Type of concrete, the second letter “H”, “L”, “C”, are Types of shear connectors, the third letter “O”, “H” are types of steel for shear connectors, and the fourth letter “W”, “E” are types of connectors (welded or epoxy).

According to the data in Table 4.5 OCHW gave the best shear force value of 876 kN, with percentage of difference around 83.77% when compared to the value obtained from OHHW.

4.3.1.2 High Strength Concrete

The test results for high strength concrete and different types of connectors were shown in Table 4.6. Figures 4.20, 4.21, and 4.22 shows the different shear connectors with different type of steel properties and bonds for high strength concrete.

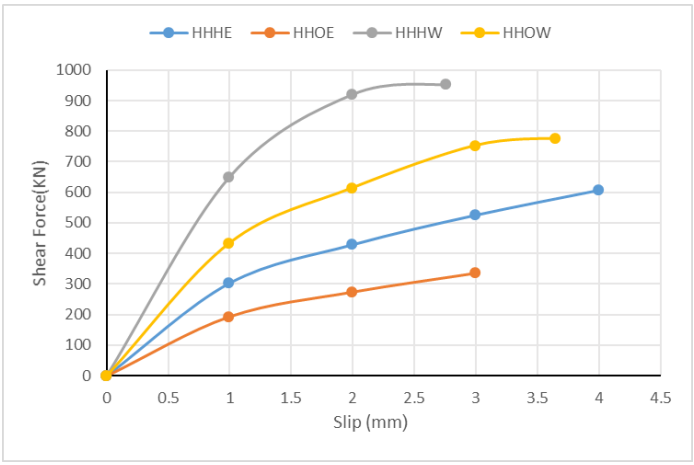


Figure 4.20 Headed stud shear connector with different type of steel properties and bonds for high strength concrete

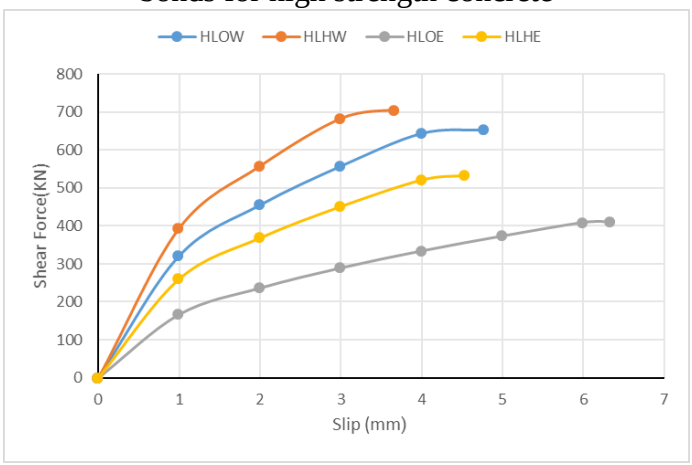


Figure 4.21 L- shaped stud shear connector with different type of steel properties and bonds for high strength concrete

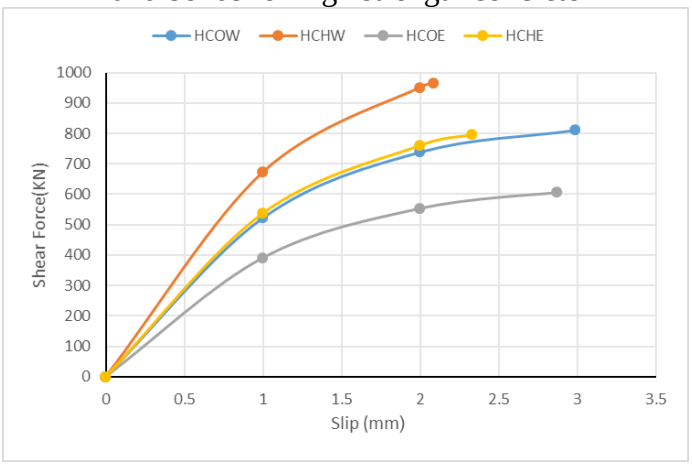


Figure 4.22 C- shaped stud shear connector with different type of steel properties and bonds for high strength concrete

Table 4.6 The test results for push-out test in high strength concrete

Symbol*	Type of shear connector	Type of steel properties	Type of bond	Ultimate shear force KN	Max. slip mm	Shear stiffness KN/mm	Percentage of difference %	Concrete crack width mm
HHOW	Headed stud	Ordinary strength	Welding	777	3.65	106.43	0	0.07
HHHW	Headed stud	High strength	Welding	954	2.76	172.82	0	0.03
HHOE	Headed stud	Ordinary strength	epoxy	453	5.43	41.71	0	0.21
HHHE	Headed stud	High strength	epoxy	690	4.66	74.03	0	0.12
HLOW	L-shaped	Ordinary strength	Welding	654	4.77	68.56	-15.8	0.09
HLHW	L-shaped	High strength	Welding	705	3.66	96.31	-26.1	0.07
HLOE	L-shaped	Ordinary strength	epoxy	411	6.34	32.41	-9.27	0.31
HLHE	L-shaped	High strength	epoxy	534	4.33	61.66	-22.6	0.21
HCOW	Channel	Ordinary strength	Welding	812	2.99	135.78	4.5	0.07
HCHW	Channel	High strength	Welding	966	2.09	231.10	1.3	0.03
HCOE	Channel	Ordinary strength	epoxy	607	2.87	105.74	34	0.09
HCHE	Channel	High strength	epoxy	797	2.33	171.03	15.5	0.08

* The symbols are used for simplification: The first letter “H” is type of concrete, the second letter “H”, “L”, “C”, are Types of shear connectors, the third letter “O”, “H” are types of steel for shear connectors, and the fourth letter “W”, “E” are types of connectors (welded or epoxy).

According to the test results, HCHW produced the best shear force of 966 kN with a percentage of difference of 1.26% when compared to the shear value obtained from HHHW.

4.3.1.3 Reactive Powder Concrete

The test results for reactive powder concrete and different types of connectors are shown in Table 4.3.

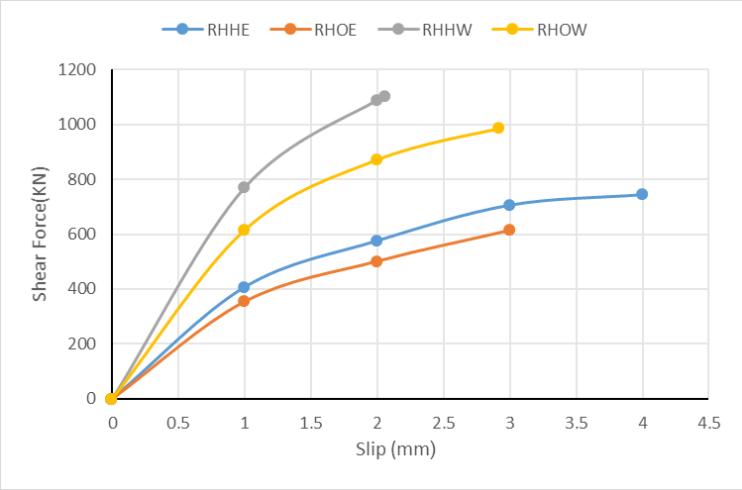


Figure 4.23 Headed stud shear connector with different type of steel properties and bonds for reactive powder concrete

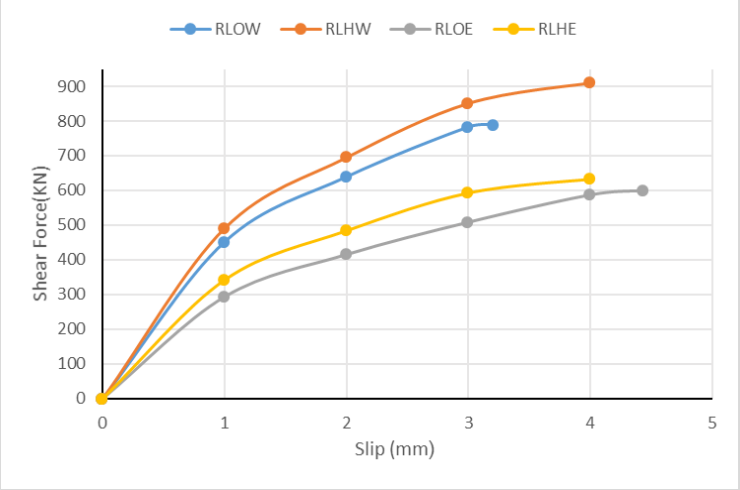


Figure 4.24 L- shaped shear connector with different type of steel properties and bonds for reactive powder concrete

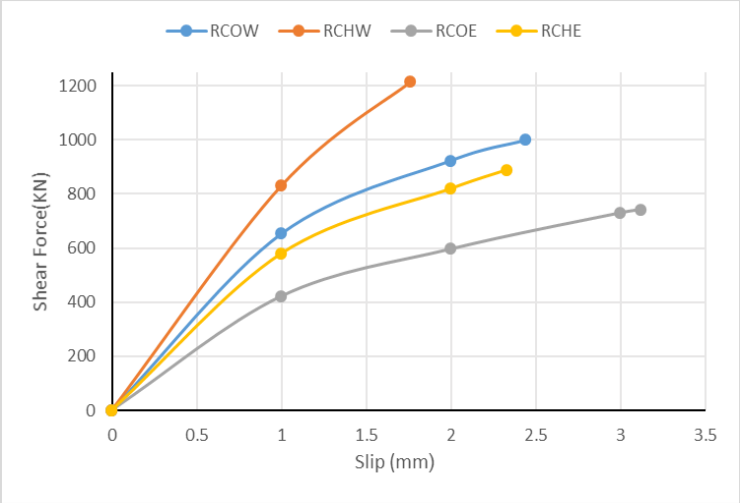


Figure 4.25 C- shaped shear connector with different type of steel properties and bonds for reactive powder concrete

Table 4.7 The test results for push-out test in reactive powder concrete

Symbol*	Type of shear connector	Type of steel properties	Type of bond	Ultimate shear force (kN)	Max. slip (mm)	Shear stiffness (kN/mm)	Percentage of difference (%)	Concrete crack width (mm)
RHOW	Headed stud	Ordinary strength	Welding	987	2.92	169.00	0	0.05
RHHW	Headed stud	High strength	Welding	1103	2.06	267.71	0	0.02
RHOE	Headed stud	Ordinary strength	Epoxy	665	3.11	106.92	0	0.18
RHHE	Headed stud	High strength	Epoxy	745	4.00	93.12	0	0.10
RLOW	L-Shaped	Ordinary strength	Welding	788	3.21	122.74	-20.16	0.08
RLHW	L-Shaped	High strength	Welding	911	3.99	114.16	-17.41	0.05
RLOE	L-Shaped	Ordinary strength	Epoxy	601	4.44	67.68	-9.62	0.21
RLHE	L-shaped	High strength	Epoxy	634	4.00	79.25	-14.90	0.11
RCOW	Channel	Ordinary strength	Welding	999	2.44	204.71	1.22	0.05
RCHW	Channel	High strength	Welding	1215	1.76	345.17	10.15	0.02
RCOE	Channel	Ordinary strength	Epoxy	743	3.12	119.09	11.73	0.06
RCHE	Channel	High strength	Epoxy	888	2.33	190.55	19.19	0.04

*The symbols are used for simplification: The first letter “R” is Type of concrete (reactive powder), the second letter “H”, “L”, “C”, are Types of shear connectors, the third letter “O”, “H” are types of steel for shear connectors, and the fourth letter “W”, “E” are types of connectors (welded or epoxy).

According to the test results, RCHW produced the best shear force value of 1215 kN, with a percentage of difference of 10.15% compared to the shear value obtained from RHHW.

The specimens with L-shaped connectors were more susceptible to concrete related failures because of a very high concentration of stresses within a smaller area of concrete, which caused maximum concrete width. In these specimens, the failure was caused by the crushing of the concrete contained below the L-shaped connectors. In general, the welding bond was better and gave higher shear force values with less slip.

4.3.2 Effect of Concrete Type

The load-slip curve for different concrete type is plotted in Figures 4.26, 4.27, and 4.28. In Fig. 4.26, it appears that the specimen with the least shear force value is OHOE and the strongest one with the maximum shear force value is RHHW. It was observed that the increase in shear force for RHHW with the same characteristics of shear connectors and bonds, was around 16% and 42% with respect to the shear force value of HHHW and OHHW, respectively as shown in figure 4.26.

The load-slip curve for RHHW fracture type of failure came to an abrupt end, whereas those related to concrete failure such as OHOE exhibited a gradual drop in load-slip failure.

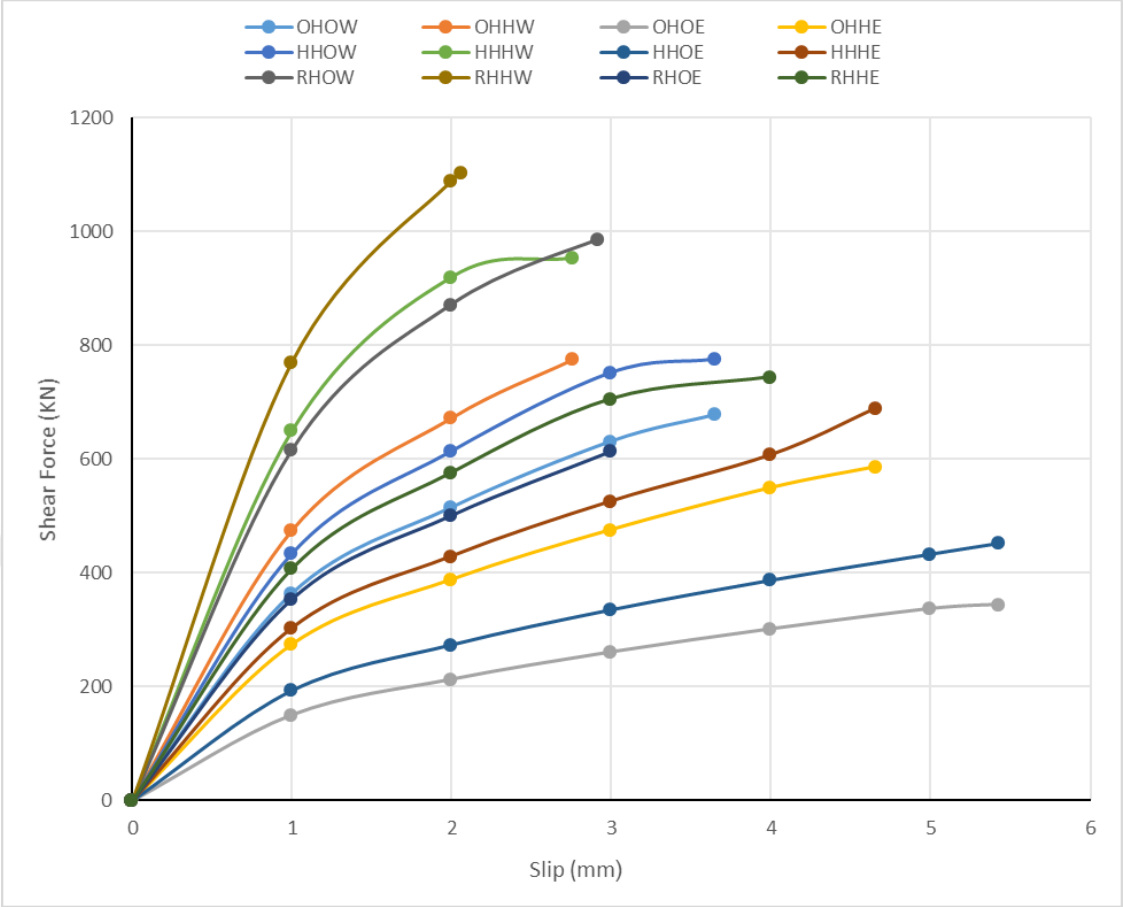


Figure 4.26 Load slip curve for different type of concrete with different type of bonds for headed stud shear connectors

In Figure 4.27, it appears that the weaker specimen in term of resisting the minimum shear force is OLOE and the strongest one with the maximum shear force is RLHW. It was observed that the increase in the shear force value for RLHW with same characteristics of shear connectors and bonds was around 29% and 61% when compared from HLHW and OLHW, respectively. Once again, the load-slip curve for RLHW fracture type of failure came to an abrupt end whereas those related to concrete failure such as OLOE exhibited a gradual drop.

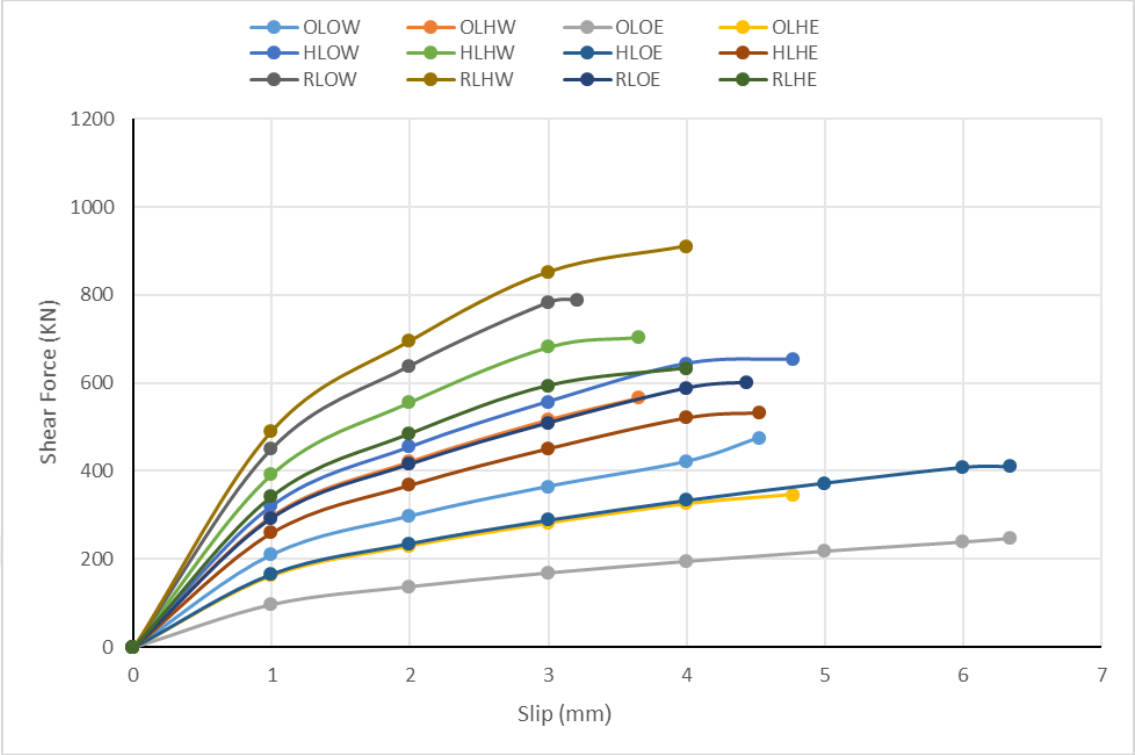


Figure 4.27 Load slip curve for different type of concrete with different type of bonds for l shaped shear connectors

Based on the results plotted in Figure 4.28, it appears that the weaker specimen with the minimum shear force values are OCOE and HCOE, while the strongest one with the minimum shear force is RCHW. It was observed that the increase in shear force for RCHW with same characteristics of shear connectors and bonds was around 26% and 39% with respect to the shear force values of HCHW and OCHW, respectively.

Once again, the load-slip curve for RLHW fracture type of failure came to an abrupt end, whereas those related to concrete failure OCOE and HCOE exhibited a gradual drop.

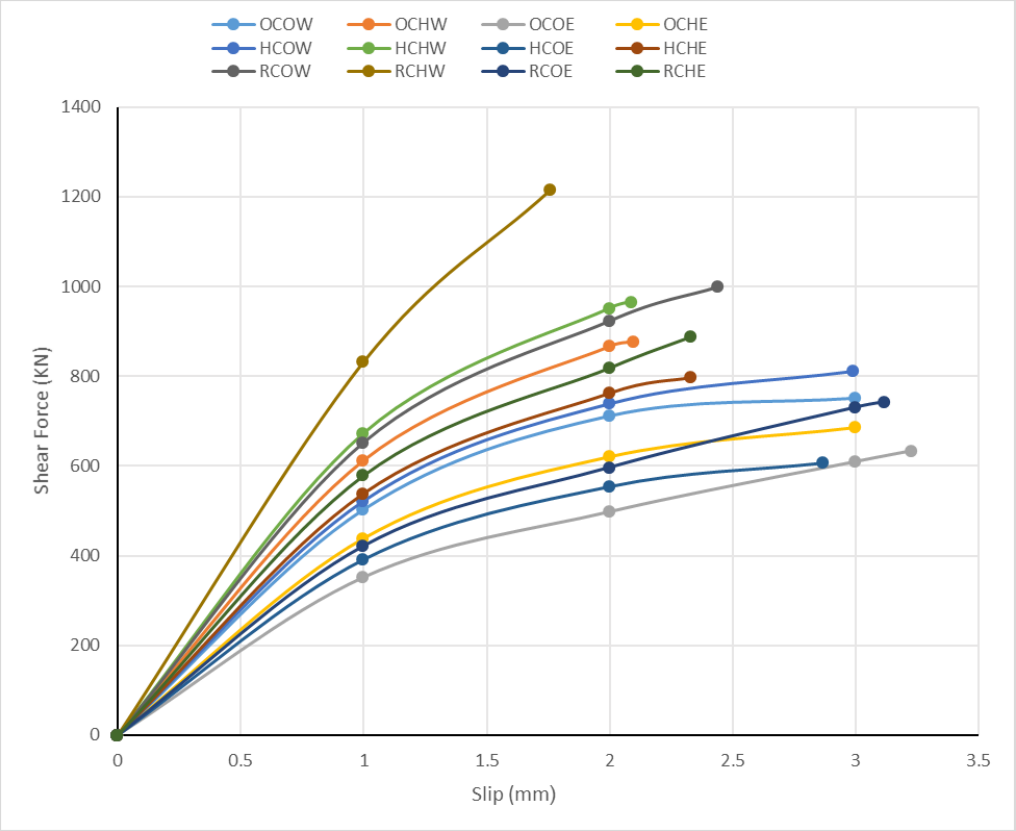


Figure 4.28 Load slip curve for different type of concrete with different type of bonds for C shape shear connectors

Based on the results from the three previous figures, it can be seen that the reactive powder concrete, RPC, generate the maximum higher results due to the increase in the compressive strength of concrete for RPC.

4.3.3 Effect of Bond Type

The load-slip curves for different bond types are presented in Figures 4.29, 4.30, and 4.31. Based on the data illustrated in Figure 4.29, it appears that the specimen with the minimum shear force capacity is OLOE and the one with the maximum shear capacity is OCHW. According to the test results, it was observed that increase in shear force for OCHW for same characteristics of shear connectors and types of concrete was around 17%, 38%, and 28% when compared to the shear forces from OCOW, OCOE, OCHE, respectively.

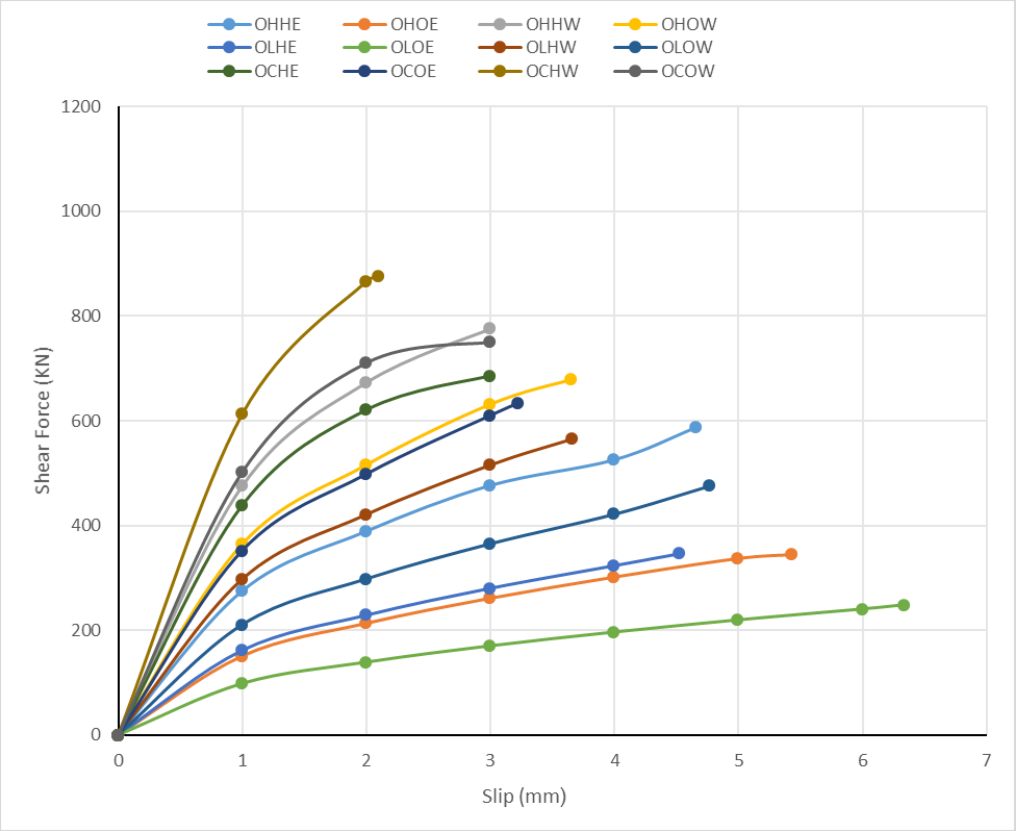


Figure 4.29 Load slip curve for different type of concrete with different type of bonds for ordinary concrete type

Based on the results plotted in Figure 4.30 It appears that the specimen that generates the minimum shear force is HLOE while the one that generates the maxium shear force is HCHW. Based on the test results, it was observed that the increase in shear force for HCHW with same characteristics of shear connectors and types of concrete was around 19%, 59%, and 21% when compared to the shear forces obtained from HCOW, HCOE, HCHE, respectively.

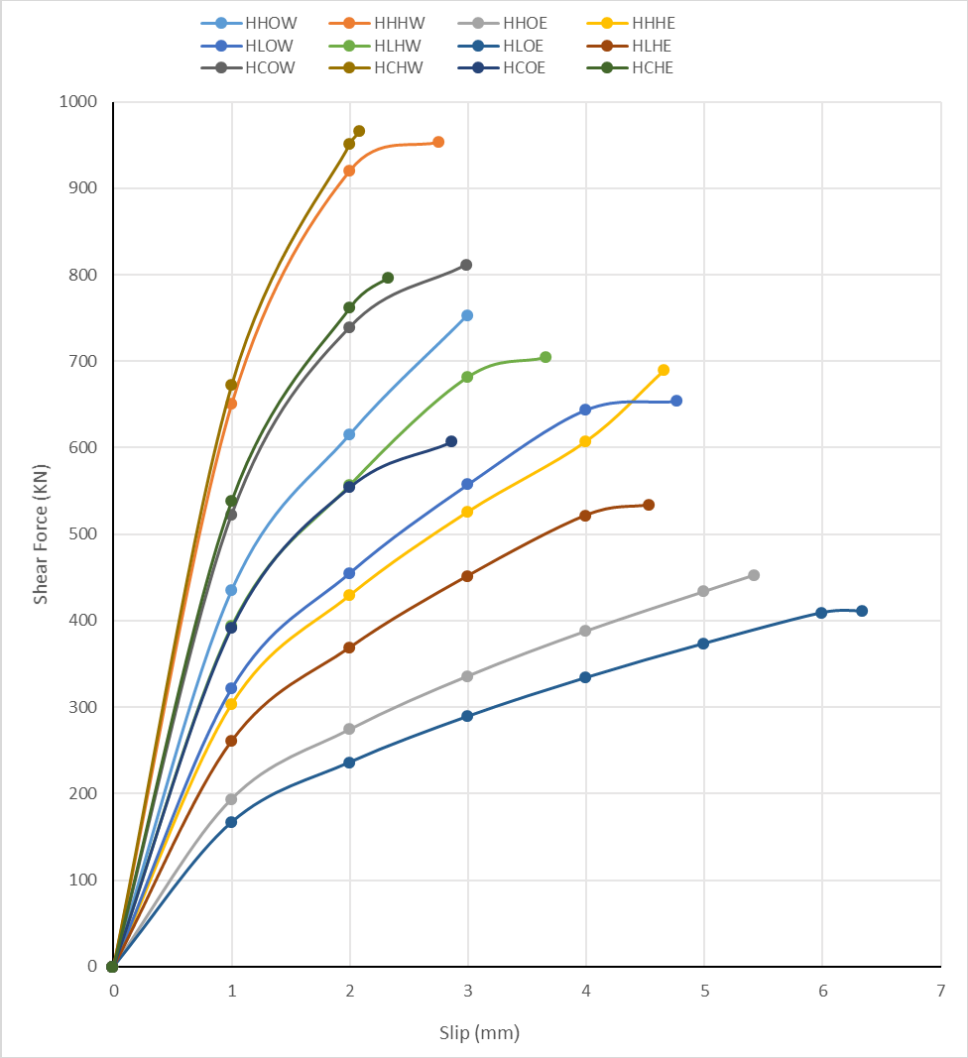


Figure 4.30 Load slip curve for different type of concrete with different type of bonds for high strength concrete type.

Figure 4.31 illustrates the load slip curves for different types of concrete with different types of bonds in reactive powder concrete type (RPC). Based on the data plotted in Figure 4.31, it appears that the specimen that generated the largest shear force is RLOE and the one that generated the smallest shear force is RCHW. It was observed that the increase in shear force for RCHW for same characteristics of shear connectors and Type of concrete, was around 22%, 64%, and 37% when compared to the shear forces for RCOW, RCOE, RCHE, respectively.

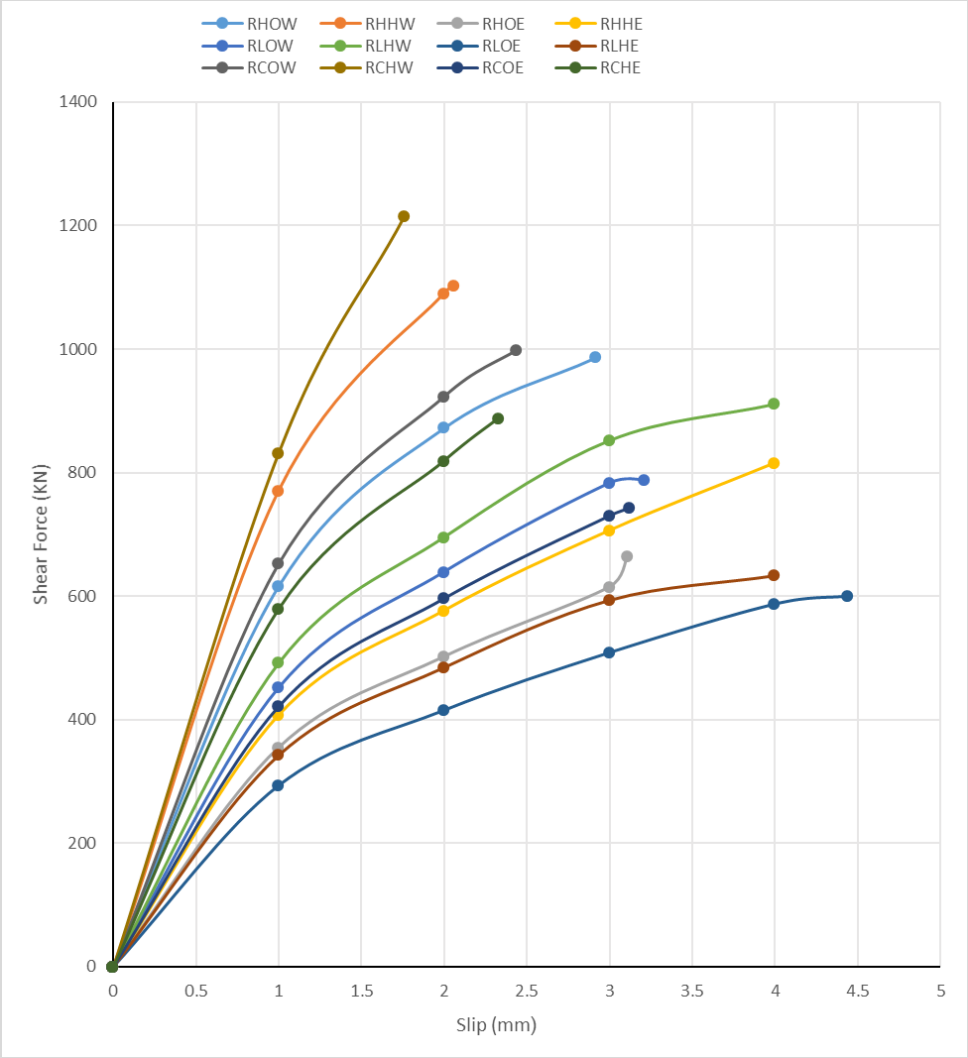


Figure 4.31 Load slip curve for different type of concrete with different type of bonds for reactive powder concrete type

Based on the results from the three previous figures (figures 4.29, 4.30, 4.31), it can be concluded that the C-shaped shear connectors gave higher results due to the increase in the surface area associated with the concrete. The load-slip curves for OCHW, HCHW, and RCHW fracture types of failure came to an abrupt end whereas those related to concrete failure OLOE, HLOE, RLOE exhibited a gradual drop.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

In this chapter, the test results are investigated in two sections. First, their mechanical properties and then the data from their push out tests are discussed.

5.1.1 Mechanical Properties

- Compressive strength increases as age increases, RPC and HSC produced the best compressive strength values by an increase of 366.17% and 236%, respectively when they were compared to the value of normal concrete at 56 days.
- Flexural and splitting strength increased using RPC. RPC and HSC gave the best results by an increase of 392% and 280% respectively, when they were compared to the value of the normal concrete at 14 days. For NFC, the splitting tensile strength did not exceed 2.1 MPa.
- Modulus of rupture strength increased when RPC was used. RPC generated the best results of rupture strength by an increase of 507 % compared to the value of normal concrete at 7 days. Similarly, HSC gave the best results at 14 days by an increase of 270% when compared to the value of normal concrete.
- Modulus of elasticity strength increased in RPC. RPC produced the best results by 76 % compared to the normal concrete a 28 days. Similarly HSC generated the best results by an increase of 35% when compared to the normal concrete.

5.1.2- Push – Out Tests

a. Push-out test results for normal weight concrete (NWC):

   OCHW generated the best shear force value of 876 KN with a percentage of difference of 84% when compared to the force obtained from OHHW.

b. Push-out test results for high strength concrete (HSC):

   HCHW generated the best shear force value of 966 kN with a percentage of difference of 1.26% compared the shear force value generated in HHHW gave the best shear force.

c. Push-out test results for reactive powder concrete (RPC):

   RCHW gave the best shear force value of 1215 KN, with a percentage of difference of 10% compared to that from RHHW.

d. Effect of concrete type:

   - The increase in the shear force value obtained in RHHW with the same characteristics of shear connectors and bonds was around 16% and 42% compared to the shear forces generated in HHHW and OHHW, respectively.
   - The increase in the shear force value for RLHW with the same characteristics of shear connectors and bonds was around 29% and 61% when compared to the values from HLHW and OLHW, respectively.
   - The increase in the shear force value for RCHW with the same characteristics of shear connectors and bonds was around 26% and 39% when compared to HCHW and OCHW respectively.

e. Effect of bond type:

   - The increase in the shear force value for OCHW with the same characteristics of shear connectors and type of concrete was around 17%, 38% and 28% when compared to those from OCOW, OCOE, OCHE, respectively.
   - The increase in shear force value for HCHW with same characteristics of shear connectors and type of concrete was around 19%, 59%, and 21% when compared to those from HCOW, HCOE, HCHE, respectively.

- The increase in the shear force value for RCHW with the same characteristics of shear connectors and Type of concrete was around 22%, 64% and 37% when compared to the shear forces generated in RCOW, RCOE, RCHE, respectively.

5.2 Recommendation

Following recommendations are proposed for this study.

a. Using epoxy as connected material gave lower efficiency as compared to welded stud connectors, therefore it is recommended not to use epoxy in composite structures.

b. Using RPC was effective and helped to increase shear force with the same characteristics of shear connectors, and types of bonds. Also RPC lowered the slippage significantly.

5.3 Suggestion for Future Works.

Following future works are listed as suggestion to further study the topic

a. Slip and separation of composite beam with partial interaction theory .

b. Behavior of multi-layer composite reactive powder concrete slab.

c. Behavior and strength of composite column with RPC (Different aspect ratio) under fatigue loading can be studied.

d. Time dependent behavior of composite column with different support conditions can be studied.

e. Composite column with RPC under the effects of fire can be studied.

f. Behavior of composite dome with partial interaction theory.

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