MATHEMATICAL MODELLING FOR BOND STRENGTH OF RECYCLED COARSE AGGREGATE CONCRETE USING GENETIC PROGRAMMING

A

Dissertation

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CERTIFICATE

I hereby declare that the thesis entitled "**Mathematical Modelling for Bond Strength of Recycled Coarse Aggregate Concrete using Genetic Programming**" is an authentic record of my study carried out as requirements for the award of the degree of **Master of Engineering** in **Structural Engineering** at **Thapar University, Patiala** under the supervision of **Dr. Naveen Kwatra, Head and Associate Professor,** and **Dr. Shweta Goyal, Assistant Professor,** Civil Engineering Department, Thapar University, Patiala during July, 2012 to July, 2014. The matter embodied in this report has not been submitted in partial or full to any other university or institute for the award of any degree.

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It is certified that the above statement made by the student is correct to the best of our knowledge and belief.

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ABSTRACT

For the past several years, production of fresh concrete using recycled materials is being increasingly encouraged so as to reduce the environmental impact of concrete construction. In last two decades coarse recycled concrete aggregate (RCA), manufactured by processing of construction and demolition waste has received considerable attention as a potential substitute for natural coarse aggregate (NCA). However, structural application of RCA concrete has been slow primarily because of apprehensions that concrete containing RCA might be inferior to concrete made with NCA. The present work focuses on the effect of RCA on the bond strength between the concrete and the reinforcement. Further, a mathematical model is generated to related Bond Strength with Compressive Strength, bar dia (12mm, 16mm and 20mm), w/c ratio of the mixes.

The experimental program consisted of studying the compressive strength and bond strength of mixes prepared at 5 different w/c ratios (0.42, 0.45, 0.48, 0.51 and 0.55) and using 4 replacement levels for coarse aggregates (0%, 30%, 60%, and 90%). The compressive strength was studied at 7 days and 28 days, while bond strength was calculated at 28 days. For studying bond strength, pull out tests were carried out as per IS:2770 (Part I), 1967 and IS: 432 (Part I), 1966 using 12mm, 16mm and 20mm dia of rebars. It was observed that both compressive strength and bond strength show a similar behaviour with respect to replacement ratio of RCA’s. with the increase in RCA content, initially bond strength and compressive strength decreased by small amount, then it increased at 90% replacement level of coarse aggregates.

Using the experimental data, the mathematical model is generated using GP(Genetic Programming). The replacement ratios, water-cement ratios, diameter of the rebar and compressive strength have been used as input parameters for developing the mathematical expression. The bond strength of the mixes can be correlated well with the input parameters, with an average error of 13%.

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CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Concrete has been proved to be a leading construction material for more than a century. It is estimated that the global production of concrete is at an annual rate of 1 m3 per capita. The global consumption of natural aggregate will be in the range of 48.3 billion metric tons after 2015. Over 1 billion tonnes of construction and demolition waste (C&DW) is generated every year worldwide. At the same time, large quantities of natural aggregates are extracted for construction every year leading to the large scale depletion of natural aggregate and the increased amounts of C&DW. The construction and demolition waste are primarily used for landfill sites which are causing significant damage to the environment and developing serious problems. The use of the recycled aggregates created from processing of construction and demolition waste in new construction has become more important over the last two decades as it conserve the non-renewable natural resource of virgin aggregates.

Though the use of recycled concrete aggregate (RCA) can lead to reduction of up to 40% in compressive strength, for economical and environmental reasons and because of the increased amount of recycled aggregates production, there has been a growing global interest in maximizing the use of recycled aggregates in construction. In view of the increased volumes of construction, demolition waste, and industrial by products and the advantages offered by the use of admixtures in modern concrete, it is considered very beneficial from different prospects with similar performance characteristics to natural aggregate concrete. When proved successful, recycled aggregate concrete (RCA) can be substituted for natural aggregate concrete in many concrete applications. It is believed that impurities, particularly old cement paste clinging to recycled aggregate (RCA), have a significant influence on the strength of RAC. The adhered mortar from the original concrete plays an important role in determining the performance with respect to permeability and strength. This was considered to be one of the most significant differences between RAC and natural aggregate (NA) concrete (NAC). The partial to full replacement of the recycled coarse aggregate to the natural coarse aggregate is also essential to understand the mechanical behaviour of the concrete in general.

1.2 PROCESS OF RCA PRODUCTION

Once a concrete structure has been demolished, large amount of concrete remain as waste material. Any steel and iron material present is then removed using magnetic torches and electromagnets. The reinforcing steel and the concrete are separated out for further processing. The production of recycled coarse aggregate from the demolished concrete debris involves several steps. Harmful substances present in recycled coarse aggregate could include: glass, plastic, plaster, oil droppings, wood, steel, etc. must be removed and arranged separately. Jaw crushers can provide an acceptable particle size distribution necessary for quality recycled coarse aggregate concrete production. Once the aggregates have been separated from demolished concrete, they must be classified according to their particle size utilizing specified standards such as ASTM C 33. After the recycled coarse aggregate source has been fixed and grading is completed. These studies have mainly involved the testing of high strength RCA concrete (Ajdukiewicz and Kliszczewicz, 2002); the shear strength of RCA concrete (Gonzalez and Martinez, 2007, Fathifazl 2008) and the bond strength of RCA concrete with reinforcing steel (Fathifazl 2008, Choi and Kang, 2008, Xiao and Falkner, 2007). However, there are very few case studies of actual structures built utilizing RCA concrete to fully or partially replace NA concrete Once the aggregates have been passed for use in fresh concrete, the mix design process can begin. Density and water absorption characteristics should be considered in order to ensure adequate workability, initial slump and strength.

1.3 PROPERTIES OF RECYCLED COARSE AGGRGATES

RCA often contains a large amount of attached mortar and cement paste. The volume percentage of old mortar may range from 20% to 30%, depending on the properties of parent concrete and the production process. The attached mortar and cement paste on recycled coarse aggregate are the principal cause of the difference between recycled coarse aggregate (RCA) and natural coarse aggregates. Test results indicated that recycled coarse aggregate has the following technical properties [Xu and Shi (2006), Xiao, (2008)]:

1) **Low bulk and saturated-surface-dry (SSD) density**: The bulk density of recycled coarse aggregate is about 1290–1470 kg/m^3.The SSD density of recycled coarse aggregate is about 2310–2620 kg/m^3.

2) High water absorption: The absorptions of recycled coarse aggregate are approximately:

- 8.34% (10 min)
- 8.82 (30 min)
- 9.25% (24 h)

These above values are larger than that of natural coarse aggregates and might be regarded as the most important characteristic.

3) High porosity: The porosity of RCA is approximately 23.3%, due to high mortar cement paste content.

4) High crushing index: The crushing index of RCA is approximately 9.2% to 23.1%.

5) High clay content: The clay content of RCA is approximately 4.08%.

In the technical code “Technical code for application of recycled concrete” (DG/TJ08-2018-2007), only recycled coarse aggregate (minimum size over 5 mm) is permitted for producing recycled coarse aggregate concrete. The grading of the recycled coarse aggregate must fall within the allowable limits for natural aggregate in JGJ 52-2006 “Standard for technical requirements and test method of sand and crushed stone or gravel”. The RCA is classified into two types in terms of their SSD density, water absorption, and brick content. Considering the physical, chemical and physical-mechanical requirements, some limitations are also made for RCA. Table 1.1 lists the requirements for RCA in the technical code DG/TJ08-2018-2007 and a comparison with those in other international specifications, such as RILEM (1994), BS8500 (2002) and JIS TRA 0006 (2000).

1.4 WHILE USING RCA ONE SHOULD BE CAUTIOUS ABOUT THE FOLLOWING POINTS:

Although RCA are normally used as direct replacement of coarse aggregate. However, certain points must be kept in mind while using recycled coarse aggregate in concrete:

Table 1.1: Requirements for RCA specified in Technical Codes (Xiao et al. 2012)

Items	DG/IJ08-2018-2007		RILEM			BS 8500	JIS TRA0006
	TYPE I	TYPE II	TYPE I	TYPE II	TYPE III		
SSD density(kg/m3)	>2400	>2200	>1500	>2200	>2400	-	-
Absorption(%)	< 7	<10	<20	<10	>3	-	<7
Masonarycontent(%)	< 5	<10	-	-	-	<5	-
Crushing value(%)	< 30		-	-	-	-	-
Soundness(mass loss %)	< 18		-	-	-	-	-
Flakiness index(%)	< 15		-	-	-	-	-
Clay content(%)	< 4		-	-	-	-	-
Sulphate content(%)	< 1.0		<1	<1	<1	<1	-
Chloride content(%)	<0.25		-	-	-	-	-
Organic material(%)	< 0.5		<1	<0.5	<0.5	-	-
Fine particle(%)	-		<3	<2	<2	<5	<2
Material with SSD<2200 kg/m3	-		-	<10	<10	-	-
Material with SSD<1800 kg/m3	-		<10	<1	<1	-	-
Material with SSD<1000 kg/m3	-		<1	<0.5	<0.5	<0.5	-
Impurity content (%)	<1		<5	<1	<1	<1	-
Asphalt content(%)	-		-	-	-	<5	-
Metal content (%)	-		<1	<1	<1	-	-
Sand content(< 4mm)(%)	-		<5	<5	<5	-	-

Table 1.2: Requirements for high quality RCA in JIS 5021:2005

ITEMS	COARSE AGGREGATE	FINE AGGREGATE
Oven dry density, g/cm^3	Not less than 2.5	Not less than 2.5
Water absorption, %	Not more than 3	Not more than 3
Abrasion, %	Not more than 3.5	NA
Solid volume percentage for shape determination	% not less than 55	Not less than 53
Amount of material passing test sieve 75 µm	% not more than 1.0	Not more than 7.0
Chloride ion content	Not more than 0.04	-

1.4.1 Absorption and Surface Moisture

Aggregates are porous materials and water can be absorbed onto the body of the aggregates. The absorption capacity is generally defined as the total amount of water required to bring the aggregate to a saturated surface dry (SSD) condition. Aggregates may exist in various moisture states: oven dry, air-dry, saturated surface dry or moist state. Figure 1.4 illustrates the various moisture states of aggregates. A difference must be made between the moisture that is absorbed by the aggregate and the additional water that is observes on the aggregate surface. In concrete mix proportions, it is surface or free moisture that is used to balance the required mixing

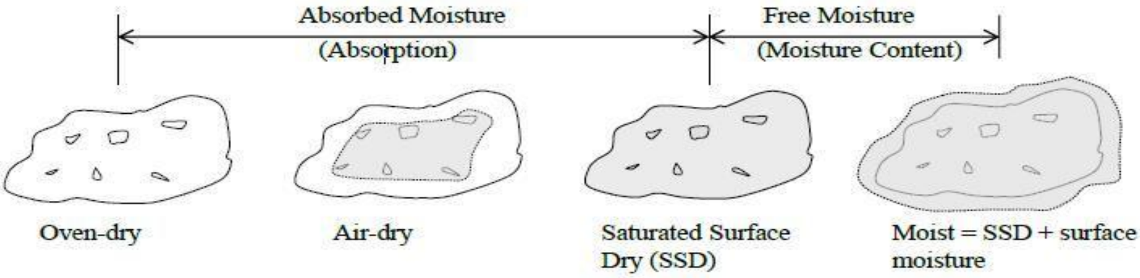


Figure 1.1: Several Moisture States of Aggregates (Neville, 1995)

water. Only free water is available for mixing in concrete and it is this moisture which contributes to the water cement ratio. It is the size and number of internal pores that are responsible for absorption of water in aggregate and it is seen that recycled coarse aggregates have a higher water requirement than natural aggregates due to the higher water absorption value of adhered mortar. The rate of absorption also plays a significant role in concrete mix proportioning including recycled coarse aggregates. In general, recycled coarse aggregates take longer time to absorb moisture than natural aggregates and, as a result, it may not reach full saturation during the mixing period. Therefore, it has been recommended to pre-soak the recycled coarse aggregates to compensate for the slower absorption rate.

1.4.2 Adhered Mortar Content

After crushing of concrete, the resultant recycled coarse aggregate concrete contains both natural stone and old mortar. This old adhered mortar can account for, about 25 to 60 percent by volume of the aggregate itself. It was noted that the finer the aggregate, the more the adhered mortar content. The residual mortar content can have negative impacts also on such concrete properties as absorption, density, abrasion resistance etc. The amount of residual mortar present on recycled coarse aggregate depends largely on the crushing process by which the aggregates are produced. As the number of crushing of the aggregates increases, the amount of adhered mortar is reduced. It was also observed that use of impact crusher produces higher percentage of recycled coarse aggregate with less amount of residual mortar and it is also suggests that the adhered mortar in recycled coarse aggregate give lower strength then the fresh mortar produced in new concrete. As a result, it concluded that the adhered mortar in recycled coarse aggregates is the weakest point in concrete produced with coarse recycled coarse aggregates.

Several methods have been investigated to determine the percent of residual mortar in recycled coarse aggregate. The most general and commonly used method is taking a sample of oven dried recycled coarse aggregate weighing 100 grams in a plastic container. In this container add 1:3 HCL solution such that the HCL solution surface was 15 mm above the aggregates. And when the level of the HCL falls down after some hours add more HCL in order to maintain the level. After 2 days the constituents of recycled coarse aggregate split up. Transfer the recycled coarse aggregate particles to a new container and add fresh HCL solution as before. Again after 2 days the complete breakdown of recycled coarse aggregate takes place. If it does not, then keep the recycled coarse aggregate immersed for a longer time. After the complete disintegration of recycled coarse aggregate remove the coarse aggregates to a 4.75 mm sieve and wash it with hot water to remove all the HCL. Note down the mass of the oven dried coarse aggregates. The percent of adhered mortar can be calculated based on the following expression:

$$\%\ \text{Adhered Mortar} = \frac{\text{Mass of RCA} - \text{Mass of RCA after removal of mortar}}{Mass\ of\ RCA}$$

1.5 ADVANTAGES AND DISADVANTAGES OF RCA CONCRETE

The following are advantages of recycled coarse aggregate

- Recycled coarse aggregate provides sustainability.
- Recycled coarse aggregate reduces the amount of material that would be delivered to a landfill
- Recycled coarse aggregate reduces the need of virgin aggregates to be created.
- RCA uses 90% less energy in production than the production of Portland cement.
- Absorbs large amount of carbon dioxide while being crushed into smaller sizes, reduces the amount of CO2 in the atmosphere.
- Use of high fineness of fly ash in recycled aggregate concrete yielded greater compressive strength.

The following are disadvantages of RCA:

- Recycling plant can cause an increase in noise levels.

- Adhered mortar content have negative impact on absorption and density
- Lack of Specification and Guidelines

1.6 OBJECTIVE OF THE STUDY

The objective of this research is to generate a mathematical model for bond strength using Genetic Programming and study the effect of different parameters on the bond strength between RCA and reinforcement by using different:

- W/C ratios (0.42, 0.45, 0.48, 0.51 and 0.55)
- Replacement ratios (0, 30, 60, 90)
- Diameter of the Rebar (12mm, 16mm and 20mm)

1.7 SCOPE OF THIS STUDY

There is lot of work yet to be done in this field as there has not been any mathematical model generated in the past researches using Genetic Programming. All the models given by the researchers are generated experimentally not by using software. Also by considering so many different parameters research has not been done on Bond Strength. So this thesis opens up a whole new door for generating different mathematical models by using different parameters and adding a new perspective of analysing the things.

1.8 ORGANISATION OF THE THESIS

This thesis has been organized in five chapters as follows:

Chapter 1: Introduction- It presents various aspects of RCA concrete. This chapter also discuss objective, scope, and the methodology adopted for this investigation.

Chapter 2: Literature Review- A review of recent literature on Bond Strength of RCA concrete and reinforcement using different parameters and how Genetic Programming can be used to develop a mathematical model has been discussed on the basis of which the need of the present investigation has been identified.

Chapter 3: Experimental Program- It describes the properties of the materials used in the test specimens, the sizes and the number of specimens, testing methods and the associated instrumentation. Procedure followed in doing Genetic Programming is also discussed.

Chapter 4: Results and Discussions- The analysis of the results, the related discussion and salient observations from the testing have been included in a sequential manner. The results and discussion pertaining to material tests have been presented first and those of the Genetic Programming have been discussed later.

Chapter 5: Conclusion- The significant conclusions obtained from experimental investigations of this study have been integrated and presented in a logical sequence and recommendations for further research made.

At the end, references used in this document are presented.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

In this chapter a review of the literature related to bond strength of RCA concrete and reinforcement has been discussed. A brief review of the published work on material and structural characteristic of RCA concrete is presented and use of Genetic Programming for generating mathematical model have been discussed and finally the need of the present investigation is identified.

2.2 EFFECT OF RCA ON FRESH PROPERTIES OF CONCRETE

2.2.1. Initial Slump

Neville (1995) mentioned that various mixing conditions such as w/c, water-reducing admixture ratio, and grading and volume of recycled aggregates would control the initial slump of recycled aggregate concrete.

Aggregate type: Yang et al. (2008) classified the aggregate as per KS14 and found that the initial slump of fresh concrete slightly decreased with the increase of the replacement level of recycled aggregates but was hardly affected by their type.

Poon et al. (2007) detected slight increase in initial slump with the increase of the replacement level of recycled coarse aggregates used in a saturated surface dry state.

Water volume: Poon et al. (2004) examined the moisture condition of the aggregate on initial slump and it showed that the initial slump of recycled aggregate concrete was well affected by the moisture condition of aggregates.

Water in aggregate: Lin et al. (2004) achieved that the initial slump of recycled aggregate concrete was mainly altered by w/c and volume ratio of recycled coarse aggregate rather than the type of recycled aggregates. In addition, particle distribution and shape of aggregates also had an impact on the initial slump of fresh concrete.

2.3 EFFECT OF RCA ON HARDENED PROPERTIES OF CONCRETE

Concrete has many hardened properties some of which are compressive strength, tensile strength, modulus of elasticity and many more. Few are discussed in the following section:

2.3.1. Compressive Strength

Duan et al. (2014) investigated that the compressive strength of concrete made with RA was mostly lower than that made with NA, regardless of the w/c ratios used. This might be due to the porous nature of RA with a larger amount of cracks and attached mortar.

Singla S. (2013) studied that as the percentage of RCA is increased the compressive strength decreases (up to 30 % RCA replacement) but when replacement ratio is increased more than 50% compressive strength increases and maximum compressive strength was encountered at the replacement level of 90% RCA. This increase in compressive strength was due to the high water absorption capacity of RCA, therefore the effective w/c ratio decreases and hence strength increases.

Xiao et al. (2012) concluded that compressive strengths of recycled coarse aggregate are generally lower than those of conventional concrete. Further compressive strength values decrease with the increase of RCA amounts. Several reasons could be responsible for the reduction of the compressive strength for RAC, including an increased concrete porosity and a weak aggregate-matrix interface bond.

Kim et al. (2012) studied that compressive strength decreased when the coarse aggregate was replaced with the recycled. Additional replacement of the fine aggregate reduced the strength as the recycled fines amount increased. When the fine aggregate replacement was greater than 60% the strength reduction became more significant. Reason for reduction in compressive strength is

(i) Remained mortar on the surface of the recycled aggregate
(ii) Cracks in the aggregate itself (which could occur during the crushing)
(iii) The original aggregate's strength. From, the observations, it is recommended that the fine aggregate should better be replaced with the recycled less than 60% in the consideration of compressive strength.

Butler et al. (2011) however, concluded that recycled coarse aggregate concrete had higher compressive strength values than the natural aggregate concrete. This is likely due to the stronger mortar–aggregate bond between the RCA and the new mortar.

Rahal (2007) when relative water absorption of aggregate is below 1.8%, the compressive strength of recycled aggregate concrete maintains more than 80% of that of the control concrete with natural aggregates, whereas the compressive strength of recycled aggregate concrete having relative water absorption of aggregate above 5.5% drops significantly, by as much as approximately 40% of that of the control concrete with natural aggregates. Insufficient hydration and a weak interface-zone formed between different components of the concrete matrix owing to a large amount of old cement paste on the surface of recycled aggregates, which can be the cause of a poor development of the compressive strength of concrete. Figure 2.1 shows that higher strength can be achieved by reducing the water to cement ratio in recycled concrete aggregate.

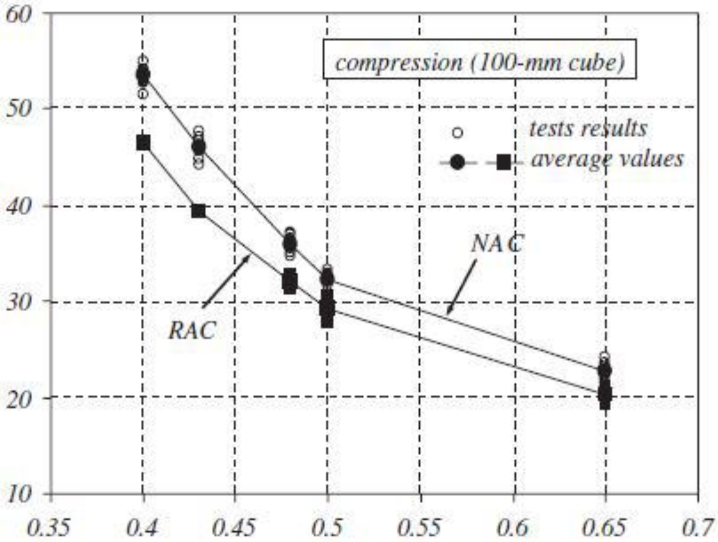


Figure 2.1: 28 days compressive strength versus w/c ratio (Rahal 2007)

Tu et al., (2006) concluded that an inconsistent surface of recycled fine aggregate would produce numerous micro cracks between aggregates and cement paste, which would reduce compressive strength of concrete.

Khatib (2005) pointed out that the absorbed water in the recycled aggregate may have helped with internal curing by providing a source of water to react with the cement. The relative compressive strength of recycled aggregate concrete decreases with the increase of relative water absorption of aggregate and the relative compressive strength can also be significantly affected by the w/c and curing condition.

Poon et al. (2004) observed that the Influence of moisture states of natural and recycled aggregates on the compressive strength of concrete, and concluded that the concrete mixtures prepared with the incorporation of recycled aggregates, the air dried (AD) aggregate concretes exhibited the highest compressive strength. The surface dried density (SSD) recycled aggregates seemed to impose the largest negative effect on the concrete strength, which might be attributed to ‘‘bleeding’’ of excess water in the pre-wetted aggregates in the fresh concrete. Based on the results of his study, aggregates in the AD state, containing not more than 50% recycled aggregate should be optimum for normal strength recycled aggregate concrete production.

Chen et al. (2003) studied the impact of washed RA when used as coarse aggregate. They found that washed RA comprised higher strength than that of unwashed RA. Greater bond effects were produced when impurities, powder and harmful materials on aggregate surface in RA are washed away. They also identified that at low w/c ratio, the compressive strength ratio of recycled concretes to normal concretes are decreased. Main factor which lead to this result is strength of the paste is increase at low w/c ratio. Based on composite material theory, they revealed that RA will become a weak material and its bearing capacity become smaller which tends to decrease the strength.

2.3.2 Splitting Tensile Strength

Xiao et al. (2012) observed that as increase recycled coarse aggregate amount tensile strength decreases, and tensile strength of recycled coarse aggregate concrete is lower as compare to those conventional concrete.

Butler et al. (2011) found strong relationship between aggregate crushing value (ACV) and splitting tensile strength. As the ACV increase, the splitting tensile strengths become more sensitive. Comparison based on ACV can be made between a particular recycled coarse aggregate source and a natural aggregate source. This comparison could be used as an early indicator of how concrete produced with recycled coarse aggregate will perform, with respect to its splitting tensile strength, compared to concrete produced with natural coarse aggregate.

Yang et al. (2008) observed that the normalized splitting tensile strength of recycled aggregate concrete decreased with the increase of relative water absorption and it was less than 0.53 for most specimens having relative water absorption larger than approximately 2.25%.

2.3.3. Modulus of Elasticity

The modulus of elasticity of concrete made with Grade I coarse RCA at 100% replacement level was above $4700\sqrt{f'_{ck}}$ as specified in ACI 318-05. In addition, the elastic modulus of concrete containing Grade II fine RCA was nearly similar to that of concrete with Grade III coarse RCA for the same replacement level. The normalized elastic modulus $E_c/\sqrt{f_{ck}}$ of recycled aggregate concrete decreased with the increase of relative water absorption, indicating that a lower elastic modulus was exhibited by recycled aggregate concrete having relative water absorption above 3.0% than that used in ACI 318-05 for concrete with natural aggregates.

Yang et al. (2008) pointed out that the impact force during the crushing process of waste concrete would result in poor strength and stiffness of recycled aggregate would in turn reduce the elastic modulus of recycled aggregate concrete.

2.4 EFFECT OF RCA ON BOND BETWEEN STEEL AND CONCRETE

The bond of reinforcement in concrete is responsible for three main features of structural performance, namely

(1) Bond is used to anchor the ends of reinforcing bars

(2) Bond transfers force from concrete in tension, thereby reducing the strain in the flexural reinforcement and enhancing member stiffness

(3) Bond helps to maintain the composite action between the reinforcing bar and surrounding concrete. Bond action is also required to ensure sufficient level of ductility in structural members. In design codes, bond is generally assumed as shear stress acting uniformly along the nominal surface area of a reinforcing bar. Practically, the bond stress varies along the length of the rebar and higher at the ends

of the rebar. Also, in ribbed rebar, the transfer of load between the reinforcing bar and surrounding concrete is initially through bearing of the ribs.

2.4.1 Mechanisms of Bond Resistance

Followings are various mechanisms due to which the bond between the rebar and concrete exists. The major mechanisms are:-

- **Chemical adhesion:** Due to adhesive property in the products of hydration (formed after hardening of concrete).
- **Frictional resistance:** Due to the surface roughness of the reinforcement bar and the grip applied by the concrete shrinkage.
- **Mechanical interlock:** Due to the surface ribs provided in deformed bars. The resistance due to ‘mechanical interlock’ is not available in plain reinforcing bar. Friction starts to play a significant role when ribbed bars are used. Figure 2.2 shows the various bond transfer mechanisms on a reinforcing bar

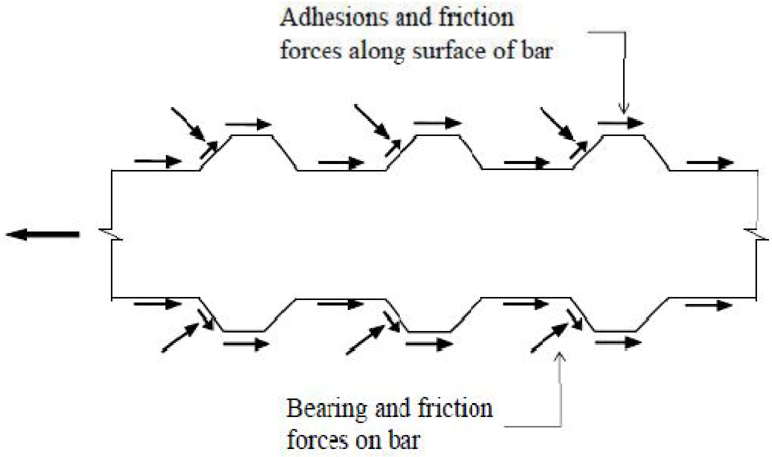


Figure 2.2: Mechanism of bond transfer (Wight et al. 2009)

Bearing and friction forces on ribbed portion of the bar and adhesions and friction forces acting along the surface of bar i.e. compressive bearing force perpendicular to the rib surface increases friction forces parallel to the surface. The forces acting on the rebar surface is balanced by the compressive and shear forces. These compressive and shear forces are then resolute into tensile forces which caused cracks parallel and perpendicular to the reinforcing bar. Generally, splitting cracks may occur if insufficient spacing or cover is provided. If cover, spacing, and transverse reinforcement are inadequate to stop splitting failure then shear failure initiating at the top of the ribs of the bar will occur and a pull-out failure will occur. In general, bond resistance is governed by the following factors:

- The mechanical properties of concrete and its components
- Concrete cover and bar spacing
- Transverse reinforcement
- Surface condition of the bar (ribbed, plain, etc.)
- Bar geometry (deformation height, spacing, width, etc.)

2.4.2 Bond between RCA Concrete and Steel Rebars

Bond strength can be measured by various test setups. The most common is Pull out test, in which the load (P) and the slip (s) at the free end of steel rebar anchored in the test specimen were measured in order to determine a load–slip relationship. The monotonically increased load was applied by the testing machine. Other methods used are beam end specimen and splice beam method. The results obtained by various researchers are discussed in the following sections.

(A) Pull-out Method

Prince et al. (2013) studied the bond performance between deformed steel bars in recycled aggregate concrete The results of sixty pullout tests carried out using 12 mm, 16 mm, 20 mm and 25 mm diameter deformed steel bars concentrically embedded in recycled aggregate concrete designed using equivalent mix proportions with coarse recycled concrete aggregate (RCA) replacement levels of 0%, 25%, 50%, 75% and 100% are reported towards investigation of bond behaviour of RCA concrete. The control concrete mixture consisting of only NCA was designed using the absolute volume method and the mixture design of RCA concrete was carried out using equivalent mix proportions wherein the mixture proportions for the NCA and the RCA concretes were nominally kept the same, except for direct weight-to-weight replacement of NCA with RCA, depending upon the desired RCA replacement level. The RCA replacement level is defined as the weight ratio of RCA to the total coarse aggregates in the concrete mixture and depending upon the desired replacement level, direct substitution of NCA with an equal weight of RCA particles was carried out. The following five weight combinations of NCA and RCA were adopted: 100% NCA (control mixture), 75% NCA + 25% RCA, 50% NCA + 50% RCA, 25% NCA + 75% RCA, 100% RCA, and the concrete mixture proportions are summarised in Table 3 wherein it may be noted that the water-cement ratio, w/c, across all the mixtures was nominally kept equal to 0.54. Since the RCA used in this investigation had water absorption values which were about 6 times higher than that of the NCA, Table 1, the uniform w/c across all the concrete mixtures was achieved by ensuring that the NCA and the RCA particles were in the saturated surface-dry (SSD) moisture condition at the time of batching.

Table 2.1: Concrete mixture proportions (kg/m^3) (Prince et al. 2013)

Mix ID	RCA Replacement (%)	Cement	Sand	NCA	RCA	Mixing Water
AR0	0	369	854	912	0	199
AR25	25	369	854	684	228	199
AR50	50	369	854	456	456	199
AR75	75	369	854	228	684	199
AR100	100	369	854	0	912	199

The pullout tests were carried out using cylindrical specimens 100 mm in diameter and 200 mm long with concentric rebar placement. Pullout specimens are widely used for investigation of bond behaviour because of their ease of fabrication. Pullout tests provide a simple means of comparing relative bond behaviour across different types of concretes and rebars as mentioned in table 2.2.

The water absorption of the RCA particles used in this investigation was about six times higher than that of the NCA and when used in the SSD moisture state the RCA particles through the phenomenon of internal curing readily release water as needed for hydration or to replace moisture lost through evaporation or self dessication. Internal curing is expected to result in better cement hydration, improved integrity of the contact zone between the RCA particles and the concrete matrix and a significant reduction of permeability due to extension of the curing time. The improved integrity and mechanical properties of the concrete conglomerate are expected to enhance the bond strength as well.It is postulated that due to internal curing action of the RCA particles, the relative bond strengths, obtained by normalising the measured bond stress with the respective compressive strength of concrete, across all the RCA replacement levels were higher for the RCA concrete compared to the NCA concrete. Further, the relative bond strengths increased with RCA replacement levels and the highest values were obtained for 100% replacement of natural coarse aggregate with RCA.

Table 2.2: Experimental Results of Pullout Specimens (Prince et al. 2013)

Specimen	**CS (MPa)**	**P_{max} (kN)**	**τ_{max} (MPa)**
A12R0	36.9	42.2	18.7
A12R25	28.9	44.2	19.5
A12R50	24.0	42.7	18.9
A12R75	26.2	43.0	19.0
A12R100	24.7	43.2	19.1
A16R0	36.9	59.9	14.9
A16R25	28.9	58.5	14.5
A16R50	24.0	50.4	12.5
A16R75	26.2	54.1	3.5
A16R100	24.7	55.0	13.7
A20R0	36.9	84.6	13.5
A20R25	28.9	77.1	12.3
A20R50	24.0	75.7	12.0
A20R75	26.2	82.6	13.1
A20R100	24.7	83.1	13.2
A25R0	36.9	95.6	9.7
A25R25	28.9	89.9	9.2
A25R50	24.0	79.4	8.1
A25R75	26.2	80.5	8.2
A25R100	24.7	82.1	8.4

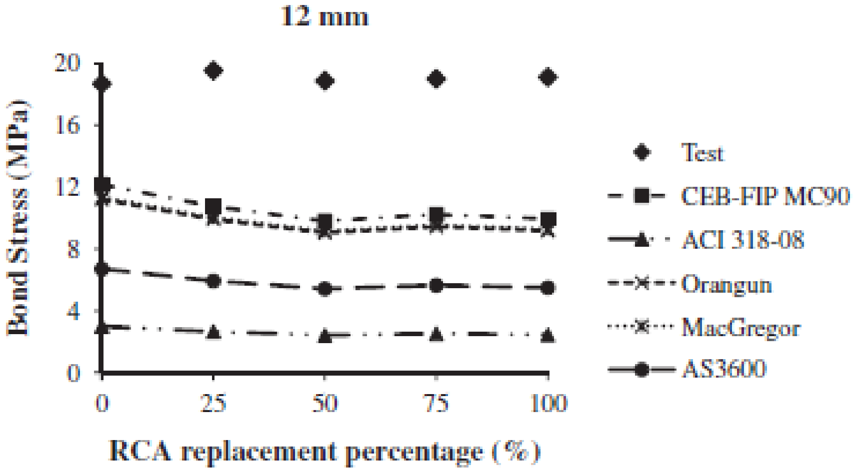


Figure 2.3 (a): Comparison of measured and predicted Bond Stress values for 12mm diameter rebar (Prince et al. 2013)

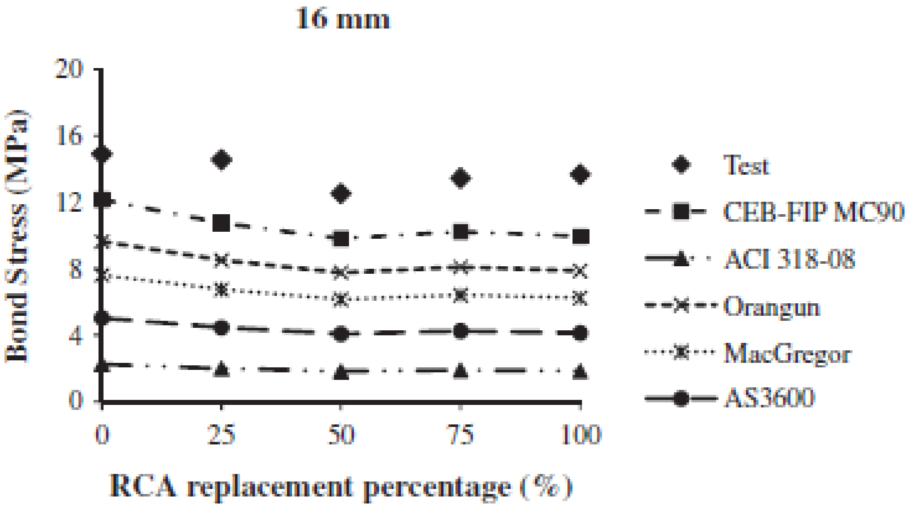


Figure 2.3(b): Comparison of measured and predicted Bond Stress values for 16mm diameter rebar (Prince et al. 2013)

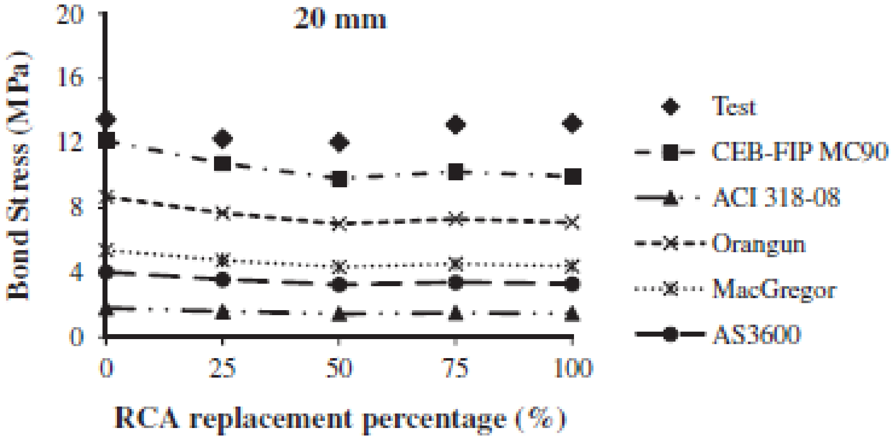


Figure 2.3(c): Comparison of measured and predicted Bond Stress values for 20mm diameter rebar (Prince et al. 2013)

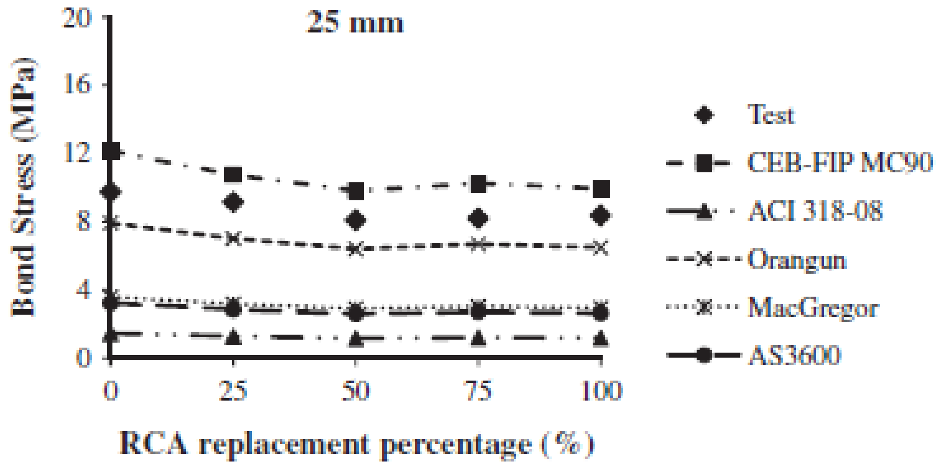


Figure 2.3 (d): Comparison of measured and predicted Bond Stress values for 25mm diameter rebar (Prince et al. 2013)

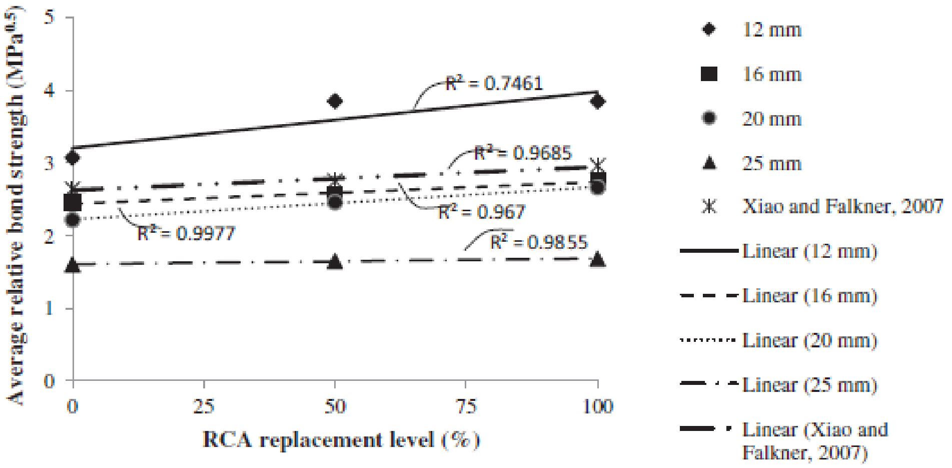


Figure 2.4: Average Relative Bond Strengths for various RCA replacement levels (Prince et al. 2013)

Choi et al. (2008) investigate the bond performance between recycled coarse aggregate and reinforcing bar. The bond strength against slip relationship between RCA and reinforcing bar is reflected as the main characteristic of reinforced-concrete construction, and the shear strength and shear failure of concrete, which effect bond strength, are also investigated. In addition, this study also checks whether the three known equations that have been most commonly used to obtain the bond strength of concrete from the compressive strength of concrete are also applicable for recycled coarse aggregate. These equations are: the Ogura–Koichi model, CEB–FIP model and ACI 408 model. To investigate the bond behaviour between recycled aggregate concrete (RCA) and rebar, this paper considers recycled aggregates (RAs) (RG of grade I and III and RS of grade II), w/c ratios (40% and 50%), and replacement ratios of RAs (0%, 30%, 50% and 100%). The results were as follows:

(a) Bond stress–slip shows tendencies like those of normal concrete up to a replacement ratio of 50%

(b) Shearing stress–Strain is influenced by grade and the replacement ratio of RAs

(c) τ_b is influenced by grade and the replacement ratio of RAs, but at high w/c, τ_b is not influenced by the quality and the replacement ratio of RA's.

(d) $\tau_b / \sqrt{f_{ck}}$ is influenced by the w/c ratio.

(e) τ_b between RCA and rebar is overestimated or underestimated by existing equations. As a result, a new equation (with modification of either the value, invariable value or others), or modified development length or lap splice length must be considered. In concrete mix design, the ratio of the amount of RA to that of the total aggregate is termed the RG and RS replacing ratio Because the used RAs have high water absorption, they were pre-soaked with additional water before mixing, to make the aggregate state saturated and surface dry in this experiment. Two w/c ratios were used: 0.4 and 0.5. The target slump value for each was 150 mm and 120 mm. The mixtures were divided into six groups, the only difference between the groups being the replacing ratios of RA, which were 0%, 30%, 50% and 100%. The mix proportions for this experiment are given in Table 2.3.

The mean pull-out bond strength as a function of both RA ratios and w/c ratios is shown in Fig. 2.5. In w/c ratio 0.5, the bond strength decreased with the increasing RA ratio and the decrease grade/class of RA. The bond strengths of all specimens that used RA were, however, higher than those of normal concrete (N). The bond strength increased as the w/c ratio decreased. This may be because the bond strength between RAC and the rebar depends on the concrete type and properties, mechanical anchorage and friction resistance. That is, concrete type and properties are different according to the RA used, and this difference changed the mechanical anchorage and friction resistance between the two materials. The bond strength can therefore be considered to be strongly influenced by RA ratio, among numerous other factors. The average bond strength of RS-II concrete was lower than that of RG-III concrete.

Table 2.3: Mix proportions (Choi et al. 2008)

Type	w/c: %	Grade		R_{RG}: %	R_{RS}: %	S/A: %	Bulk density: kgf/m^3					
		Coarse	Sand				w	c	NG	NS	RG	RS
N	50	Natural	Natural	0	0	42	175	350	983·7	726·7	0	0
RG-I 30		I		30					688·6		301·1	
RG-I 50				50					491·9		501·8	
RG-I 100				100					0		1003·5	
RG-III 30		III		30					688·6		285·6	
RG-III 50				50					491·9		476	
RG-III 100				100					0		952	
RS-II 30		Natural	II	0	30				983·7	508·7	0	200·8
RS-II 50					50					363·4		334·6
RS-II 100					100					0		669·3
N	40		Natural	0	0			437·5	943·8	697·2	0	0
RG-I 30		I		30					660·6		288·8	
RG-I 50				50					471·9		481·4	
RG-I 100				100					0		962·8	
RG-III 30		III		30					660·6		274	
RG-III 50				50					471·9		456·7	
RG-III 100				100					0		913·3	
RS-II 30		Natural	II	0	30				943·8	488	0	192·6
RS-II 50					50					348·6		321
RS-II 100					100					0		642·1

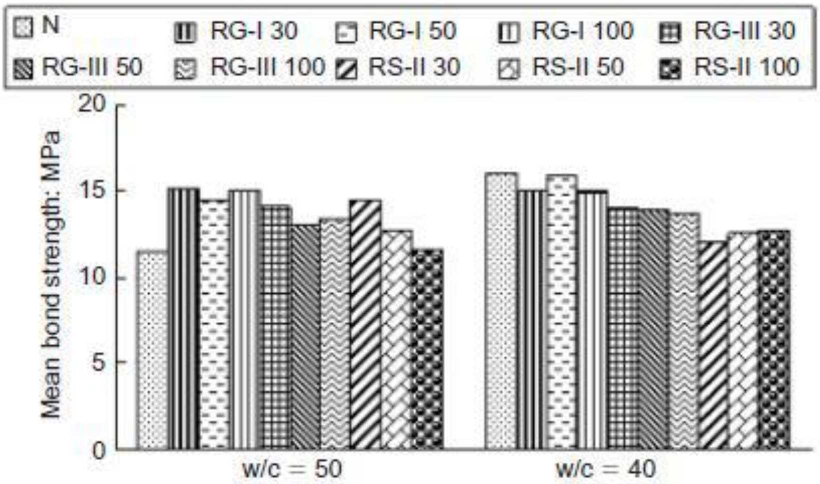


Figure 2.5: Mean pull-out bond strength with the replacement ratio of RA and w/c ratio. (Choi et al. 2008)

The type of RA (RG or RS) has a larger negative effect on the bond strength of RAC than the quality of the RA Interestingly, concretes using relatively goodqualityRS still had a poorer performance than concretesthat used RG in the bond.

In w/c ratio of 0.4, the bond strength between the RACand the rebar decreased with an increasing RA ratioand a decreasing RA grade/class—a trend similar tothat of the w/c ratio 0.5. In contrast, however, with the w/c ratio of 0.5, the bond strengths of all specimensthat used RA were lower than that of normal concrete.When the RA ratio was 100% and the w/c ratios werethe same, the bond strength—as a function of the RA ratio— increased by 22.7% (5RG-I 100), 13.6% (5RGIII100) and 0.1% (5RS-II 100) for a w/c ratio 0.5, anddecreased by 7.1% (4RG-I 100), 16.3% (4RG-II 100)and 26.1% (4RS-II 100) for a w/c ratio of 0.4 morethan that of normal concrete (5N and 4N). These results show that the bond strength of RAC was hardly influencedby the w/c ratio but was greatly influenced bythe replacement ratio and the grade of the RA.

Xiao et al. (2007) studied the recycled coarse aggregate replacement ratio and the steel rebar style as the main experimental parameters. The main aim of this work is to investigate the bond behaviour between recycled aggregate concrete and steel rebars and to find a bond stress versus slip relationship between recycled aggregate concrete and steel rebar. Thirty six pull-out test specimens were tested in order to investigate the bond behaviour. Steel rebar (i.e. plain and deformed) and recycled coarse aggregate (RCA) replacement percentages (i.e., 0%, 50% and 100%) were the main parameter considered in this paper. The mean values of the bond strengths are compared in Figure 2.6. From Figure 2.6, it can be concluded that under the equivalent mix proportion (i.e., the mix proportions are the same, except for different recycled coarse aggregate replacement percentages), the bond strength between the recycled aggregate concrete and the plain rebar decreases by 12% and 6% for an recycled coarse aggregate replacement percentage of 50% and 100%, respectively; while the bond strength between the recycled aggregate concrete and the deformed rebar is similar, regardless of the recycled coarse aggregate replacement percentage. Therefore, it is concluded that for the plain rebar, with increase in recycled coarse aggregate replacement ratio, bond strength decreases, whereas for the deformed bar the bond strength between the recycled coarse aggregate and the deformed rebar remains same.

(B) Beam-End Specimens

Butler et al. (2011) investigateconcrete bond strength by replacing natural coarse aggregate with recycled coarse aggregate (RCA). For the investigation two sources of RCA were used along with one natural aggregate source. All the aggregate properties were tested for all aggregate sources. Two different types of concrete mix proportions were developed in which 100% of the natural aggregate was replaced with RCA. In the first one the same water–cement ratio was maintained and in the second type the mix proportion was designed to achieve the same compressive strengths. Beam-end specimens were casted. The mix proportions obtained are shown in table 2.3 & 2.4

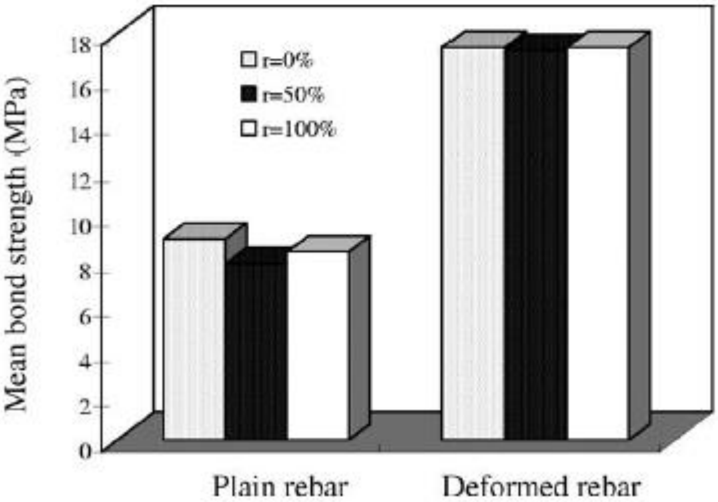


Figure 2.6: Comparison for the Mean Value of Bond Strength (Xiao et al. 2007)

Table 2.4: Control concrete mixture proportions and test results (Butler et al. 2011)

Material	NAC-30	NAC-50
Water(kg/m^3)	160	180
Cement(kg/m^3)	267	474
Coarse aggregate(kg/m^3)	1106	1106
Fine aggregate	861	633
w/c ratio	0..60	0.38
Slump(mm)	90	90
Compressive strength (MPa)	34.4	54.7

Table 2.5: Direct replacement concrete mix proportion and test results (Butler et al. 2011)

Material	RAC1-30	RAC1-50	RAC2-30	RAC2-50
Water(kg/m^3)	160	180	160	180
Cement(kg/m^3)	267	474	267	474
Coarse aggregate(kg/m^3)	975	975	949	949
Fine aggregate	863	635	863	635
w/c ratio	0.60	0.38	0.60	0.38
Slump(mm)	25	35	45	75
Compressive strength(MPa)	44.1	59.0	36.9	54

The strength-based mixtures were developed for two reasons:

1. To determine whether RCA mix proportions could be developed to obtain similar compressive strength and workability to that of natural aggregate concrete
2. To determine the effect that natural aggregate replacement with RCA has on bond strength. Figure 2.5 shows the test apparatus used in this investigation

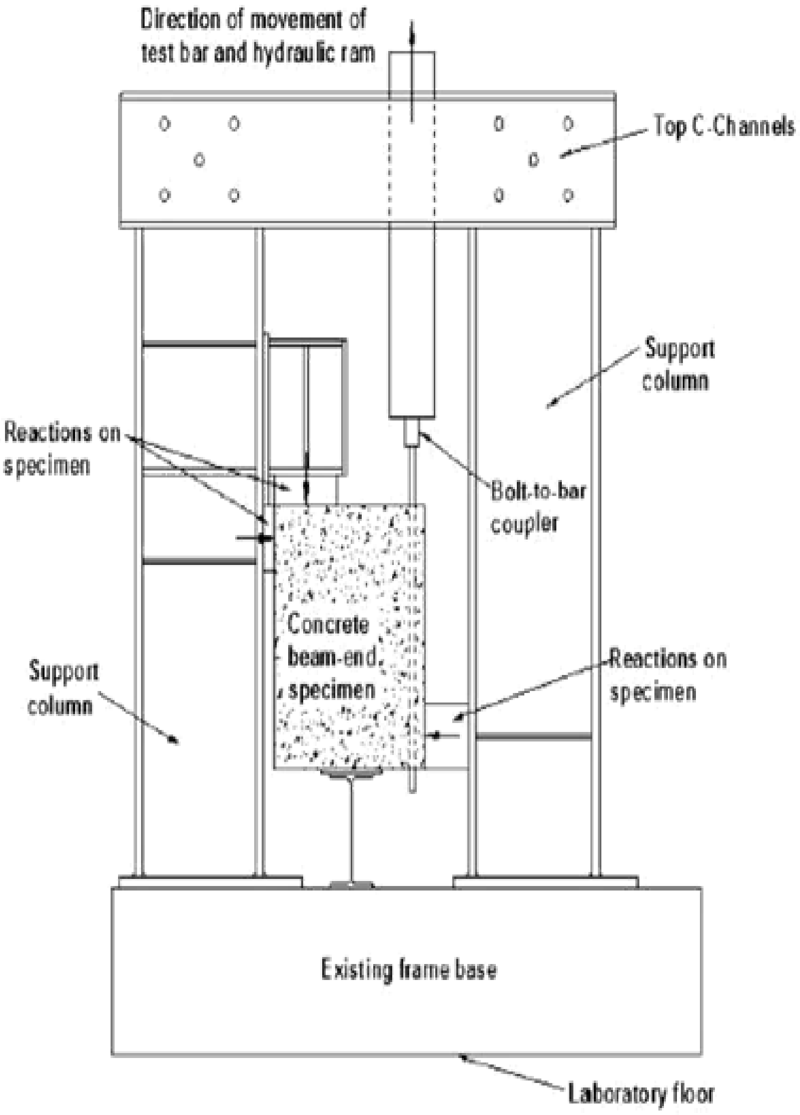


Figure 2.7: Beam-End Test Frame Set-Up (Butler et al. 2011)

Beam-end specimens were tested to investigate the relative bond strength of RCA and natural aggregate concrete. Average bond stress values, for the 125 mm and 375 mm bonded lengths are summarized in Fig. 2.8 (a) and (b), respectively. The natural aggregate concrete beam-end specimens with bonded lengths of 125 mm had average bond strengths that were 11.4 to 19.0% higher than the RCA-1 concrete specimens and 13.2 to 21.3% higher than the RCA-2 specimens. It was concluded that, natural aggregate concrete specimens had 9 to 19% higher bond strength value than the equivalent RCA specimen.

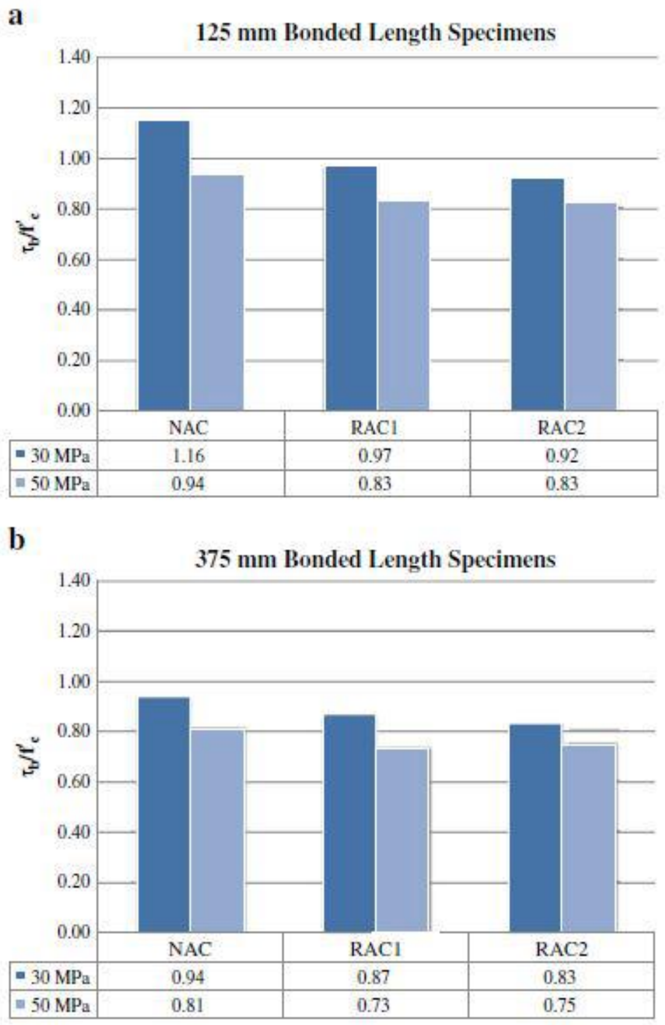


Figure 2.8 (a) and (b): Average Bond Stress data for NAC, RAC1 and RAC2 (a) 125mm bonded length specimens. (b) 375 mm bonded length specimens. (Butler et al. 2011)

Darwin et al. (1992) studied the new bar geometries to determine the degree of improvement in bond strength. The parameters which played a critical role in bond strength were concrete cover, bar spacing, and confining reinforcement. Foremost among the changes is a switch from bond stress to bond force as the measure of strength. The switch was made because bond stress is usually expressed as an average value at failure, when, infact, bond stress varies significantly over the length of a bar at failure. Since bond strength is a structural, rather than material property, bond force provides a better measure of member response than bond stress. The equation proposed was:

$$(A_b f_s)/\sqrt{f_c'} = 6.67 l_d \,(C + 0.5 d_b)\left(0.92 + 0.08\frac{Cmax}{Cmin}\right) + 300 A_b$$

A_b = Area of deformed bar

f_c' = compresssive strength

d_b = diameter of the deformed bar

C_{max} = max[min (C_x, C_y, $C_s/2$), C_y]

C_{min} = min (C_x, C_y, $C_s/2$)

C_x = side cover

C_y = bottom cover

C_s = spacing between the bars

Orangun et al. (1977) studied the effect of development length and splices and gave a mathematical model. The equaation was developed for calculating development length nd splice length for deformed bars. The equation was based on the non linear regression analysis of test results of beams with lap splices and reflects the effect of length, cover, spacing,bar diameter, concrete strength and transverse reinforcement on the strength of anchored bars. The equation:

$$u^*/\sqrt{f_c'} = 1.22 + 3.23\, C/d_b + 53 d_b/l_d$$

where,

u^* = average bond stress

f_c' = compressive strength

d_b = diameter of deformed bar

l_d = develpoment length

2.5 GENETIC PROGRAMMING

2.5.1 Review of Literature Of Genetic Programming

Aggarwal D. (2013) studied that Genetic Programming can be used to obtain mathematical models to predict wind induced pressures on tall rise buildings. The data obtained from the experimental study is used to develop the database for the gable roof building. The angle of wind incidence, roof slope and pressure coefficients for various zones has been used as the parameters for the development of the equations for the estimation of wind induced pressure on gable roof building. GP has also been used for the estimation of Gust factor and Base Moment on tall buildings.

Muduli et al. (2013) examined the potential of multi-gene genetic programming (MGGP) based classification approach to evaluate liquefaction potential of soil in terms liquefaction index (LI) using a large database from post liquefaction cone penetration test (CPT) measurements and field manifestations. The database consisted of CPT measurements; cone tip resistance (q c), friction ratio (R f), vertical total stress (σ v) and vertical effective stress of soil (σ v′), seismic parameters; peak horizontal ground surface acceleration (a max) and earthquake moment magnitude (M w), and the depth under consideration (z). The MGGP models were developed for predicting occurrence and non-occurrence of liquefaction on basis of combination of above input parameters. The performance of the models was found to be more efficient compared to available artificial neural network models.

Gandomi et al. (2010) proposed the formulation of compressive strength of carbon fibre reinforced plastic (CFRP) confined cylinders using Linear Genetic Programming (LGP).The LGP-based models were constructed using two different sets of input data. The first set of inputs comprised of diameter of concrete cylinder, unconfined concrete strength, tensile strength of CFRP laminate and total thickness of utilized CFRP layers. The second set included unconfined concrete strength and ultimate confinement pressure which were the most widely used parameters in the CFRP confinement existing models. The models were developed based on experimental results collected from the available literature. The results demonstrated that the LGP-based formulas predicted the ultimate compressive strength ofan acceptable level of accuracy. The LGP results were also compared with several CFRP confinement models presented in the literature and found to be more accurate in nearly all of the cases.

Saridemir (2010) developed two models in gene expression programming (GEP) approach for predicting compressive strength of concretes containing rice husk ash at the age of 1, 3, 7, 14, 28, 56 and 90days. For purpose of building the models, experimental results for 188 specimens produced with 41 different mixture proportions were obtained from the literature. According to these experimental results, the models were arranged by using seven different input variables in GEP approach. In according to these input variables, the compressive strength values from mechanical properties of concretes containing rice husk ash were predicted in GEP approach models. The results of training, testing and validation sets of the models were compared with experimental results. All of the results showed that GEP is a strong technique for the prediction of compressive strength values of concretes containing rice husk ash.

Kermani et al. (2009) presented the use of genetic programming for predicting the equations for the ratio of maximum velocity to maximum acceleration (v_{max}/a_{max}) of strong ground motions. The predictive equations were established using a reliable database released by Pacific Earthquake Engineering Research Centre (PEER) for three types of faulting mechanisms including strike slip, normal and reverse. The proposed models provided reasonable accuracy to estimate the frequency content of site ground motions in practical projects.

Nitsure et al. (2009) used Genetic Programming to estimate an important oceanic parameter i.e. Significant Wave Height (SWH) using the wind information. Wave and wind measurements taken by moored ocean buoys were used to develop the GP models. The GP based estimations were found to have a reasonable accuracy in estimation of significant wave heights as evident from wave plots and accompanying high values of correlation coefficient.

Heshmati et al. (2008) proposed new formulations for soil classification using Linear Genetic Programming (LGP). Properties of soil, namely liquid limit, plastic limit, colour of soil, percentage of gravel, sand and fine grained particles are used as input variables. The database used for the study was obtained from the previous published literature. The results of the proposed formulations were compared with the existing models found in the literature. From the comparisons it was observed that the LGP based formulations predicted the targeted values with higher degree of accuracy.

Johari et al. (2006) employed Genetic Programming approach to predict the soil-water characteristic curve (SWCC) of soils. The input set consisted of initial void ratio, initial gravimetric water content, logarithm of suction normalised with respect to atmospheric air pressure, clay content and silt content. The output set consisted of the gravimetric water content corresponding to the assigned input suction. Results from pressure plate tests carried on clay, silty clay, sandy loam and loam compiled in the Soil Vision software, were adopted as a database for developing and validating the GP model. GP software (GPLAB) provided by MATLAB was employed for analysis. GP simulations were compared with the experimental results as well as the models proposed by other investigators. The comparison indicated superior performance of the proposed model for predicting SWCC.

2.6 CLOSING REMARKS

In this chapter we have discussed about various experimental and analytical work done in the field of recycled coarse aggregates. Many researchers have worked on the bond strength so far but none of them had worked on bond strength using so many different parameters and not generated a mathematical model using Genetic Programming.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

The details of experiment programme in terms of material properties, test set-up for measuring different parametersare the testing procedurediscussed in this chapter.

3.2 MATERIAL PROPERTIES

Cement, fine aggregates, coarse aggregates, recycled coarse aggregate, super-plasticizer and water is used for present investigation. The propertiesof these materials are discussed in the following sections.

3.2.1 Cement

Cement is a fine, grey powder. It is mixed with water and materials such as sand, gravel, and crushed stone to make concrete. The cement and water form a paste that binds the other materials together as the concrete hardens. The ordinary cement contains two basic ingredients namely argillaceous and calcareous. In argillaceous materials, clay predominates and in calcareous materials calcium carbonate predominates. Ordinary Portland cement of grade – 43 (J.K cement) conforming to Indian standard IS:8112-1989 has been used in the present study. The results of the various tests on cement properties are given in Table 3.1.

3.2.2 Fine Aggregates

The material which passes through 4.75 mm sieve is termed as fine aggregate. Usually natural sand is used as a fine aggregate at places where natural sand is not available crushed stone is used as a fine aggregate. The sand used for the experimental works is locally procured and conformed to grading zone III. The sieve analysis of fine aggregates is shown in Table 3.2. The physical properties are provided in Table 3.3.

Table 3.1: Physical Properties of Ordinary Portland cement

Physical properties	Values obtained	Values as per IS 8112:1989
Setting time (minutes)		
Initial	127	Not less than 30 minutes
Final	183	Not greater than 60 minutes
Compressive strength (MPa)		
3 days	33.4	23
7 days	42.5	33
28 days	49.9	43
Standard consistency (%)	28.2%	-
Specific gravity	3.01	-
Fineness (m^2/kg)	281.3	Not less than 225

Table 3.2: Sieve Analysis of Fine Aggregates

Sr. No.	Sieve No.	Weight retained (Grams)	Percentage retained (%)	Percentage Passing (%)	Cumulative percentage retained (%)
1.	4.75 mm	5	0.50	99.50	0.50
2.	2.36 mm	59	5.90	93.60	6.40
3.	1.18 mm	136	13.60	80.00	20.00
4.	600 mm	243	24.30	55.70	44.30
5.	300 mm	415	41.50	14.20	85.80
6.	150 mm	122	12.20	2.00	98.00
7.	Pan	20	2.00	-	-
					$\Sigma F = 255$

Fineness Modulus of fine aggregate = $\Sigma F/100 = 255/100=2.55$

Table 3.3: Physical Properties of Fine Aggregates

Sr. No.	Characteristics	Value
1.	Type	Natural sand
2.	Specific Gravity	2.58
3.	Fineness Modulus	2.55
4.	Grading Zone	Type III

3.2.3 Natural Aggregate

The broken stone is generally used as a coarse aggregate. The nature of work decides the maximum size of the coarse aggregate. Locally available coarse aggregate having the maximum size of 20 mm was used in the present work. The properties of natural aggregate are presented Table 3.4

Table 3.4: Sieve Analysis of Natural Coarse Aggregates

Sr. No.	Sieve size	Weight retained (kg)	% retained	% passing For NCA	Cumulative % weight Retained
1.	20 mm	0	0	100	0
2.	12.5 mm	2.1865	72.883	27.117	72.833
3.	10 mm	0.6745	22.483	4.634	95.366
4.	4.75 mm	0.1390	4.633	0.01	99.99

3.2.4 Recycled Coarse Aggregate

Tested concrete specimens e.g. cubes, cylinders and beams lying in the dump yard of structure laboratory at Thapar University has been used as a source of RCA. To make RCA, the specimens without reinforcement were manually broken down into small pieces and then crushed using jaw crusher as shown in Fig. 3.1 and 3.2. The larger fraction, passing through 20 mm sieve but retained on 12.5 mm sieve was designated RCA20 – 12.5 mm. The fraction passing through 12.5 mm sieve was discarded.

Table 3.5: Sieve Analysis of Recycled Coarse Aggregates

Sr. No.	Sieve size	Weight retained (kg)	% retained For RCA	%passing For RCA	Cumulative % weight retained
1.	20 mm	0	0	100	0
2.	12.5 mm	2.355	78.5	21.5	78.5
3.	10 mm	0.615	20.5	1	99
4.	4.75 mm	0.03	1	0	100

3.2.5 Water

Water is an important ingredient of concrete as it actively participates in the chemical reaction with cement. Since it helps to form the strength giving cement gel, the quantity and quality of water is required to be looked into very carefully. Potable

Figure 3.1: Waste Concrete Rubble for obtaining RCA

Figure 3.2: Jaw Crusher to obtain RCA

water is generally considered satisfactory. In the present investigation, tap water is used for both mixing and curing purposes.

Table 3.6: Physical Properties of the Recycled and Natural Course Aggregate

Sr. No.	Properties	Natural aggregates	Recycled aggregates
1.	Specific gravity	2.31	2.67
2.	Water absorption (%)	1.8	3.4
3.	Fineness modulus(%)	7.6	7.8

3.2.6 Reinforcing Steel

High strength deformed steel bars (Fe 500) with a nominal diameter of 12mm, 16mm and 20mm of tensile strength 520 MPa, 533.412 MPa and 600MPa respectively was used as main longitudinal reinforcements in all pull out test specimens. Along with the main bar, the cube is reinforced with a helix of 6 mm diameter plain mild steel reinforcing bar conforming to Grade I of IS: 432 (Part I)-1982 at pitch 25 mm pitch, such that the outer diameter of the helix is nearly equal to size of the cube.

3.3 MIX DESIGN

Concrete mix has been designed using the provisions under the IS code specification of IS 10262:2009. All the concrete mixes were designed for 5 different water-cement ratios (0.42, 0.45, 0.48, 0.51 and 0.55). The desired workability of the mix is kept at 60-70 mm slump. The mix proportion of corresponding mixes for each water-cement ratio was prepared by replacing natural aggregate by recycled coarse aggregate. In this, mixture proportions for the natural coarse aggregate (NCA) and the recycled coarse aggregate (RCA) concretes were nominally kept the same, except for replacement of NCA with recycled coarse aggregate, depending upon the desired recycled coarse aggregate replacement percentage.

The RCA replacement percentage is defined as the weight ratio of recycled coarse aggregate to the total coarse aggregates in the concrete mixture and depending upon the selected replacement percentage, direct substitution of NCA with an equal weight of recycled coarse aggregate particles is carried out. The following four weight combinations of NCA and recycled coarse aggregate are adopted:

- 100 % NCA (control mixture)

- 60 % NCA + 30 % RCA
- 30 % NCA + 60 % RCA
- 10 % NCA + 90 % RCA

The concrete mixture proportions and the corresponding mix designations are presented in Table 3.8.In concrete batching, first the natural coarse aggregates and RCA are added in the mixer, subsequently, fine aggregates and cement are added to the mixer the ingredients are dry mixed in the mixer for 2 minutes. Then half of water is added and again mixed for 1 minute. After this, the rest of the water mixed for another 2 minutes. The mixture is now ready to be poured in the moulds.

Table 3.7: Mix Proportion for Controlled Sample

w/c ratio	Cement (kg/m^3)	Fine aggregate (kg/m^3)	Natural aggregate (kg/m^3)	Water (kg/m^3)
0.42	486	640	1256	206
0.45	453	651	1274	206
0.48	426	657	1291	206
0.51	400	664	1303	206
0.55	370	672	1312	206

3.4. CASTING OF SPECIMENS

In this section casting procedure for compressive strength test and pull out strength test are discussed

3.4.1 Casting for Compressive Strength Test

150mm cube is used to study the compressive strength of various mixes. The cubes are filled with fresh concrete using vibrating table. Immediately after casting cubes, the specimens are covered with gunny bags to prevent water evaporation. Three cubes are casted for each parameter. The compressive strength test is carried out for 7 days and 28 days. Therefore, six identical specimens are casted for each concrete mix. The cubes after casting are shown in Figure 3.3.

Table 3.8: Mix Proportion for Mixes Different Replacement Of NA And RCA

Designation	w/c Ratio	Replacement (%)	Cement (kg/m^3)	Fine aggregate (kg/m^3)	Natural aggregate (kg/m^3)	Recycled aggregate (kg/m^3)	Water (kg/m^3)
A1–0	0.42	0	486	640	1256	-	206
A1–30		30	486	640	879	376	206
A1–60		60	486	640	376	879	206
A1–90		90	486	640	124	1131	206
A2–0	0.45	0	453	651	1274	-	206
A2–30		30	453	651	891	382	206
A2–60		60	453	651	382	891	206
A2–90		90	453	651	131	1152	206
A3–0	0.48	0	426	657	1291	-	206
A3–30		30	426	657	903	388	206
A3–60		60	426	657	388	903	206
A3–90		90	426	657	129	1162	206
A4–0	0.51	0	400	664	1303	-	206
A4–30		30	400	663	912	391	206
A4–60		60	400	663	391	912	206
A4–90		90	400	663	130	1173	206
A5–0	0.55	0	370	672	1312	-	206
A5–30		30	370	672	918	394	206
A5–60		60	370	672	394	891	206
A5–90		90	370	672	131	1179	206

3.4.2 Casting for Pull out Strength Test

Pull out specimens are widely used for investigation of bond behaviour between rebar and concrete because of their ease of fabrication and the simplicity of the test. Pull out tests provide a simple means of comparing normalized bond behaviour. In the present investigation, cube of size150mm is used for carrying out the pull out strength test. The specimen is prepared as per the IS code guidelines from IS: 2770 (Part I) – 1967(Methods of testing bond in reinforced concrete). In this, three rebar of 12mm, 16 mm and 20mm diameter is used as concentric reinforcement that will be pulled for finding pull out strength. The pulls out specimens are cast in a vertical position in the laboratory using steel moulds. The embedded length is kept 5 d_b (rebar diameter) and

Figure 3.3: Casting of cubes

was so selected to avoid yielding of the steel bar under pull out load. Contact between the concrete and the rebar along the debonded length was broken using a coaxially placed soft rubber tube. Along with this, a helix reinforcement (as specified by IS: 2770 (Part I) – 1967) of 6 mm diameter conforming to grade I of IS: 432 (Part I) – 1966 at pitch of 25 mm such that the outer diameter of the helix is equal to the size of the cube. In present test for casting of the cubes, steel moulds are taken and helix is placed in the moulds leaving a small cover at the bottom and sides. Then the rebar is placed exactly in the centre. Then in the mould, the concrete mixed is poured and compacted using vibrating table. To prevent excessive evaporation from the fresh concrete, the pull out specimens are covered with a plastic sheet after casting and demoulded after 24 h following which they are moist cured in the laboratory for a nominal period of 28 days from the day of casting by keeping them wet by gunny bags. The water is sprinkle on gunny bags, twice a day. To ensure repeatability of results, two nominally identical companion specimens are cast for each mix under investigation. The typical sample specimens are shown in Figure 3.4.

3.5 TESTING OF SPECIMENS

In this section test setup for both the tests pull out strength test are discussed.

Figure 3.4: Pull out Test Specimens

Figure 3.5: Helical Reinforcement as per IS: 2770 (Part 1)

Figure 3.6 Arrangement of the Mould Before Casting

3.5.1 Test setup for Compressive Strength Test

As shown in Figure 3.7, three identical specimens are crushed at 7days and three identical specimens are crushed at 28 days. The compressive strength is calculated by dividing the failure load by average cross sectional area.

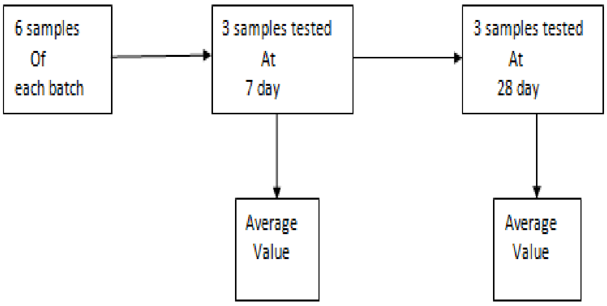


Figure 3.7: Compressive Strength Evaluation Chart of Concrete Cube Specimens

The compressive strength testing machine of capacity 3000 KN is used for determining the maximum compressive loads carried by concrete cubes. At the test age the specimens are taken out of the curing tank and kept outside for 10 minutes. Then one specimen is placed on the steel platen of the machine such that the specimen is tested perpendicular to the casting position. Then the test is carried out at the loading rate of 5 KN/s specified IS: 516– 1959.

3.5.2 Test Setup for Pull out Strength

Pull out strength test are carried out on universal testing machine of capacity 1000 KN shown in Figure 3.8.

Figure 3.8: Pull out test machine

The test setup for pull out test is shown in Figure 3.8. A special arrangement of 40 mm thick steel plate is made to carry out the test shown in Figure 3.9. The plates are connected by four nut-bolts of 25 mm diameter. Lower plate is welded with rod having diameter 25mm. And opening at the top plate just allow the deformed steel bar to pass.

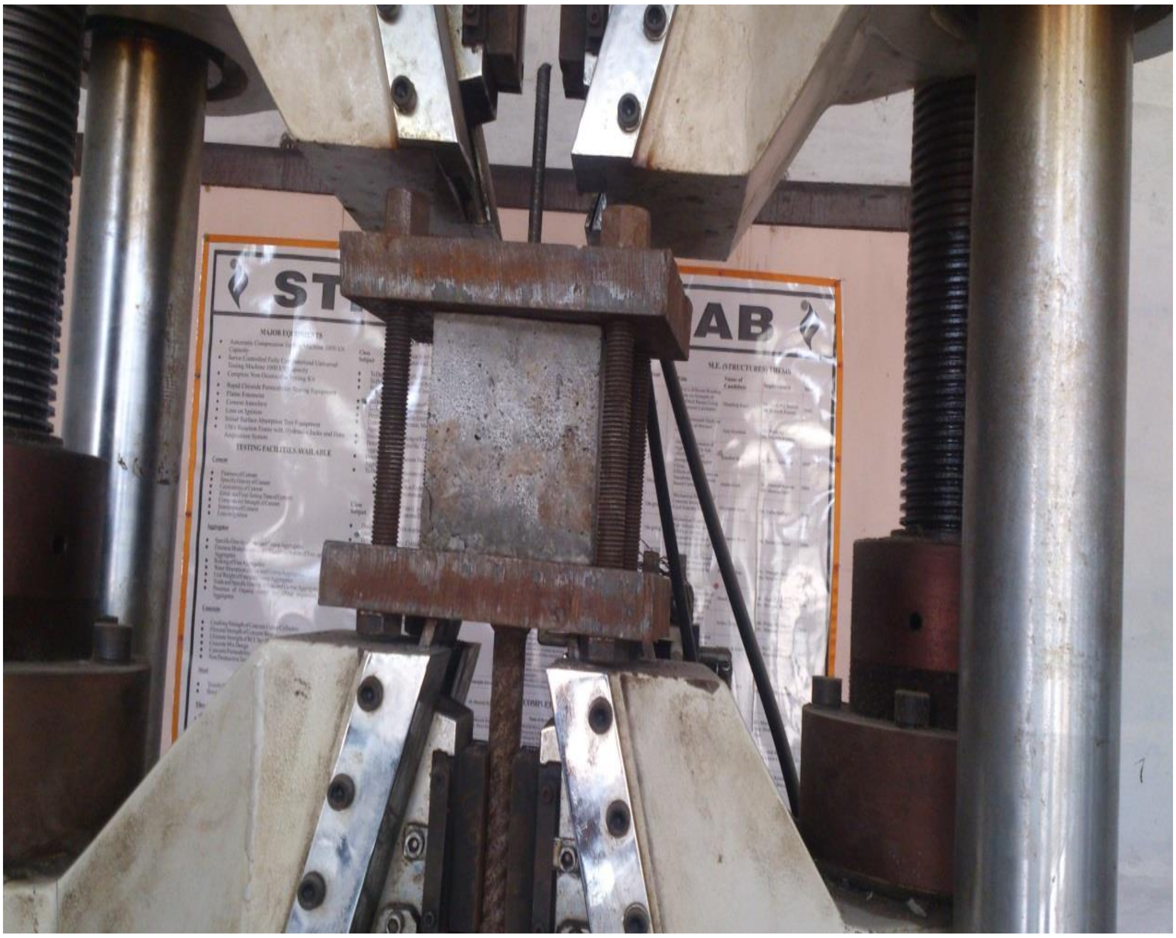
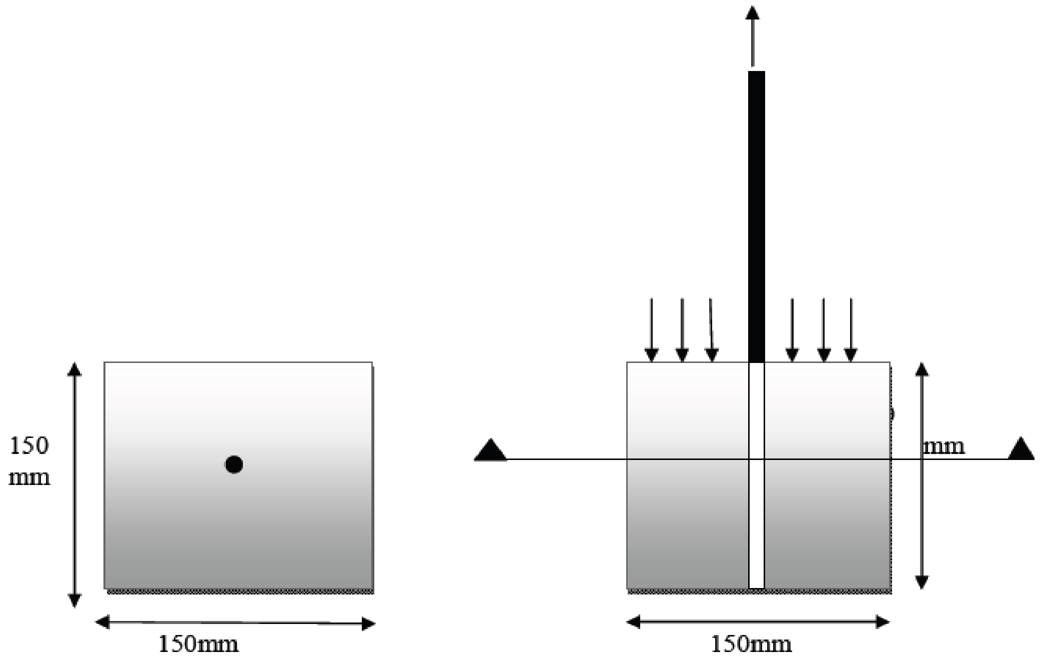


Figure 3.9: Test Setup for Pull Out Strength Test

To carry out test, the rod of 25 mm is welded to lower plate of the setup is fixed in the lower jaw of the machine. The specimen is kept in between the two plates and is fixed by tightening the nut-bolts. The rebar (12mm, 16mm, 20mm) embedded in the specimen is passed through the hole in the upper plate and is fixed to the upper jaw of the universal testing machine. The rebar is pulled out at the rate of 2.25 kg/min for all test specimens.

Figure 3.10 shows the dimensions of pull out test specimens, the location and the length of embedded deformed steel bar in concrete specimens. The bond strength is calculated by dividing the pull out force by the surface area of the embedded length of steel bars as follows:



Top view **Side view**

Figure 3.10: The dimensions of pull out test specimens, the location and the length of embedded deformed steel bar in concrete specimens

$$\text{Bond Strength} = \frac{P}{\pi D L}$$

Where,

P = pull out load, (KN)

D = Diameter of rebar, it is kept as 12mm, 16mm and 20mm for experimental programme.

L = embedment length, the value is taken as 60mm, 80mm and 100mm for 12mm,16mm and 20mm respectively for experimental programme.

The pull out strength test is carried out at the age of 28 days. The different samples are tested for each mix at the specified age. After completion of test, sample was removed from the test setup, physical verification of crack and type of slip is observed as shown in Fig.3.11. It is observed that specimen failed by pull out rather than by splitting of concrete.

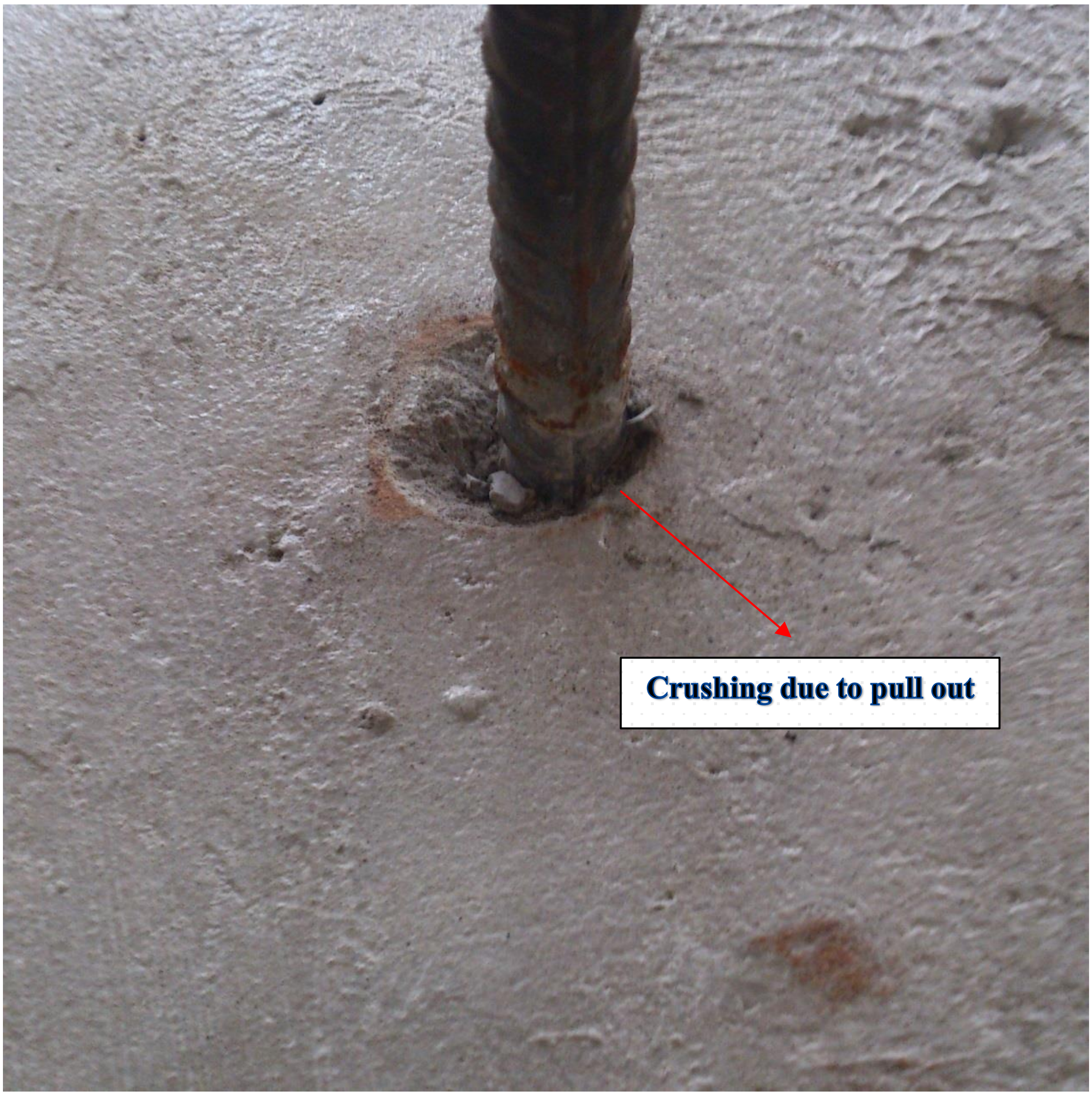


Figure 3.11: Crushing due To Pull out Bar Lugs after Pull out Test

3.6 ANALYTICAL PROGRAMMING

Experimental work on bond strength has been compared analytically using Genetic Programming by producing different models.

3.6.1 General

Genetic Programming (GP) is a domain independent, problem-solving approach in which computer programs (which in general are the equations) are evolved to find solutions to the problems. The solution technique is based on the Darwinian principle of ‘survival of the fittest’. Genetic Programming (GP) is similar to genetic algorithm

(GA) but unlike the latter its solution is a computer program or an equation as against a set of numbers in the GA (Gaur and Deo, 2008).

Genetic programming is a search technique that explores the space of computer programs. The search for solutions to a problem starts from a group of points (random programs) in the search space. Those points that are of above average quality are then used to generate a new generation of points through crossover, mutation, reproduction and possibly other genetic operations. This process is repeated over and over again until a termination criterion is satisfied. Because genetic programming is a stochastic search technique, in different runs different trajectories are observed.

Therefore, many real runs are performed and variations of certain numerical descriptors (like the average fitness or the average size of the programs in the population at each generation, the average difference between parent and offspring fitness, etc) are recorded. The result which is finally compatible with the empirical observation is considered.

This exercise is very error prone, though, because a genetic programming system is a complex adaptive system with zillions of degrees of freedom. So, any small number of statistical descriptors is likely to be able to capture only a tiny fraction of the complexities of such a system.

3.6.2 Primitives of Genetic Programming

Every solution evolved by GP is assembled from two sets of primitive nodes; terminals and functions. The terminal set contains nodes that provide an input to the GP system. The terminal set contains the arguments for the function and consists of variables or numerical constants. The function set contains nodes that process values already in the system which may be arithmetical operations such as +, -, mathematical operations such as log, exponential, sin, cos etc. Constants may also be used in GP by including them in the terminal set. The primitives of GP, the function and terminal nodes, are then assembled into a structure before they are executed.

3.6.3 Algorithm of Genetic Programming

The genetic programming paradigm breeds computer programs to solve problems by executing the following three steps:

1. An initial population of individual computer programs is randomly created composing the available functions and terminals.

2. The initial population is now tested for its fitness. The best fitted individual program is then selected for participating in the genetic operations to be conducted to

form a new population. Fitness may be measured in many different ways such as coefficient of determination (COD), root mean square error (RMS), Unit Error (deviation from dimensional error) and fitness per node (measurement of the simplicity of the expression of the individuals). The population can be tested by some or all the fitness parameters mentioned above.

Various genetic operators are now applied to the best fitted individual program to form a new individual program(s) for the new population. There are three major evolutionary operators within a GP system:

Reproduction: In this process it selects an individual from within the current population to be copied exactly into the next generation. In fig.3.12 individual (a) of the current population is as such copied into the new population as the individual (a) in fig.3.13.

Crossover: mimics sexual recombination in nature, where two parent solutions are chosen and parts of their sub tree are swapped. In fig. 3.12 individuals (a) and (b) are the two parents chosen for the crossover operation. The + function of parent (a) and leftmost terminal x of parent (b) are swapped to form individuals (c) and (d) of the new population in fig.3.13.

Mutation: causes random changes in an individual before it is introduced into the subsequent population. Unlike crossover, mutation is asexual and thus only operates on one individual. During mutation, either all functions and terminals are removed beneath an arbitrarily determined node and a new branch is randomly created, or a single node is swapped for another. In fig 3.12 individual (c) is muted wherein terminal 2 is picked as the mutation site and a subtree is inserted in its place which again is randomly created to form a individual (b) of the new population in fig.3.13.

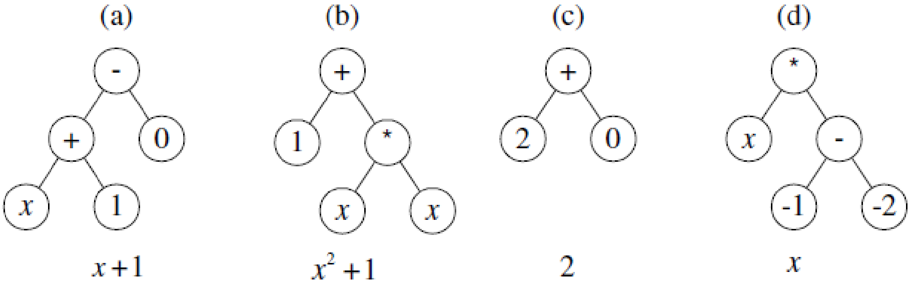


Figure 3.12: Initial Population of four Randomly Created Individuals

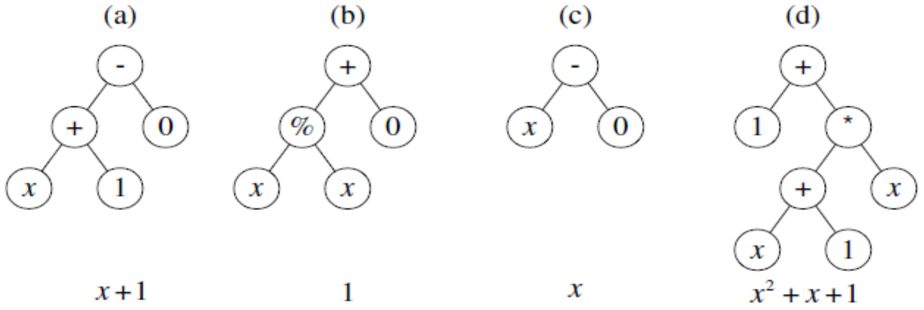


Figure 3.13: New population (a) after Reproduction, (b) after Mutation and (c) & (d) after Crossover operation

After the above mentioned operations are performed on the current population, the population of offspring (i.e., the new generation) replaces the old population (i.e., the old generation).Each individual in the new population of computer programs is then again measured for fitness, and the process is repeated for many generations. However, the genetic programming is a never ending process and therefore it is required to define some control parameters. Some of the control parameters that are required to be set are as follows:-

1. **Population size**: A larger population allows a greater exploration of the problem at each generation and increases the chance of evolving a solution.
2. **Maximum number of generations**: The greater the maximum number of generations the greater the chance of evolving a solution. However, even if after the evolution of a population, a solution is not found then it is better to start again with a different initial population. However if, after a user-defined number of generations, a sufficiently successful individual has not evolved then the process should stop.
3. **Probability of crossover** is the proportion of the population that will undergo crossover before entering the new population. If the probability of crossover is 0.90 it means that the crossover is performed on 90% of the population for each generation.
4. **Probability of reproduction** is the proportion of individuals in a population that will undergo reproduction.

3.6.4 Advantages of Genetic Programming

A key advantage of GP when compared to traditional modelling approaches is that it does not assume a prior functional form of the solution. For instance, in a typical regression method, the model structure is specified in advance (which is in general difficult to do) and the model coefficients are determined. For neural networks, the time consuming task of initially defining the network structure has to be undertaken and then the coefficients (weights) are found by the learning algorithm. On the other hand, in GP, the building blocks (the input and target variables and the function set) are defined initially, and the learning method subsequently finds both the optimal structure of the model and its coefficients. Moreover, since GP evolves an equation or formula relating to the input and output variables, a major advantage of the GP approach is its automatic ability to select input variables that contribute beneficially to the model and disregard those that do not. GP can thus reduce substantially the dimensionality of the input variables. However, a common limitation of GP is that it cannot handle more numbers of constants. In GP, as in any data-driven prediction model, the selection of appropriate model input is extremely important. Inclusion of irrelevant input parameters leads to poor model accuracy and creation of complex models, which are more difficult to interpret as compared to simpler ones.

Flowchart for the genetic programming paradigm has been presented in figure 3.14 (Koza, 1992).

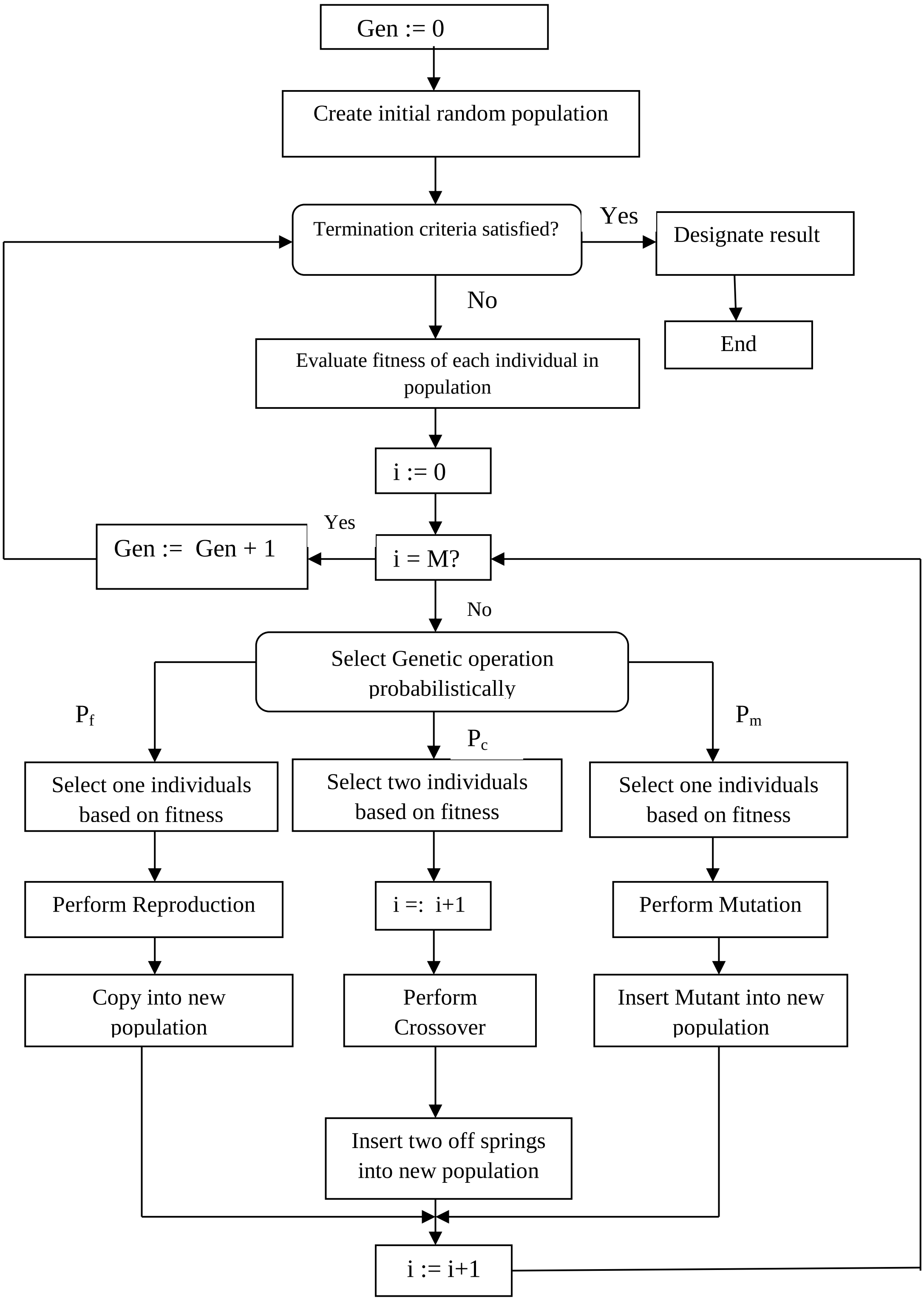


Figure 3.14: Flow Chart of Genetic Programming

3.7 CLOSING REMARKS

The experimental programme described in this chapter includes the significant material properties and specifications of the ingredients of concrete; reinforcement steel etc., the testing procedure and the analytical procedure of generating a mathematical expression using GP have been discussed.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

In the first part of this chapter, the effect of replacement ratio of recycled coarse aggregate on compressive strength of concrete is discussed. The effect is studied at a range of 5 different w/c ratios. The second part consists of discussion on the effect of recycled coarse aggregate on the bond strength of concrete. The third part consists of the comparison of bond strength calculated by Genetic Programming.

4.2 COMPRESSIVE STRENGTH

Three cubes (150mm×150mm×150mm) from each batch of concrete mix are casted and cured for 7 and 28 days in order to determine compressive strength of RCA concrete. All specimens are cast in a single mix and direct weight to weight replacement of natural coarse aggregate is carried out with recycled coarse aggregate at a replacement ratio of 0, 30, 60, and 90 %. The mixes are casted at water-cement ratio of 0.42, 0.45, 0.48, 0.51 and 0.55. This corresponds to range of strength varying from low strength concrete to moderate strength concrete. Table 4.1 shows the value of compressive strength of cube tested at 7 and 28 days. The data is further represented in the form of bar graphs in Figure4.1 - 4.5, for water- cement ratio of 0.42, 0.45, 0.48, 0.51 and 0.55 respectively. The results obtained are discussed in the following sections:

4.2.1 Effect of Recycled Coarse Aggregate on Compressive Strength

It was observed that for all water-cement ratios, the 28 days compressive strength increased as the percentage of replacement increases and this trend can be seen in figure 4.1-4.5. Maximum compressive strength is achieved at 90% replacement at all water-cement studied. However the same trend is not seen at 7 days strength, where the strength decreases initially and then increases. The final compressive strength is lesser than the compressive of control mixes. It may be because at 7 days, the hydration is not complete.

Table 4.1: Cube Compressive Strength at 7 days and 28 days

Specimen ID	Water-cement ratio	Replacement (%)	Compressive strength	
			7 days (MPa)	28 days(MPa)
A1-0	0.42	0	40.75	49.07
A1-30		30	35.80	43.65
A1-60		60	39.70	50.06
A1-90		90	40.70	52.31
A2-0	0.45	0	36.37	41.68
A2-30		30	29.13	41.28
A2-60		60	33.75	45.64
A2-90		90	35.50	50.66
A3-0	0.48	0	32.20	36.04
A3-30		30	26.04	39.65
A3-60		60	28.08	43.15
A3-90		90	31.56	44.73
A4-0	0.51	0	28.67	32.48
A4-30		30	19.07	33.30
A4-60		60	25.60	38.78
A4-90		90	28.03	40.97
A5-0	0.55	0	23.02	24.89
A5-30		30	16.40	30.03
A5-60		60	18.90	35.86
A5-90		90	19.87	37.68

Therefore, the bond between recycled aggregate and new concrete paste has not developed yet. The failure in this case will occur in the interfacial transition zone (ITZ) which is weaker in recycled coarse aggregate concrete as is observed by Xiao et al, (2012).

This trend of decrease in strength is very prominent at 30% replacement level. It may be due to at 30% replacement level, we have concrete with two types of aggregate (i.e. natural and recycled) therefore, and at this level interfacial transition zone is of mixed characteristic, which is rather playing a negative role in overall behaviour of concrete. It can be concluded that if recycled coarse aggregate are used, they must be

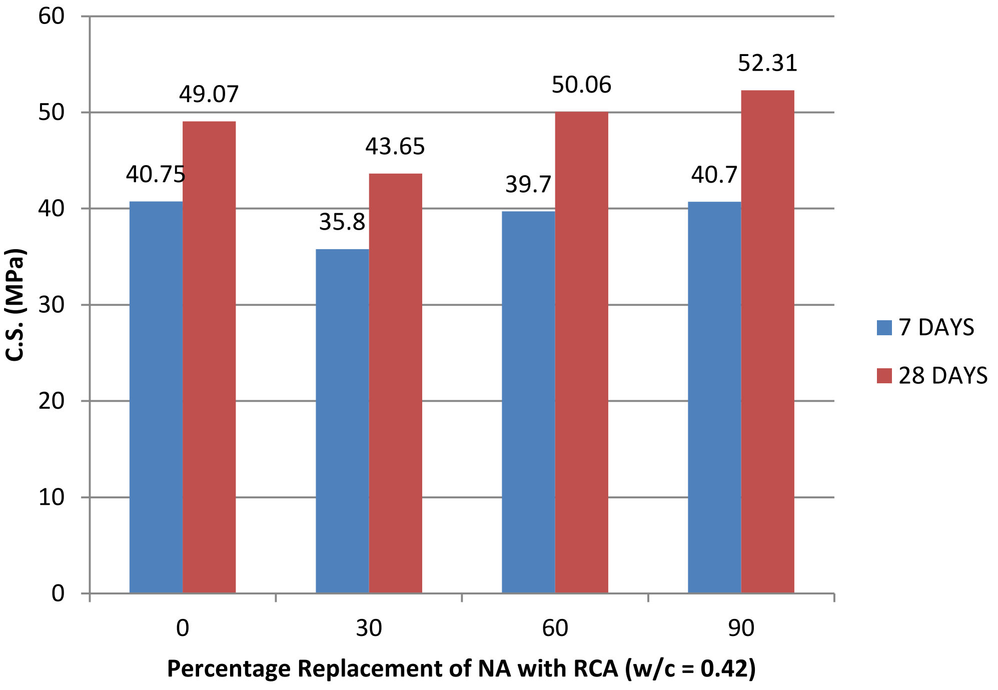


Figure 4.1: 7 and 28 days Compressive Strength for w/c ratio 0.42

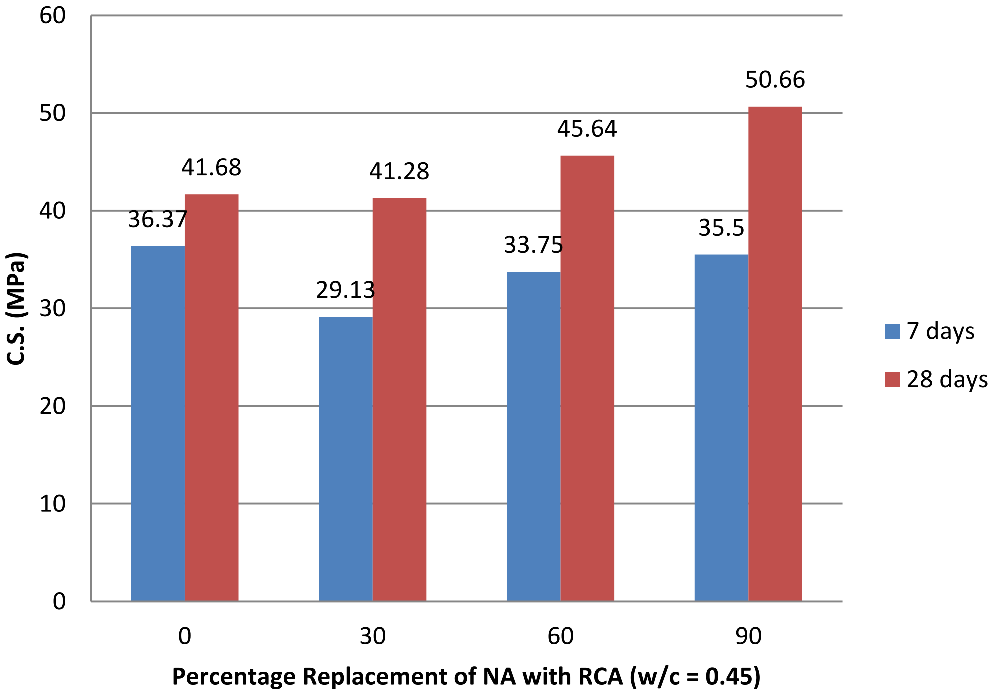


Figure: 4.2: 7 and 28 days Compressive Strength for w/c ratio 0.45

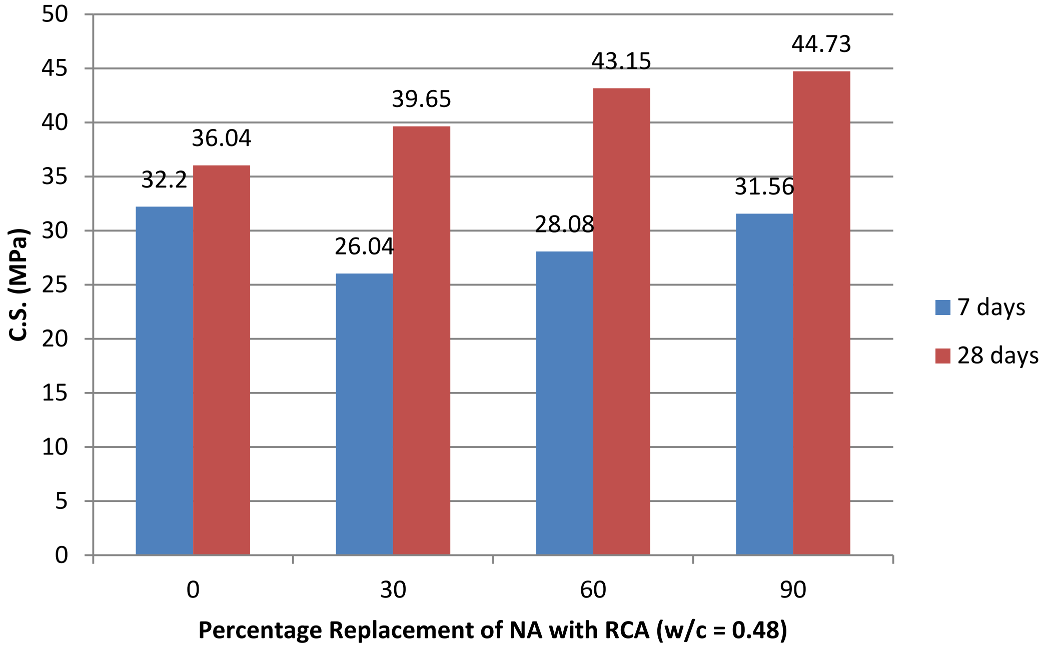


Figure 4.3: 7 and 28 days Compressive Strength for w/c ratio 0.48

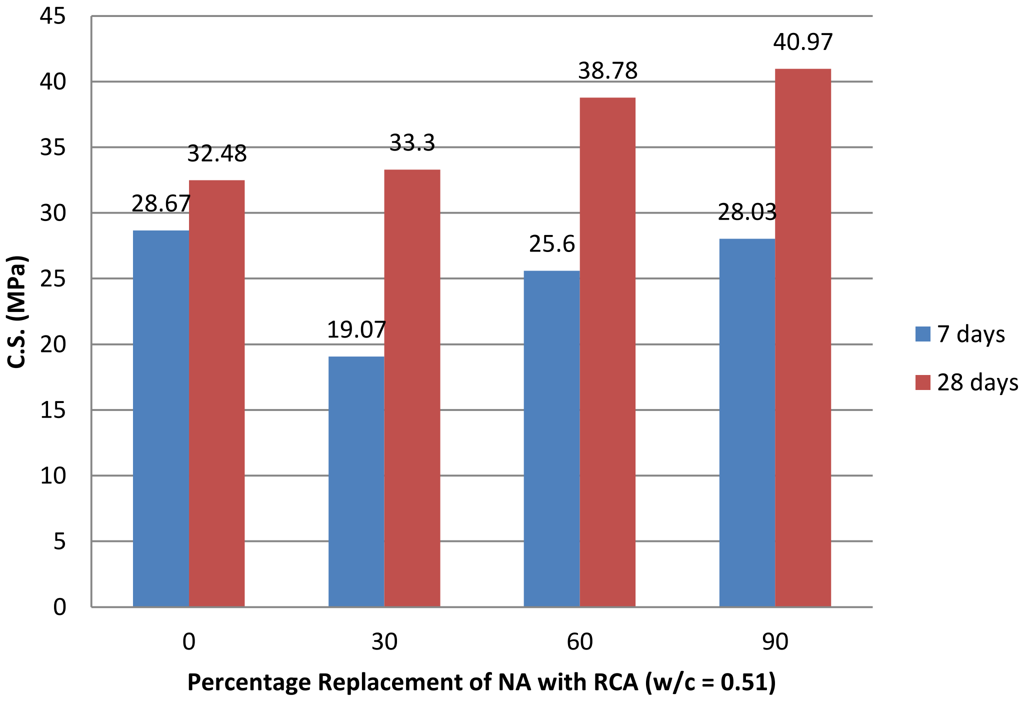


Figure 4.4: 7 and 28 days Compressive Strength for w/c ratio 0.51

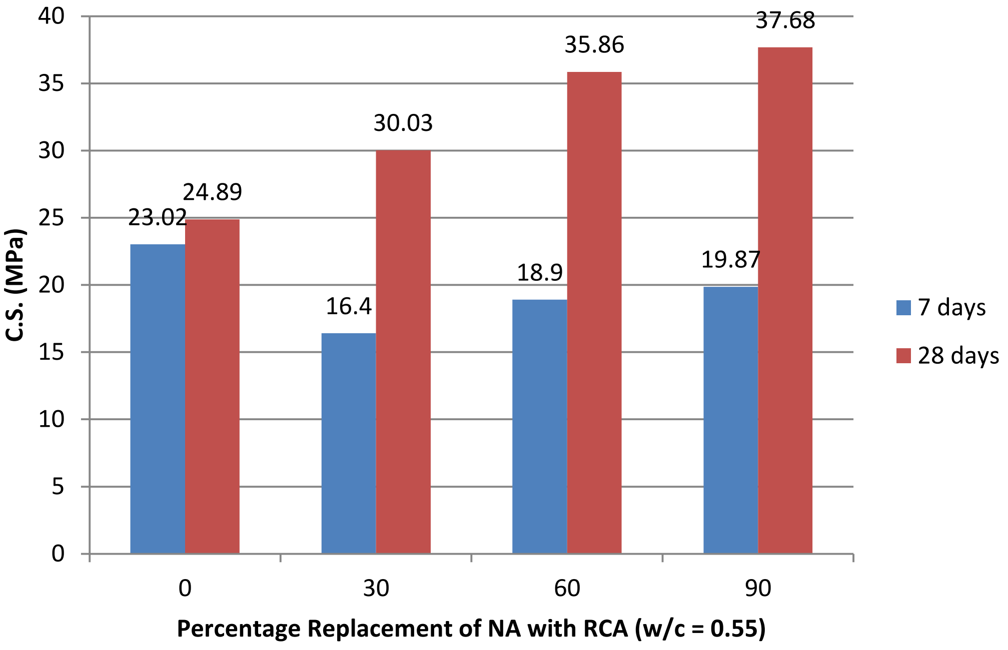


Figure4.5: 7 and 28 days Compressive Strength for w/c ratio 0.55

used at higher replacement levels. In fact, 90% replacement levels give the maximum efficiency at 28 days.

As we increase the recycled aggregate content the, 28 day compressive strength increase. It is because at high recycled aggregate content, these aggregate absorb more water. Therefore, the effective water- cement ratio decreases and hence the strength increases. The similar trend is observed by Butler et al. (2011). On the basis of microstructure studies, they further concluded that recycled coarse aggregate improve the interfacial transition zone between the new mortar and aggregate. This improvement is due to more roughed surface texture of recycled coarse aggregate particles as compare to natural aggregate.

Also, the hydration products formed will penetrate deep into the cracks of recycled coarse aggregate, thus improves the ITZ further. Similar observation was made by Kou et al. (2011) when recycled coarse aggregate was used along with various mineral admixtures.

4.2.2 Effect of Water-Cement Ratio on Compressive Strength

It was observed that as water-cement ratio decreases, compressive strength increases. Rate of increase of compressive strength for recycled coarse aggregate concrete with water-cement ratio is not as high as the corresponding rate when only natural aggregate are used. It is because strength gain with the use of recycled aggregate is very high at higher water-cement ratios. And as the water-cement ratio reduces, the strength gain is not very prominent as can be seen from figure 4.6. Similar trend was obtained by Singla S. (2013) and Rahal (2007).

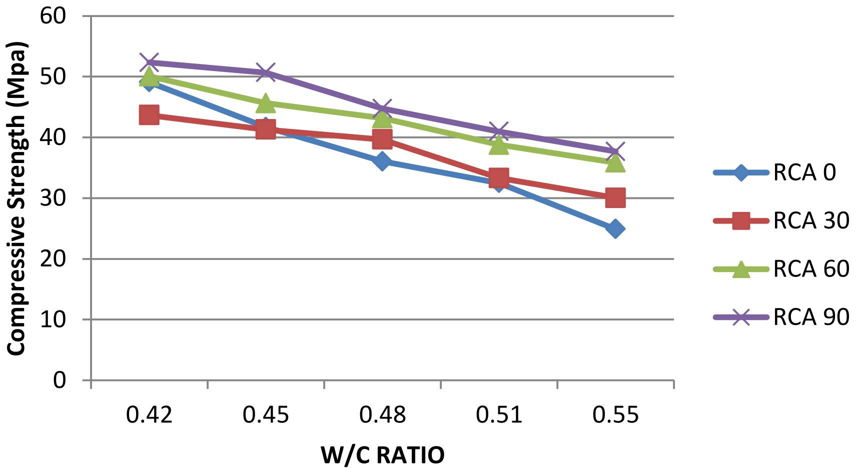


Figure 4.6: 28 days Compressive Strength vs W/C Ratio

4.3 BOND STRENGTH

Three cubes for each batch of concrete mix are casted and cured for 28 days in order to determine bond strength of all the mix concrete. The mixes are casted at water-cement ratio 0.42, 0.45, 0.48, 0.51 and 0.55. Table 4.2 shows bond strength of cubes tested at 28 day curing. The data is further represented systematically in the form of bar graphs as shown in Figure4.7, 4.8, and 4.9. Bond strength between concrete and deformed steel bars increases as percent of recycled aggregate increases. The results obtained are discussed in the following sections:

4.3.1 Effect of RCA on Bond Strength

It was discovered that with the increase of recycled coarse aggregate replacement initially bond strength decreases and then increases. Bond strength was maximum at replacement level of 90% coarse aggregate. Similar trend was observed for all water-cement ratios. This increase in bond strength was may be due to same modulus of elasticity of recycled coarse aggregates and the cement paste of recycled coarse aggregate concrete which at the level of concrete microstructure should improve composite action between these two phases and reduce incompatibilities of deformations under applied loads as suggested by Poon et al. (2004).

Table 4.2: Bond Strength obtained of all the 3 rebars used (12mm, 16mm, 20mm)

Specimen ID	Replacement ratio	w/c ratio	Pull out load (kN)			Bond strength (MPa)		
			12mm	16mm	20mm	12mm	16mm	20mm
A1-0	0	0.42	46.5	84.64	86.39	20.56	22.29	13.75
A1-30	30		42.95	84.65	68.43	18.99	21.05	10.89
A1-60	60		48.18	93.03	98.97	21.31	23.13	15.75
A1-90	90		51.21	96.53	102.43	22.20	24.00	16.31
A2-0	0	0.45	31.28	81.51	72.99	13.83	20.07	11.62
A2-30	30		26.40	80.25	54.53	11.67	19.95	8.68
A2-60	60		34.93	82.00	70.50	12.35	20.39	11.22
A2-90	90		44.58	84.55	99.78	19.87	21.02	15.88
A3-0	0	0.48	26.50	77.25	62.80	11.72	19.21	10.00
A3-30	30		24.97	73.16	60.34	11.04	18.19	9.60
A3-60	60		37.50	79.10	85.98	16.58	19.67	13.69
A3-90	90		48.31	82.20	95.10	21.80	20.44	15.31
A4-0	0	0.51	31.38	68.50	60.90	13.87	17.03	9.69
A4-30	30		31.38	64.12	74.91	13.87	15.94	11.92
A4-60	60		33.67	72.25	82.23	14.89	17.96	13.09
A4-90	90		42.00	74.25	93.65	18.57	18.46	14.91
A5-0	0	0.55	34.53	57.13	64.10	15.27	14.20	10.20
A5-30	30		30.71	52.50	62.36	13.58	13.05	9.93
A5-60	60		37.12	67.16	80.27	16.41	16.70	12.78
A5-90	90		40.91	69.52	90.69	18.09	17.28	14.44

4.3.2 Effect of Water-Cement Ratio on Bond Strength

It is observed that bond strength depends on water-cement ratio which is illustrated by Figure 4.7, 4.8 and 4.9. As water-cement ratio decreases bond strength increases or vice versa. The bond strength of control mix and 90% RCA mix shows a significant increase in the respective figures. But this trend of increase in bond strength between control mix (0% RCA) and design mix (90% RCA) is up to the w/c ratio of 0.48. This may be because the adhered mortar present on RCA absorb the water and thereby decreasing w/c ratio and resulting into shrinkage and this curing action of RCA increases the bond strength between recycled coarse aggregate concrete and the rebar as suggested by Prince et al. (2013). When w/c ratio is greater than 0.48 i.e. 0.51 and

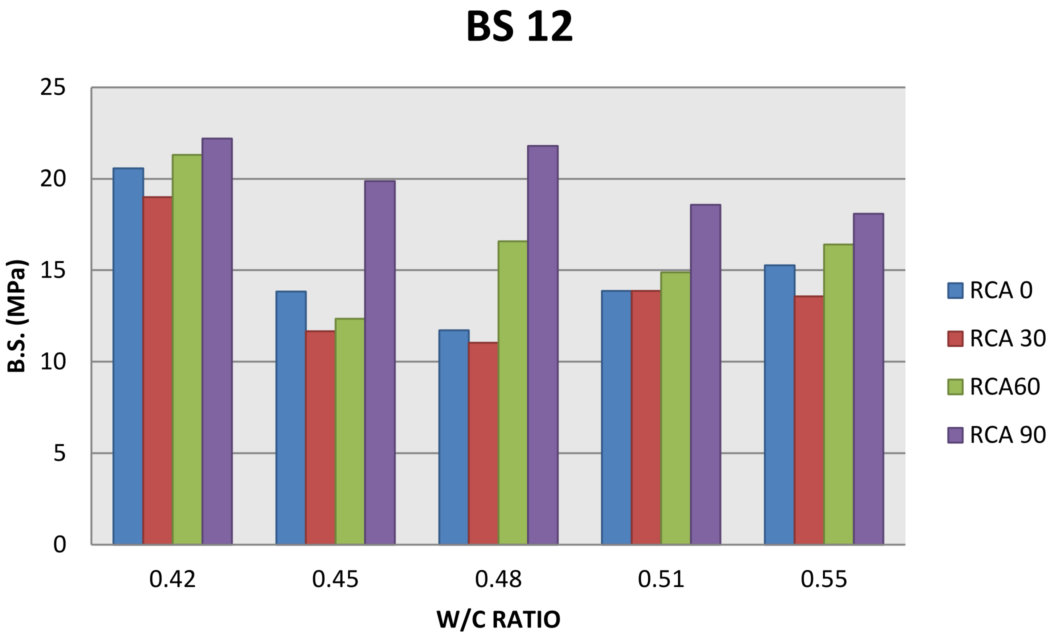


Figure 4.7: Variation of bond strength with RCA replacement percentage for all w/c ratios for 12mm rebar

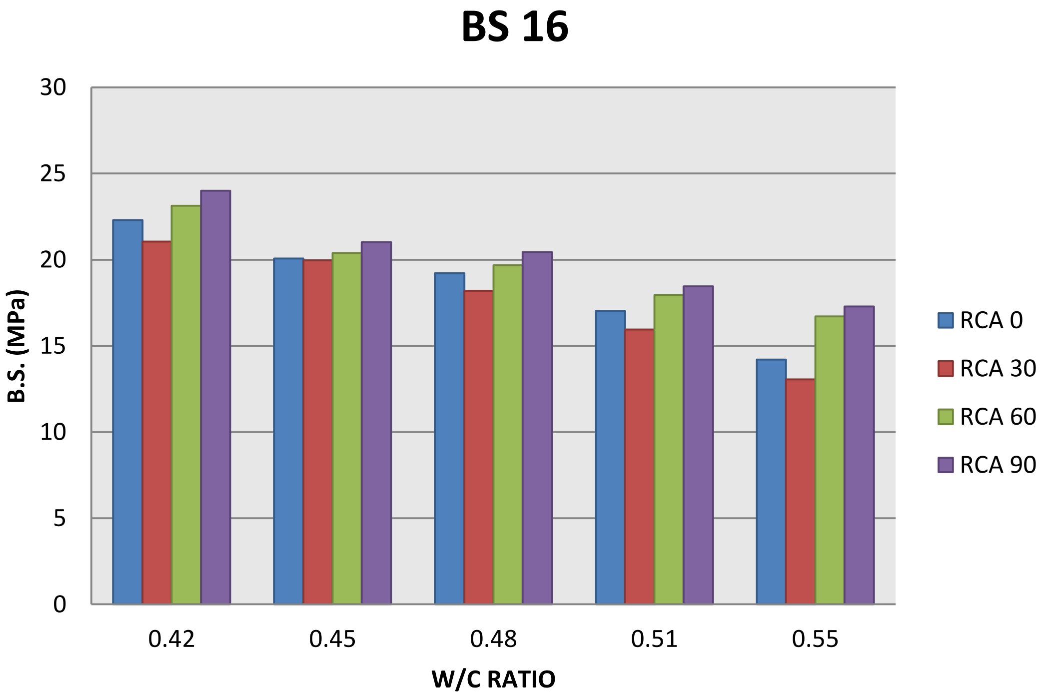


Figure 4.8: Variation of bond strength with RCA replacement percentage for all w/c ratios for 16mm rebar

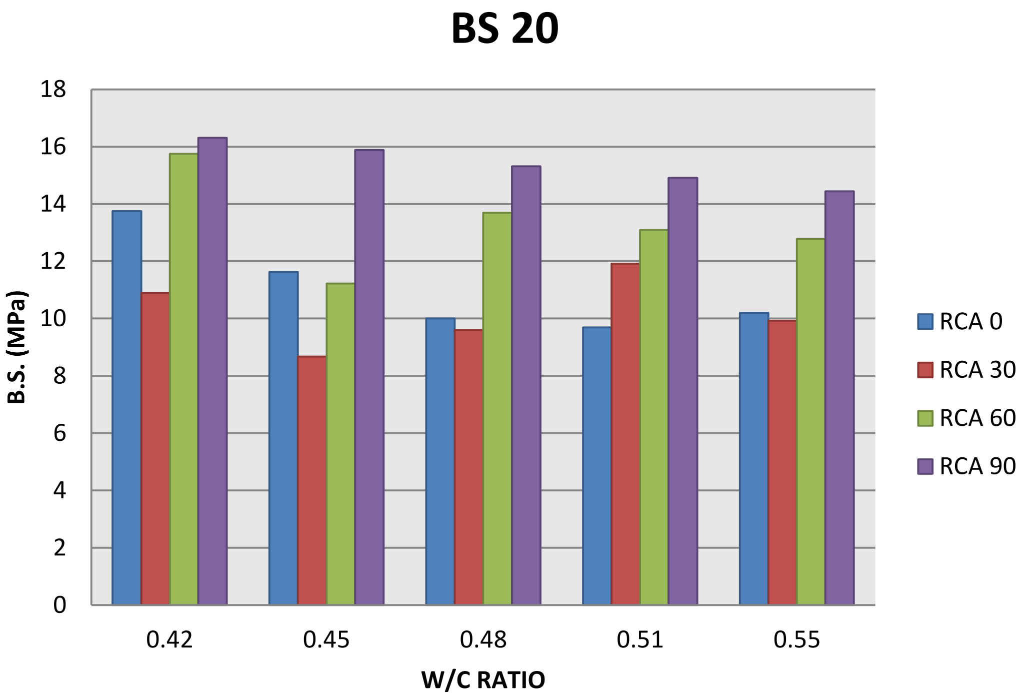


Figure 4.9: Variation of bond strength with RCA replacement percentage for all w/c ratios for 20mm rebar

0.55, the excess water now is consumed by RCA for its saturation and hence w/c ratio is maintained so the shear interlock between the surrounding concrete and deformed bar is not that much strong due to less shrinkage confining effect of RCA. All this behaviour of RCA is due to high water absorption percentage and RCA used in this experimental work is Air Dried (AD), so the water absorption capacity has increased and this increased water absorption percentage has led to the increase in shrinkage confining effect. It also depends on the mechanical anchorage ends of the rebar and friction resistance to sliding and interlock as the reinforcing element is subjected to tensile stress. The bond strength can therefore be considered to be strongly influenced by recycled aggregate ratio, w/c ratio and the concrete type.

4.4. COMPARISON OF GP RESULTS AND EXPERIMENTAL RESULTS

4.4.1 Mathematical Relation Generated by GP

The equation for bond strength obtained by GP by using various factors:

- Water cement ratios (0.42, 0.45, 0.48, 0.51, 0.55)
- Replacement ratios
    - 100% NCA + 0% RCA
    - 70% NCA + 30% RCA

    - 40% NCA + 60% RCA
    - 10% NCA + 90% RCA

    (NCA: Natural Coarse Aggregate; RCA: Recycled Coarse Aggregate)

- Diameter:
    - 12 mm ϕ deformed bar
    - 16 mm ϕ deformed bar (from previous thesis)
    - 20 mm ϕ deformed bar

The equation obtained is

$$B.S = \frac{x}{3x - y^2(1 + 0.54x^2) - z + 11.14} \times 100$$

x = CS/D (Compressive strength & diameter)

y = RR (Replacement ratio)

z = WC (water- cement ratio)

4.4.2 Application of Genetic Programming for Predicting Bond Strength

The Replacement ratio, water cement ratio, compressive strength, diameter have been used as input parameters and bond strength (experimental) as the output parameter. The data for various bond strength has been presented in table 4.3.

The equations for bond strength have been developed using the values of input and output parameters for various combinations of crossover rate, number of generations, population size and no. of children to produce. This process is continued until the most accurate equation is obtained by reaching the maximum number of generations. The statistic measures used to assess the accuracy of these equations are Coefficient of Determination (COD) and Root Mean Square (RMS) wherein the objective is to have COD nearly equal to 1 and RMS nearly equal to 0.

The values obtained from the equation were compared with the experimental results for different diameter deformed bars (12mm, 16mm and 20mm). Typical plots for various parameters are obtained showing the comparison of bond strength by GP and bond strength by experimental results presented in figure 4.10- 4.13.

Table 4.3: Input values used to generate the bond strength by GP

Input Values

RR	WC	CS/D	BS (expt)
0	0.42	4.089167	20.56
0.3	0.42	3.6375	18.99
0.6	0.42	4.171667	21.31
0.9	0.42	4.359167	22.2
0	0.45	3.473333	13.83
0.3	0.45	3.44	11.67
0.6	0.45	3.803333	12.35
0.9	0.45	4.221667	29.87
0	0.48	3.003333	11.72
0.3	0.48	3.304167	11.04
0.6	0.48	3.595833	16.58
0.9	0.48	3.7275	21.8
0	0.51	2.706667	13.87
0.3	0.51	2.775	13.87
0.6	0.51	3.231667	14.89
0.9	0.51	3.414167	18.57
0	0.55	2.074167	15.27
0.3	0.55	2.5025	13.58
0.6	0.55	2.988333	16.41
0.9	0.55	3.14	18.09
0	0.42	3.066875	22.29
0.3	0.42	2.728125	21.05
0.6	0.42	3.12875	23.13
0.9	0.42	3.269375	24
0	0.45	2.605	20.07
0.3	0.45	2.58	19.95
0.6	0.45	2.8525	20.39
0.9	0.45	3.16625	21.02
0	0.48	2.2525	19.21
0.3	0.48	2.478125	18.19
0.6	0.48	2.696875	19.67
0.9	0.48	2.795625	20.4
0	0.51	2.03	17.03
0.3	0.51	2.08125	15.94
0.6	0.51	2.42375	17.96
0.9	0.51	2.560625	18.46
0	0.55	1.555625	14.2
0.3	0.55	1.876875	13.05

0.6	0.55	2.24125	16.7
0.9	0.55	2.355	17.28
0	0.42	2.4535	13.75
0.3	0.42	2.1825	10.89
0.6	0.42	2.503	15.75
0.9	0.42	2.6155	16.31
0	0.45	2.084	11.62
0.3	0.45	2.064	8.69
0.6	0.45	2.282	11.22
0.9	0.45	2.533	15.88
0	0.48	1.802	10
0.3	0.48	1.9825	9.6
0.6	0.48	2.1575	13.69
0.9	0.48	2.2365	15.13
0	0.51	1.624	9.69
0.3	0.51	1.665	11.92
0.6	0.51	1.939	13.09
0.9	0.51	2.0485	14.91
0	0.55	1.2445	10.2
0.3	0.55	1.5015	9.93
0.6	0.55	1.793	12.78
0.9	0.55	1.884	14.44

Table 4.4: Output Values obtained using GP

BS(expt)	BS (GP)
20.56	17.78838
18.99	17.40814
21.31	21.42716
22.2	29.81793
13.83	16.45323
11.67	16.91141
12.35	20.11386
29.87	28.72588
11.72	15.26833
11.04	16.56338
16.58	19.3772
21.8	24.99353
13.87	14.43529
13.87	15.0093
14.89	18.03266
18.57	22.86883
15.27	12.33679
13.58	14.1373

16.41	17.12764
18.09	21.1331
22.29	15.39521
21.05	14.78616
23.13	17.54712
24	21.77964
20.07	14.07701
19.95	14.32169
20.39	16.49299
21.02	21.15903
19.21	12.93214
18.19	13.99737
19.67	15.89446
20.4	18.89537
17.03	12.14089
15.94	12.55873
17.96	14.78639
18.46	17.52097
14.2	10.19599
13.05	11.7609
16.7	14.03334
17.28	16.35088
13.75	13.56961
10.89	12.88008
15.75	15.03928
16.31	17.73922
11.62	12.30054
8.69	12.44581
11.22	14.12385
15.88	17.28571
10	11.21599
9.6	12.14356
13.69	13.6019
15.13	15.57775
9.69	10.47583
11.92	10.8119
13.09	12.63073
14.91	14.51006
10.2	8.688307
9.93	10.08088
12.78	11.96921
14.44	13.58408

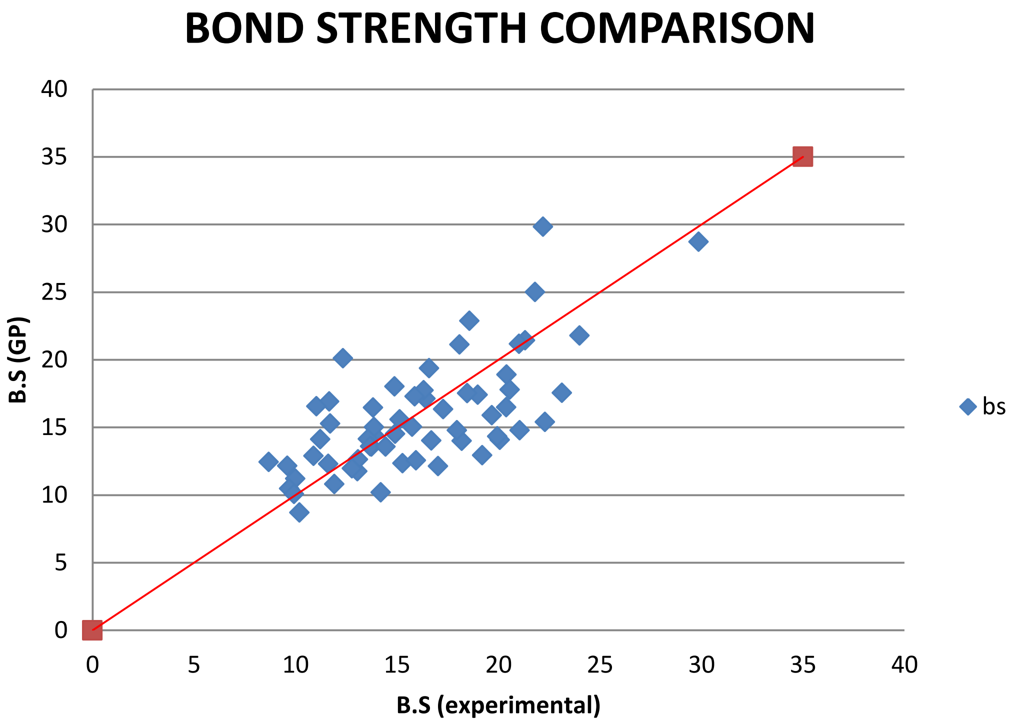


Figure 4.10: Comparison of Bond Strength obtained experimentally and by GP for rebar diameters (12mm, 16mm & 20mm)

It can be observed from these plots that the values obtained from GP are more or less matching with the experimental values. From the comparison the maximum error to be observed was 13%. The values obtained for COD is equal to 0.48 and that of RMS is 0.03. The plot between bond strength experimental and bond strength by GP shows that some values calculated using GP are more or less equal to experimental values, and few match exactly with the experimental values which are coinciding with the line shown in figure 4.10.

This illustrates that mathematical expression obtained by using the input parameters (replacement ratio, w/c ratio, diameter and compressive strength) gives an average error of ± 13%.

It is observed from figure 4.11, that the values of bond strength calculated by experiment and Genetic Programming are found to be nearly equal and this is further explained by bar graphs that bond strength decreases as the w/c ratio increases. The average error in bond strength of experimental and GP for all the 20 design mixes having different w/c ratios (0.42, 0.45, 0.48, 0.51 and 0.55) and replacement ratios (0, 30, 60 and 90) of RCA are found out to be 15% in case of 12mm rebar.

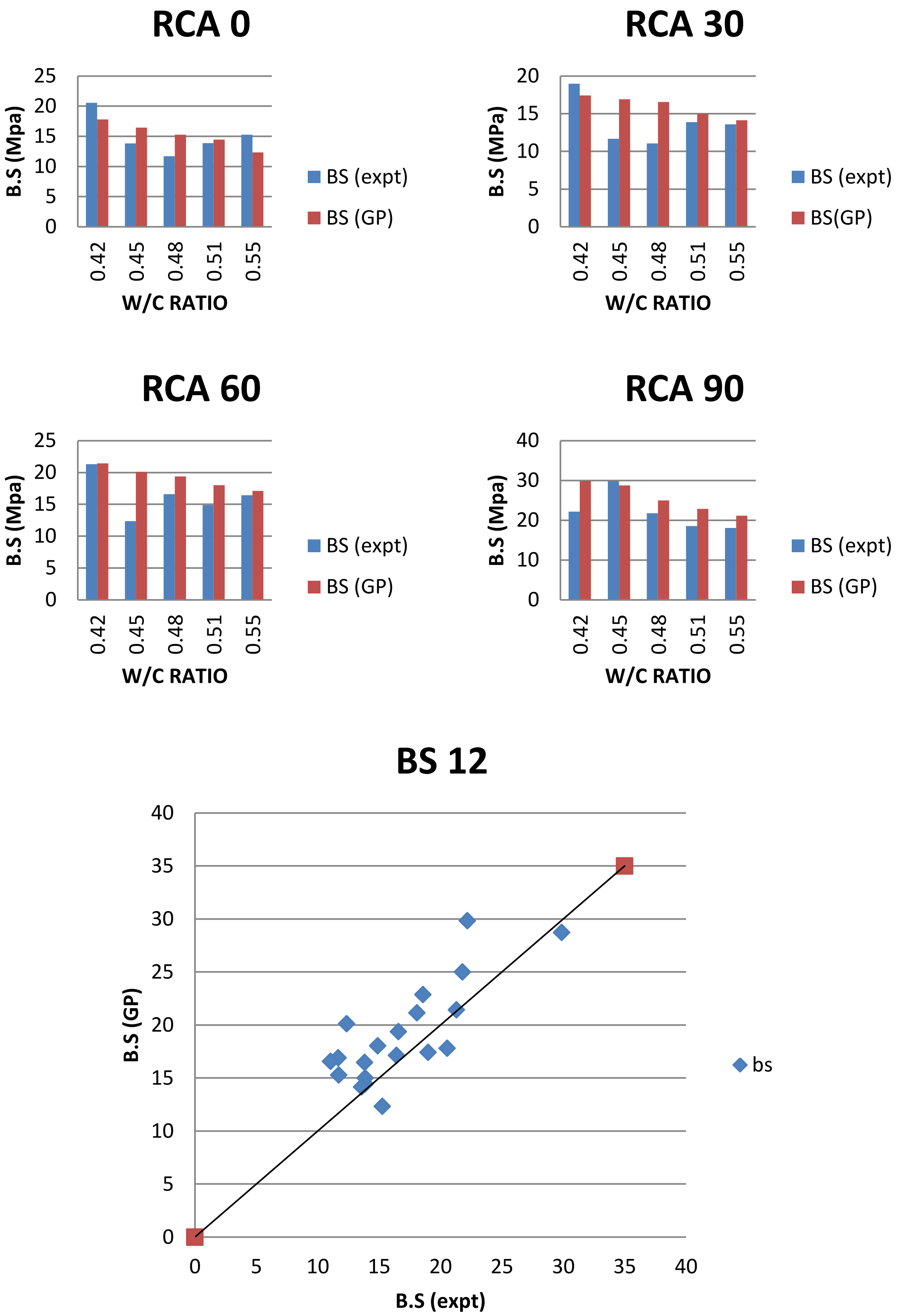


Figure 4.11: Comparison of Bond Strength obtained experimentally and by GP for 12 mm rebar

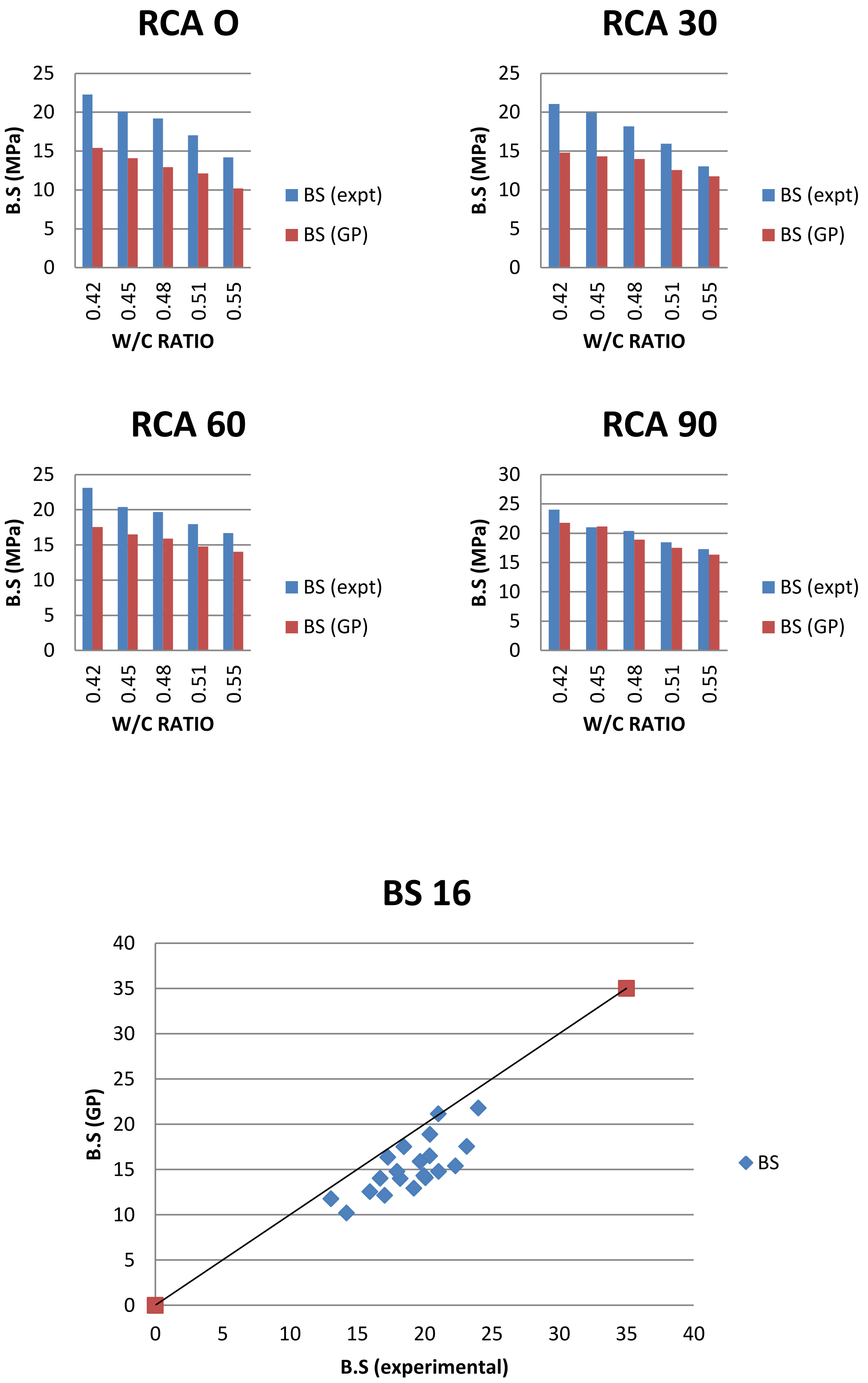


Figure 4.12: Comparison of Bond Strength obtained experimentally and by GP for 16 mm rebar

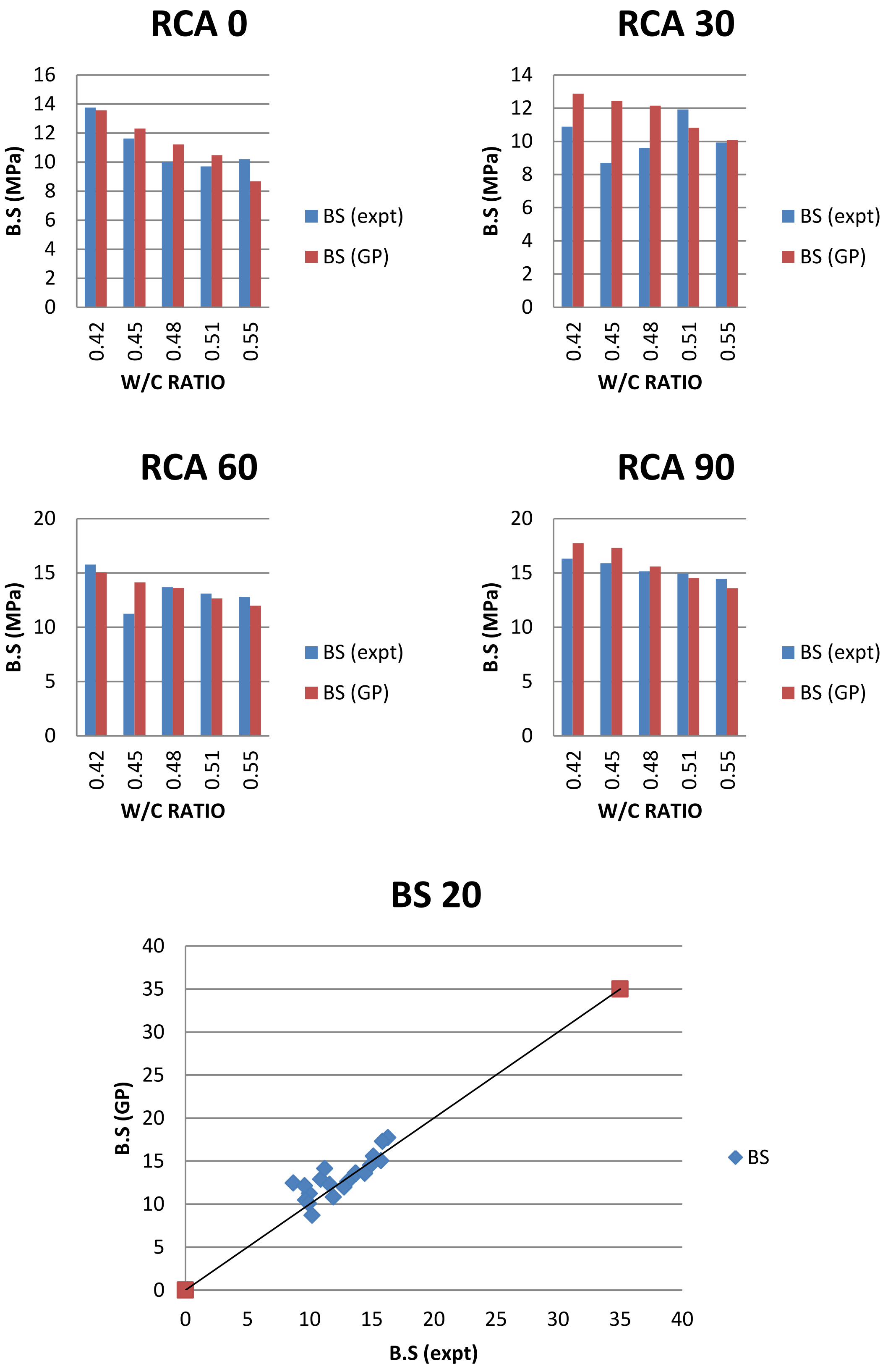


Figure 4.13: Comparison of Bond Strength obtained experimentally and by GP for 20 mm rebar

It is observed from figure 4.12, that the values of bond strength calculated by experiment and Genetic Programming are found to be nearly equal and this is further explained by bar graphs that as the w/c ratio increases bond strength decreases. The average error in bond strength calculated by experiment and GP of all the 20 design mixes having different w/c ratios (0.42, 0.45, 0.48, 0.51 and 0.55) and replacement ratios (0, 30, 60 and 90) of RCA is found out to be 19% in case of 16mm rebar.

It is observed from figure 4.13, that the values of bond strength calculated by experiment and Genetic Programming are found to be nearly equal and this is further explained by bar graphs that as the w/c ratio increases bond strength decreases. The average error in bond strength calculated by experiment and GP of all the 20 design mixes having different w/c ratios (0.42, 0.45, 0.48, 0.51 and 0.55) and replacement ratios (0, 30, 60 and 90) of RCA is found out to be 5% in case of 20mm rebar.

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4.5 CLOSING REMARKS

The result discussed in this chapter had shown that the mathematical model used to calculate the Bond Strength has obtained values nearly equal to the ones generated experimentally with an average error of 13%.

CHAPTER 5

CONCLUSION

5.1 INTRODUCTION

The main objective of the present study was to investigate the bond strength of mixes made with using different w/c ratios, replacement ratios of RCA and different diameters of deformed bars .The bond strength and the corresponding compressive strength was obtained experimentally for each mix.

The model relating bond strength to compressive strength, rebar diameter, replacement ratio and w/c ratio was generated by using Genetic Programming. Following conclusions can be drawn from the research work:

- With the increase in RCA content, the compressive strength of the mixes decreases till 30% replacement level, thereafter with further increase in RCA content, compressive strength start increasing. The maximum value of compressive strength is obtained at 90% replacement level. It can be due to high water absorption capacity of RCA, therefore the effective w/c ratio decreases and hence strength increases.
- The bond strength of RCA concrete shows a similar trend to the compressive strength. It decreases for 30% replacement and increases when RCA content is increased to 60% and above. Bond strength is maximum at 90% because the percentage of RCA is the dominating factor and due to it shrinkage confining occurs at large scale resulting into increased the bond strength.
- A mathematical expression has been generated to calculate bond strength using parameters like replacement ratio, w/c ratio, diameter of rebar and compressive strength. The final model generated by using GP relates Bond Strength to Compressive strength, diameter of rebar, w/c ratios and replacement ratios gives values which are more or less equal to the experimental values with the average error of 13%. Therefore, this model can be used for predicting Bond Strength of RCA mixes with fare accuracy.

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