

RECYCLED AGGREGATE CONCRETE AT ELEVATED TEMPERATURES

by

Bülent Çelik

B.S., Civil Engineering, Boğaziçi University, 1997

M.S., Civil Engineering, Boğaziçi University, 2001

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
the requirements for the degree of
Doctor of Philosophy

Graduate Program in Civil Engineering
Boğaziçi University

2007

To my dearest mother and father

Safiye and Fethi

To my lovely wife and daughter

Müge and Burcu

ACKNOWLEDGEMENTS

I sincerely thank to my thesis supervisor, Prof. Dr. Turan Özturan, for his valuable guidance, suggestions and continuous support throughout this thesis in his busy schedule. It is an honor for me to work with him over ten years.

I would like to express my thanks to the members of the supervisory committee, Prof. Dr. Cengiz Karakoç, Prof. Dr. Hulusi Özkul, Prof. Dr. Mehmet Ali Taşdemir, Assist. Prof. Nilüfer Özyurt, and Assist. Prof. Cem Yalçın for their advices, creative suggestions and encouragements.

I am also grateful to Prof. Dr. Neşet Acerkan, ITU Mine Engineering Department, Assoc. Prof. Dr. Gültekin Göller, ITU Metallurgical and Materials Engineering Department, Kemal Çolakoğlu, Samandra Betonsa, Nihat Girit, Grace Construction Products Co., Turgay Önal, Cebeci Dünya Beton, and Pelin Türker, Turkish Cement Manufacturers' Association for providing the necessary materials and services.

I am very grateful to my friend Ahmet Gökçe for the interest he has shown. I also would like to thank to technician Mesut Kardaş for his help during the experiments. I am thankful to my friends Savaş, Mehmet Emre, Uğur, Tümer and Osman for their valuable assistance during the testing phase of the study.

I would like to express my deepest thanks to Murat, Ramazan, Tuğba, Elif, Seda and Betül for their contributory technical and moral support. My special thanks to Arif, Kağan, Cenk, Melek and Didem are due to their friendship.

My hearted thanks goes to my parents Safiye and Fethi and my brothers Levent and Ahmet for their continuous support.

Finally, I would like to extend my deepest thanks to my lovely wife Müge and my daughter Burcu, for their patience, encouragement and love.

ABSTRACT

RECYCLED AGGREGATE CONCRETE AT ELEVATED TEMPERATURES

Recycling the demolished concrete as aggregate and reuse of the recycled aggregates in concrete can help to prevent environmental pollution and keep the natural resources. However, fresh and hardened properties of concretes produced with these recycled aggregates must be guaranteed. This study presents the effects of elevated temperatures on mechanical and durability properties of recycled aggregate concretes. Concrete and other construction materials sometimes exposed to elevated temperatures. Concrete mixtures containing recycled coarse aggregates in amounts of 25, 50, 75, and 100 percent were prepared. Test specimens were exposed to elevated temperatures up to 250 °C. After exposure, the mechanical properties were determined using compressive strength, static modulus of elasticity and splitting tensile strength tests, while the durability properties were investigated through the permeability tests, such as with water absorption, rapid chloride permeability, sorptivity, and mercury intrusion porosimetry (MIP) tests, and crack pattern observations and porosity measurements by image analysis on micrographs taken through scanning electron microscope (SEM). The effect of the source concrete quality, water cement ratio, replacement ratio, exposure temperature level and age were examined. The effect of source concrete quality was not found to be significant, while the most significant factor affecting the mechanical and durability properties was found as water/cement ratio. Recycled aggregate content influenced the durability properties much more than the mechanical properties. In general the age factor has a positive effect on measured properties, whereas the elevations in temperatures ranging from room temperature to 250 °C indicated different patterns. The effect of elevated temperatures on the durability properties of concrete was more pronounced for concrete mixtures produced with recycled aggregates than normal concretes. Ten percent replacement of natural aggregate by recycled aggregate assured to retain mechanical and durability properties of concretes after elevated temperatures.

ÖZET

GERİ KAZANILMIŞ AGREGA İLE ÜRETİLEN BETONLARIN YÜKSEK SICAKLIKTAKİ ÖZELLİKLERİ

Yıkılan yapılardan elde edilecek agregaların beton üretiminde yeniden kullanılması çevre kirliliğinin önlenmesine ve doğal kaynakların korunmasına yardımcı olacaktır. Öte yandan bu geri kazanılmış agregalar ile üretilen betonların taze ve sertleşmiş beton özellikleri garanti edilmelidir. Bu çalışma geri kazanılmış agrega ile üretilen betonların mekanik ve dayanıklılık özelliklerine yüksek sıcaklık etkisini sunmaktadır. Beton ve diğer yapı malzemeleri bazen yüksek sıcaklığa maruz kalmaktadır. Geri kazanılmış agrega miktarı yüzde 25, 50, 75 ve 100 olan beton karışımları hazırlanmıştır. Deney numuneleri 250 °C'ye kadar yüksek sıcaklıklara maruz bırakıldıktan sonra mekanik özellikleri basınç dayanımı, elastisite modülü ve yarma çekme dayanımı testleri ile saptanmış olup dayanıklılık özellikleri ise su emme testi, hızlı klor geçirgenlik testi ve kılcal su emme testi gibi geçirimsizlik testleri ve cıva sürüklemeli boşluk testi ile, taramalı elektron mikroskobu ile çekilmiş fotoğraflar üzerinde imaj analizi ile boşluk ölçümü ve de çatlak gözlemleri yapılarak araştırılmıştır. Kaynak betonun kalitesi, su/çimento oranı, geri kazanılmış agrega oranı, sıcaklık ve yaş etkileri incelenmiştir. Mekanik ve dayanıklılık özelliklerini etkileyen ana etkenin su/çimento oranı olduğu ve kaynak beton kalitesinin önemli bir etkisinin bulunmadığı ortaya çıkmıştır. Geri kazanılmış agrega miktarı, dayanıklılık özelliklerini mekanik özelliklerden daha fazla etkilemiştir. Ölçülen özelliklerde yaş faktörü genel olarak olumlu katkı sağlarken, oda sıcaklığı ile 250 °C arasındaki sıcaklık artışı farklı sonuçlar göstermiştir. Yüksek sıcaklıkların özellikle betonun dayanıklılık özelliğine etkisi normal betonlara göre geri kazanılmış agrega ile üretilen betonlarda çok daha fazladır. Doğal agreganın yüzde on oranında geri kazanılmış agrega ile değiştirilmesi betonun mekanik ve dayanıklılık özelliklerini yüksek sıcaklıklardan sonra da muhafaza etmiştir.

TABLE OF CONTENTS

APPROVAL	ii
DEDICATION	iii
ACKNOWLEDGEMENTS	iv
ABSTRACT.....	v
ÖZET.....	vi
LIST OF FIGURES.....	xi
LIST OF TABLES.....	xxv
LIST OF SYMBOLS / ABBREVIATIONS.....	xxx
1. INTRODUCTION.....	1
2. SCOPE.....	3
3. LITERATURE REVIEW.....	5
3.1. Background of Recycled Aggregate and Recycled Aggregate Concrete.....	5
3.2. Properties of Recycled Aggregates.....	7
3.3. Fresh Concrete Properties of Recycled Aggregate Concretes.....	11
3.4. Mechanical Properties of Recycled Aggregate Concretes.....	15
3.5. Durability Properties of Recycled Aggregate Concretes.....	22
3.6. Effects of Elevated Temperature on Concrete	30
4. EXPERIMENTAL STUDY AND PROCEDURE	41
4.1. Materials.....	41
4.1.1. Cement.....	42
4.1.2. Aggregates.....	43
4.1.3. Admixture (Superplasticizer).....	43
4.1.4. Concrete Coloring Agent.....	44
4.2. Production of Source Concretes.....	45
4.3. Casting and Curing of Source Concretes.....	47
4.4. Production of Recycled Aggregates.....	48
4.5. Properties of Recycled Aggregates.....	51
4.6. Tests and Measurements for Aggregates.....	51
4.6.1. Specific Gravity and Absorption Tests of Coarse Aggregates	51

4.6.2. Specific Gravity and Absorption Tests of Fine Aggregates.	53
4.6.3. Total Evaporable Moisture Content of Aggregate by Drying.	54
4.6.4. Sieve Analysis of Fine and Coarse Aggregates	55
4.6.5. Adhered Mortar Content of Recycled Coarse Aggregates	55
4.7. Design of Experiments.	58
4.8. Production of Normal and Recycled Aggregate Concretes.	62
4.9. Casting and Curing of Normal and Recycled Aggregate Concretes.	66
4.10. Tests and Measurements for Fresh Concrete	67
4.10.1. Density (Unit Weight) and Air Content of Concrete.	67
4.11. Tests for Determination of Mechanical Properties of Hardened Concrete . .	68
4.11.1. Compressive Strength and Static Modulus of Elasticity.	70
4.11.2. Splitting Tensile Strength	71
4.12. Tests for Determination of Durability Performance of Hardened Concrete .	71
4.12.1. Density, Absorption and Voids in Hardened Concrete.	72
4.12.2. Measurement of Rate of Absorption of Water in Hydraulic Cement Concrete.	73
4.12.3. Chloride Ion Permeability.	76
4.13. Microstructural Studies of Hardened Concrete	77
4.13.1. Determination of Porosity by Mercury Intrusion Porosimetry.	79
4.13.2. Study of the Interfacial Transition Zone (ITZ) Using Optic Microscope and Scanning Electron Microscope.	81
4.13.3. Determination of Porosity by Image Analysis	85
5. TEST RESULTS AND EVALUATION.	86
5.1. Mechanical Properties of Source Concretes	87
5.2. Fresh Concrete Properties of Natural and Recycled Aggregate Concretes. . .	88
5.3. Mechanical Properties of Natural and Recycled Aggregate Concretes.	89
5.3.1. Compressive Strength.	89
5.3.2. Static Modulus of Elasticity.	110
5.3.3. Splitting Tensile Strength.	127
5.4. Durability Properties of Natural and Recycled Aggregate Concretes.	143
5.4.1. Water Absorption	143
5.4.2. Rapid Chloride Permeability Test.	158
5.4.3. Water Absorption (Sorptivity).	173

5.5. Adhered Mortar Content of Recycled Aggregates.	186
5.6. Optimization.	186
5.7. Relationships between Concrete Properties.	194
5.7.1. Relationship between Compressive Strength and Splitting Tensile Strength.	194
5.7.2. Relationship between Compressive Strength and Static Modulus of Elasticity.	197
5.7.3. Relationship between Compressive Strength and Saturated Bulk Density.	200
5.7.4. Relationship between Compressive Strength and Permeable Pore Volume.	202
5.7.5. Relationship between Compressive Strength and Total Charge Passed.	203
5.7.6. Relationship between Total Charge Passed and Water Absorption. . .	205
5.7.7. Relationship between Total Charge Passed and Sorptivity.	206
5.7.8. Relationship between Sorptivity and Water Absorption.	209
5.7.9. Relationship between Total Charge Passed and Initial Current	210
5.8. Microstructural Studies and Image Analysis on Hardened Concrete.	213
5.8.1. Determination of Pore Structure by Mercury Intrusion Porosimetry (MIP).	213
5.8.2. Examination of Interfaces Using Optical Microscope (OM) and Scanning Electron Microscope (SEM).	225
5.9. Evaluation of the Objective of the Study.	244
6. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORKS.	247
6.1. Conclusions.	247
6.2. Recommendations for Future Works.	250
APPENDIX A: RECYCLED AGGREGATE PRODUCTION.	253
APPENDIX B: DETERMINATION OF ADHERED MORTAR CONTENT S. . . .	254
APPENDIX C: D OPTIMAL ALGORITHM.	256
APPENDIX D: PRODUCTION OF CONCRETES.	257
APPENDIX E: TESTS FOR DETERMINATION OF MECHANICAL PROPERTIES OF HARDENED CONCRETE.	258

APPENDIX F: TESTS FOR DETERMINATION OF DURABILITY

PERFORMANCE OF HARDENED CONCRETE.	259
APPENDIX G: COMPRESSIVE STRESS-STRAIN CURVES.	260
REFERENCES.	261

LIST OF FIGURES

Figure 3.1.	Weight percent of cement paste adhering to original aggregate particles in recycled aggregate produced from original concretes made with different water-cement ratios	10
Figure 3.2.	Relation between flow table value, aggregate core moisture and time .	12
Figure 3.3.	Changes of slump of concrete mixes	13
Figure 3.4.	Variation of fresh concrete unit weights with waste concrete aggregate (WCA) content	14
Figure 3.5.	Start of ceiling strength at a) 28 days and b) 7, 60 and 90 days of RCA and NA concretes	18
Figure 3.6.	The constituents of a concrete incorporating recycled coarse aggregate	21
Figure 3.7.	Effects of recycled aggregate content on durability properties	29
Figure 3.8.	Physicochemical changes in portland cement concrete during heating	32
Figure 3.9.	Forces acting in heated concrete	33
Figure 3.10.	Residual compressive strength of HSCs	35
Figure 3.11.	Residual compressive strength of NSCs	36
Figure 3.12.	Resistance of HSC against chloride ion penetration	36

Figure 3.13.	Resistance of NSC against chloride ion penetration	37
Figure 4.1.	Grading curve of laboratory produced source concrete	47
Figure 4.2.	Flow of producing recycled aggregates	50
Figure 4.3.	Recycled aggregate and its composition	56
Figure 4.4.	Grading curves of concrete mixtures produced with natural aggregates	64
Figure 4.5.	Temperature-time curve of concrete specimens exposed to 105 °C . . .	69
Figure 4.6.	Temperature-time curve of concrete specimens exposed to 175 °C . . .	69
Figure 4.7.	Temperature-time curve of concrete specimens exposed to 250 °C . . .	70
Figure 4.8.	A cylindrical concrete specimen after cutting	72
Figure 4.9.	Absorption (I) versus time ^{1/2}	75
Figure 4.10.	Capillary absorption test setup	75
Figure 4.11.	Rapid chloride permeability test setup	77
Figure 4.12.	Classification of pore size	78
Figure 4.13.	Quantachrome Poremaster equipment for MIP	81
Figure 4.14.	Fracture surface of concrete specimen	82
Figure 4.15.	Interfaces in recycled aggregate concrete (sample from mix 6)	83

Figure 4.16.	Optical microscope setup	83
Figure 4.17.	Field emission scanning electron microscope setup	84
Figure 5.1.	Compressive strength of first stage natural and recycled aggregate concretes	91
Figure 5.2.	Predicted versus actual values of compressive strengths (MPa)	100
Figure 5.3.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)	101
Figure 5.4.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)	101
Figure 5.5.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)	102
Figure 5.6.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)	102
Figure 5.7.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)	103
Figure 5.8.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)	103
Figure 5.9.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)	104
Figure 5.10.	3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)	104

Figure 5.11.	Interaction graph of w/c and replacement ratio at 28 days	105
Figure 5.12.	Interaction graph of w/c and replacement ratio at 90 days	106
Figure 5.13.	Interaction graph of w/c and temperature	107
Figure 5.14.	Interaction graph of replacement ratio and temperature	108
Figure 5.15.	Interaction graph of temperature and age	109
Figure 5.16.	Predicted versus actual values of static modulus of elasticity (GPa) . .	117
Figure 5.17.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)	118
Figure 5.18.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)	118
Figure 5.19.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)	119
Figure 5.20.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)	119
Figure 5.21.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)	120
Figure 5.22.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)	120
Figure 5.23.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)	121

Figure 5.24.	3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)	121
Figure 5.25.	Interaction graph of w/c and replacement ratio at 28 days	122
Figure 5.26.	Interaction graph of w/c and replacement ratio at 90 days	123
Figure 5.27.	Interaction graph of w/c and temperature	124
Figure 5.28.	Interaction graph of replacement ratio and temperature	125
Figure 5.29.	Interaction graph of temperature and age	126
Figure 5.30.	Predicted versus actual values of splitting tensile strength (MPa)	133
Figure 5.31.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)	134
Figure 5.32.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)	134
Figure 5.33.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)	135
Figure 5.34.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)	135
Figure 5.35.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)	136
Figure 5.36.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)	136

Figure 5.37.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)	137
Figure 5.38.	3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)	137
Figure 5.39.	Interaction graph of w/c and replacement ratio at 28 days	138
Figure 5.40.	Interaction graph of w/c and replacement ratio at 90 days	139
Figure 5.41.	Interaction graph of w/c and temperature	140
Figure 5.42.	Interaction graph of replacement ratio and temperature	141
Figure 5.43.	Interaction graph of temperature and age	142
Figure 5.44.	Predicted versus actual values of water absorption (%)	148
Figure 5.45.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)	149
Figure 5.46.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)	149
Figure 5.47.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)	150
Figure 5.48.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)	150
Figure 5.49.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)	151

Figure 5.50.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)	151
Figure 5.51.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)	152
Figure 5.52.	3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)	152
Figure 5.53.	Interaction graph of w/c and replacement ratio at 28 days	153
Figure 5.54.	Interaction graph of w/c and replacement ratio at 90 days	154
Figure 5.55.	Interaction graph of w/c and temperature	155
Figure 5.56.	Interaction graph of replacement ratio and temperature	156
Figure 5.57.	Interaction graph of temperature and age	157
Figure 5.58.	Predicted versus actual values of RCPT charge (coulombs)	163
Figure 5.59.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)	164
Figure 5.60.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)	164
	. . .	
Figure 5.61.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)	165
Figure 5.62.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)	165

Figure 5.63.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)	166
Figure 5.64.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)	166
Figure 5.65.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)	167
Figure 5.66.	3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)	167
Figure 5.67.	Interaction graph of w/c and replacement ratio at 28 days	168
Figure 5.68.	Interaction graph of w/c and replacement ratio at 90 days	169
Figure 5.69.	Interaction graph of w/c and temperature	170
Figure 5.70.	Interaction graph of replacement ratio and temperature	171
Figure 5.71.	Interaction graph of temperature and age	172
Figure 5.72.	Predicted versus actual values of log10 sorptivity (mm/sqrt (sec)) . . .	177
Figure 5.73.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)	178
Figure 5.74.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)	178
Figure 5.75.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)	179

Figure 5.76.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)	179
Figure 5.77.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)	180
Figure 5.78.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)	180
Figure 5.79.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)	181
Figure 5.80.	3D surface view of sorptivity of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)	181
Figure 5.81.	Interaction graph of w/c and replacement ratio at 28 days	182
Figure 5.82.	Interaction graph of w/c and replacement ratio at 90 days	183
Figure 5.83.	Interaction graph of temperature and age, w/c ratio =0.35 (replacement ratio =0%)	184
Figure 5.84.	Interaction graph of temperature and age, w/c ratio =0.35 (replacement ratio =100%)	184
Figure 5.85.	Interaction graph of temperature and age, w/c ratio =0.60 (replacement ratio =0%)	185
Figure 5.86.	Interaction graph of temperature and age, w/c ratio =0.60 (replacement ratio =100%)	185

Figure 5.87.	Response surface plot of the composite desirability (D) at 22 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously	190
Figure 5.88.	Response surface plot of the composite desirability (D) at 105 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously	190
Figure 5.89.	Response surface plot of the composite desirability (D) at 175 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously	191
Figure 5.90.	Response surface plot of the composite desirability (D) at 250 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously	191
Figure 5.91.	Response surface plot of the composite desirability (D) at 22 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously	192
Figure 5.92.	Response surface plot of the composite desirability (D) at 105 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously	192
Figure 5.93.	Response surface plot of the composite desirability (D) at 175 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously	193
Figure 5.94.	Response surface plot of the composite desirability (D) at 250 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously	193

Figure 5.95.	Relationship between split tensile strength and compressive strength .	195
Figure 5.96.	Relationship between static modulus of elasticity and compressive strength	199
Figure 5.97.	Relationship between compressive strength and saturated bulk density	200
Figure 5.98.	Relationship between compressive strength and volume of permeable pore space	202
Figure 5.99.	Relationship between total charge passed and compressive strength . .	204
Figure 5.100.	Relationship between total charge passed and water absorption	206
Figure 5.101.	Relationship between total charge passed and sorptivity	208
Figure 5.102.	Relationship between sorptivity and water absorption	209
Figure 5.103.	Relationship between total charge passed and initial current	211
Figure 5.104.	Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 1-6	217
Figure 5.105.	Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 7-11	218
Figure 5.106.	Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 12-18	218
Figure 5.107.	Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 19-24	219

Figure 5.108. Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 25-31	219
Figure 5.109. 3D surface view of mean distribution pore diameter (age: 28 days, temperature: 22 °C)	221
Figure 5.110. 3D surface view of mean distribution pore diameter (age: 28 days, temperature: 105 °C)	222
Figure 5.111. 3D surface view of mean distribution pore diameter (age: 28 days, temperature: 175 °C)	222
Figure 5.112. 3D surface view of mean distribution pore diameter (age: 28 days, temperature: 250 °C)	223
Figure 5.113. 3D surface view of mean distribution pore diameter (age: 90 days, temperature: 22 °C)	223
Figure 5.114. 3D surface view of mean distribution pore diameter (age: 90 days, temperature: 105 °C)	224
Figure 5.115. 3D surface view of mean distribution pore diameter (age: 90 days, temperature: 175 °C)	224
Figure 5.116. 3D surface view of mean distribution pore diameter (age: 90 days, temperature: 250 °C)	225
Figure 5.117. Sectional view of interface #1 (interface between natural (gray) aggregate (GA) and new (red) mortar (RM) of mix 6)	226
Figure 5.118. Sectional view of interface #1 (interface between natural (gray) aggregate (GA) and new (red) mortar (RM) of mix 6)	227

Figure 5.119. Sectional view of interface #2 (interface between adhered (gray) mortar (GM) and original coarse (white) aggregate (WA) of mix 6) . .	227
Figure 5.120. Image analysis procedure	234
Figure 5.121. Examples of thresholding	235
Figure 5.122. Interfaces in recycled aggregate concrete (replacement ratio <100 %).	235
Figure 5.123. 3D surface view of mortar porosity (age: 28 days, temperature: 22 °C)	240
Figure 5.124. 3D surface view of mortar porosity (age: 28 days, temperature: 105 °C)	241
Figure 5.125. 3D surface view of mortar porosity (age: 28 days, temperature: 175 °C)	241
Figure 5.126. 3D surface view of mortar porosity (age: 28 days, temperature: 250 °C)	242
Figure 5.127. 3D surface view of mortar porosity (age: 90 days, temperature: 22 °C)	242
Figure 5.128. 3D surface view of mortar porosity (age: 90 days, temperature: 105 °C)	243
Figure 5.129. 3D surface view of mortar porosity (age: 90 days, temperature: 175 °C)	243
Figure 5.130. 3D surface view of mortar porosity (age: 90 days, temperature: 250 °C)	243
Figure 6.1. In situ compressive test machine	252

Figure A.1.	Recycled aggregate production stages	253
Figure B.1.	Hydrochloric dissolution method.	254
Figure B.2.	Heating method.	255
Figure D.1.	Production of concretes.	257
Figure E.1.	Tests for determination of mechanical properties of hardened concrete.	258
Figure F.1.	Test for determination of chloride permeability of hardened concrete.	259
Figure G.1.	Compressive stress-strain curve of concrete specimens of mix 1 (w/c: 0.35, RR: 0%, T:22 °C, Age: 90 days)	260
Figure G.2.	Compressive stress-strain curve of concrete specimens of mix 2 (w/c: 0.35, RR: 0%, T:105 °C, Age: 28 days)	260

LIST OF TABLES

Table 3.1.	Recycled aggregate applications in different countries.	6
Table 3.2.	Composition of categories for recycled aggregates in concrete and mortar	9
Table 4.1.	Chemical composition, physical properties & strength characteristics of cement	42
Table 4.2.	Sieve analysis and physical properties of the natural aggregates.	44
Table 4.3.	Physical and chemical properties of superplasticizer.	44
Table 4.4.	Mixture proportions of the plant source concretes (kg/m^3)	46
Table 4.5.	Mixture proportion of laboratory produced source concretes (in kg/m^3)	46
Table 4.6.	Percentage of recycled aggregates by weight (first stage)	49
Table 4.7.	Percentage of recycled fine and coarse aggregates by weight (second stage)	49
Table 4.8.	Sieve analysis of the recycled aggregates (first stage)	51
Table 4.9.	Sieve analysis and physical properties of the recycled coarse aggregates (second stage)	53
Table 4.10.	The design layout of the experiments of the study.	61

Table 4.11.	Mixture proportions of concretes of first stage (in kg/m ³)	62
Table 4.12.	Mixture proportions of concretes of second stage.	65
Table 4.13.	Fresh concrete properties of second stage.	66
Table 4.14.	Chloride ion penetrability based on charge passed.	77
Table 5.1.	Compressive strength of first stage source concretes at 28 days.	87
Table 5.2.	Compressive strength of second stage source concretes at 28 days. . . .	88
Table 5.3.	Splitting tensile strength of second stage source concretes at 28 days. .	88
Table 5.4.	Compressive strength of first stage natural and recycled aggregate concretes at 28 days.	90
Table 5.5.	Compressive strength of first stage natural and recycled aggregate concretes at 90 days.	90
Table 5.6.	Compressive strength of second stage natural & recycled aggregate concretes.	92
Table 5.7.	Coding of w/c ratio factor.	98
Table 5.8.	Coding of replacement ratio factor.	98
Table 5.9.	Coding of temperature factor.	98
Table 5.10.	Coding of age factor.	98
Table 5.11.	Actual and predicted values of compressive strength.	99

Table 5.12.	Results of ANOVA for compressive strength response surface 2FI model.	100
Table 5.13.	Static modulus of elasticity values of second stage natural & recycled aggregate concretes.	111
Table 5.14.	Actual and predicted values of static modulus of elasticity.	112
Table 5.15.	Results of ANOVA for static modulus of elasticity response surface 2FI model.	115
Table 5.16.	Splitting tensile strength values of second stage natural & recycled aggregate concretes.	128
Table 5.17.	Actual and predicted values of splitting tensile strength.	129
Table 5.18.	Results of ANOVA for splitting tensile strength response surface 2FI model.	133
Table 5.19.	Water absorption of first stage concretes at 28 days.	143
Table 5.20.	Voids in hardened concrete, water absorption, and density values of second stage natural & recycled aggregate concretes.	144
Table 5.21.	Actual and predicted values of water absorption.	146
Table 5.22.	Results of ANOVA for water absorption response surface 2FI model. .	148
Table 5.23.	RCPT charge (coulombs) of second stage natural & recycled aggregate concretes.	159
Table 5.24.	Actual and predicted values of RCPT charge.	160

Table 5.25.	Results of ANOVA for RCPT charge response surface 2FI model. . . .	163
Table 5.26.	Initial and secondary water absorption (sorptivity) of second stage natural and recycled aggregate concretes.	174
Table 5.27.	Actual and predicted values of sorptivity.	175
Table 5.28.	Results of ANOVA for sorptivity response surface linear model.	177
Table 5.29.	The optimal values of design variables and predicted response values at 28 days.	189
Table 5.30.	The optimal values of design variables and predicted response values at 28 days.	189
Table 5.31.	Relationship between static modulus of elasticity and compressive strength at specific situations.	198
Table 5.32.	Relationship between compressive strength and saturated bulk density at specific situations.	201
Table 5.33.	Relationship between compressive strength and volume of permeable pore space at specific situations.	203
Table 5.34.	Relationship between total charged passed and compressive strength at specific situations.	205
Table 5.35.	Relationship between total charge passed and water absorption at specific situations.	207
Table 5.36.	Relationship between total charge passed and sorptivity at specific situations.	208

Table 5.37.	Relationship between sorptivity and water absorption at specific situations.	210
Table 5.38.	Relationship between total charge passed and initial current at specific situations.	212
Table 5.39.	Analysis results of MIP tests.	215
Table 5.40.	Results of ANOVA for maximum pore diameter surface 2FI model. . .	220
Table 5.41.	Results of ANOVA for threshold pore diameter surface linear model. .	220
Table 5.42.	Results of ANOVA for mean pore diameter surface linear model. . . .	220
Table 5.43.	Results of ANOVA for mean distribution pore diameter surface 2FI model.	221
Table 5.44.	Evaluation of interface micrographs.	228
Table 5.45.	Summary of interface comparison.	233
Table 5.46.	Effect of temperature on crack formation.	233
Table 5.47.	Porosity of the mortars at interfaces.	236
Table 5.48.	Porosity of the mortars of mixes 1-31.	239
Table 5.49.	Results of ANOVA for mortar porosity response surface 2FI model. . .	240
Table 5.50.	Comparison of natural aggregate concrete (NAC) and recycled aggregate concrete (RAC) properties at varying situations (Replacement ratio = 10%).	246

LIST OF SYMBOLS / ABBREVIATIONS

a	regression coefficient
A	Air content (percentage of voids) in concrete
B	Mass of measure filled with water to calibration mark
c_1	Coefficient
C	Mass of measure filled with sample and water
CV	Coefficient of variation
d	Mean pore size
d_{ra}	Density of recycled aggregate
d_a	Density of original coarse aggregate
d_m	Density of adhered mortar
d_j	Desirability function
D	Density of concrete
E_c	Static modulus of elasticity
f_c	Compressive strength
f_{st}	Split tensile strength
I	Intercept
k	Number of factors
kg	Kilogram
kN	Kilo Newton
L	Length of the specimen
$\min f_j$	Minimum actual (experimental) values of j th response
$\max f_j$	Maximum actual (experimental) values of j th response
m_{ra}	Mass of recycled aggregate
m_a	Mass of original coarse aggregate
m_m	Mass of adhered mortar
M	Total mass of all materials batched
M_c	Mass of measure filled with concrete
M_m	Mass of measure
N	Newton

p	Total evaporable moisture content of sample
P	Maximum applied load
$P_{\text{interface1}}$	Computed porosity values at interface 1
Q	Total charge passed (coulombs)
r_a	Original coarse aggregate content, mass fraction
r_m	Adhered mortar content, mass fraction
R	Response
S	Sorptivity (mm/sqrt(sec))
T	Temperature
V	Total absolute volume of the component ingredients in the batch
V_a	volume of original coarse aggregate
V_m	Volume of adhered mortar
W	Mass of original sample
w/c	Water/cement ratio
x	factors
X'	transpose of matrix X
Y	Response function
Y_j	Fitted value
β	Coefficient
ε	Error
v	Volume of permeable pore space (%)
ρ	Saturated bulk density (g/cm ³)
σ	Standard deviation
ASTM	American Society for Testing and Materials
ACI	American Concrete Institute
IC	Initial current (ampere, A)
MIP	Mercury Intrusion Porosimetry
OM	Optic Microscope

RILEM	The International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM, from the name in French)
RCPT	Rapid chloride permeability test
RR	Replacement ratio
SEM	Scanning Electron Microscope
TS	Turkish Standard

1. INTRODUCTION

Utilization of the waste materials has gained importance for many years because of economical and environmental aspects. Due to the reservation of natural resource, prevention of environmental pollution, and economic consideration the waste materials, for example fly ash and silica fume [1], waste tire [2-4], waste glass [5-6], and waste plastic [7-9], have been used for producing concrete. In this respect, the handling of demolished concrete as a solid waste can not be excluded. Like other waste materials, waste concrete aggregate is being increasingly used in concrete production [10]. The use of construction waste as a source of aggregate for the production of new concrete has become more common in the recent decade. The increasing charges for landfill, on the one hand, and the scarcity of natural resources for aggregate, on the other hand, encourage the use of waste from construction sites as a source for aggregates [11]. Recycled concrete aggregate, sometimes referred to as crushed concrete, comes from the demolition of Portland cement concrete elements of buildings, roads, and other infrastructures [12], and sometimes even from collapsed structures due to wars and earthquakes. Reuse of aggregates from demolished concrete structures was introduced into practice many years ago, and from the beginning it has been considered in two main environmental aspects: solving the increasing waste storage problem and protection of limited natural sources of aggregates. At present, plenty of demolished reinforced concrete or pre-stressed concrete structures, originally erected from concrete of moderate or high strength, create the significant source of rubble aggregate of relatively good quality [13]. Rubble obtained from demolition of concrete road pavement, runways, foundations and building structures has been reused successfully around the world. Successful application of recycled aggregate in construction projects has been reported in some European and American countries [14].

Recycling of waste concrete and reusing of it is needed because it is the only way to decrease the waste. The tendency to the use of recycled aggregates is the solution to the problem of an excess of waste concrete, by not forgetting the parallel trend of improvement of final product quality. Concrete technology must consider the need for preserving natural resources from exhaustion and find proper ways to reuse waste concrete.

For this purpose, many researchers have studied the properties of concretes made with recycled aggregates. Many tests have been carried out to determine the properties of recycled aggregates obtained from waste concrete. These tests are carried out with the purpose of evaluating the appropriateness of waste concrete material as an aggregate as set by ASTM C 33, standard specification for concrete aggregates. The mechanical and durability properties of concretes made with recycled aggregates are different than those made with natural aggregates.

The typical applications of recycled aggregates are as follows: general bulk fill, base or fill in drainage projects, subbase or surface material in pavement and road construction, and new concrete manufacture. Until now, there was not any study published dealing with recycled aggregate concretes that are exposed to elevated temperatures, although sometimes, construction materials are submitted to high level of temperatures (reactor vessels, thermal shock, fire, coal gasification vessels, pressure furnaces, chimneys, etc.). The objective of this thesis is to analyze behaviour of recycled aggregate concrete at elevated temperatures.

2. SCOPE

This research work in the form of doctoral thesis covered the recycled aggregate concrete at elevated temperatures. The main aim of the study was to examine the properties of concrete, namely mechanical and durability properties, incorporating natural and recycled aggregate, at room and elevated temperatures. The aim was achieved by the following performance evaluation tests in macro level, and measurements and analysis in micro level:

1. Macro level:

Mechanical properties

- Compressive strength test
- Static modulus of elasticity test
- Splitting tensile test

Durability properties (Permeability tests)

- Rapid chloride permeability test
- Water absorption test
- Capillary absorption test (Sorptivity test)

2. Micro level:

- Porosity measurement by mercury intrusion porosimetry
- Optic microscope and scanning electron microscope (SEM) analysis
- Image analysis

The below factors were investigated within the scope of the study in order to see their effects on concrete properties:

- Quality of original (source) concretes to be recycled
- Water/cement ratio (w/c) of natural and recycled aggregate concrete
- Replacement ratio, the ratio of natural coarse aggregates replaced by recycled coarse aggregates by weight
- Exposure temperature of natural and recycled aggregate concretes
- Age of natural and recycled aggregate concretes

The first stage of this experimental work was consisting of investigating the effect of quality of original (source) concrete on recycled aggregate concrete. In order to do this three different source concretes with low, medium and high strengths were crushed and thus recycled in order to see the basic characteristics of recycled aggregate, and recycled aggregate concrete such as compressive strength and water absorption.

The second stage of the experimental work was performed in order to study the effect of water/cement ratio, recycled aggregate replacement ratio, exposed temperature and age on natural and recycled aggregate concrete properties in macro and micro scale. For the second stage, a computer program was used to design the experiments with above mentioned factors. According to this software a number of 31 mixtures were produced with varying water cement and recycled aggregate replacement ratios, and then exposed to different temperatures and tested at specific ages. The details of the experimental study, test procedures and the measured properties of the materials used for concrete production are presented in chapter 4. Chapter 5 contains the test results of the study and evaluation of these test results. In chapter 6, the conclusions of the investigation and recommendations for future work are listed. In addition to the scope of the study chapter “chapter 2”, and introductory chapter “chapter 1”, background information about recycled aggregate concrete and effect of elevated temperature on concrete are provided in chapter 3.

3. LITERATURE REVIEW

3.1. Background of Recycled Aggregate and Recycled Aggregate Concrete

Recycling of demolished construction materials and reuse of these recycled particles gained importance especially after the Second World War. Many researchers have investigated the properties of recycled aggregates and the minimum requirements for their utilization in concrete and properties of concretes made with these recycled aggregates since mid 1970s. The state of knowledge on the use of recycled concrete as an aggregate in new concrete was reviewed in 1978 by Nixon [15]. The extensive but fragmented researches on recycled concrete aggregates and recycled aggregate concrete, which have been carried out in various parts of the world from 1945 to 1985, have been collated to form a comprehensive state-of-the-art document by Hansen [16].

RILEM Committee 121-DRG in year 1994 has published recommendations [17] for the use of recycled aggregates by classifying them into three groups:

- Group I: aggregates which are implicitly understood to originate from masonry rubble;
- Group II: aggregates which are implicitly understood to originate primarily from concrete rubble; and
- Group III: aggregates which are implicitly understood to consist of a blend of recycled aggregates and natural aggregates, a mixture of natural aggregates (>80%) and rubble from the other two groups (with up to 10% of Type I).

Group III can be used for the production of all types of concrete, whereas restrictions, listed by the committee, limit the applications of the other two groups.

Roos [18] reported the applications of the recycled aggregates in different countries, which are summarized in Table 3.1.

Table 3.1. Recycled aggregate applications in different countries [18]

Country	Applications	Allowable recycled materials, volume %	Using of recycled sand < 4 mm	Allowable concrete class	Allowable amount of other materials, mass %
Belgium	Only in not aggressive environment	0-100	Only when achieving the specifications of natural aggregates	According to aggregates up to strength C30	< 1 non mineral components
Denmark	Only in passive and moderator environment	0-100 for particles > 4 mm	Allowed	According to aggregates up to 21 MPa strength	-
Germany	Not for pre-stressed concrete and strong chemical attack	0-42 only crushed concrete	Max. 7 % < 2 mm	Up to C35	< 0.2 wood, plastic materials
Japan	Only in subordinated elements	0-100 only crushed concrete	Allowed	According to aggregates up to 30 MPa strength	Up to 10 kg/m ³ gypsum & Up to 2 kg/m ³ asphalt
Netherlands	Only in passive and moderator environment	20 only crushed concrete	Up to 20 % allowed	Up to strength 40 MPa	< 1 bitumen & < 0.15 organic materials
Switzerland	For pre-stressed only with special tests	0-100 depending on the use	Allowed	All according to concrete building specification	< 0.3 wood, plastic materials
USA	Concrete, reinforcement and pre-stressed concrete	0-100 only crushed concrete	Allowed	All according to ACI 318-95	No information

3.2. Properties of Recycled Aggregates

Recycled aggregates have several properties which are rather different than those of natural aggregates, for example a coarser and more angular shape as a result of the crushing operations, a varying particle size distribution, also depending on the crusher settings; and a lower density. One of the most typical features of recycled aggregate is the adhered mortar on the surface of the original natural aggregates. This adhered mortar increases the water absorption percentage and decreases the specific gravity, bonding capacity and abrasion resistance [19].

According to the test results studied by Limbachiya *et al.* [20], recycled concrete aggregates had 7 to 9 percent lower relative density and two times higher water absorption than natural aggregates, reflecting the porosity of the hydrated cement paste attached to the recycled concrete aggregate. Lower specific gravities and higher water absorption characteristics of recycle concrete aggregates were also reported by other researchers [21-22]. The strength of original concrete, crushing process [23], and size of the aggregates also have effects on recycled aggregate properties, a recycled aggregate that has been obtained from a high strength concrete will have a higher specific gravity. The specific gravity of the recycled aggregates depends on the recycled aggregate quality. The aggregates with a higher amount of attached mortar will have lower specific gravity. It is seen in many studies that specific gravity of recycled concrete aggregate reduces with smaller sizes of aggregates, because the mortar percentage of finer recycled aggregates is higher than those of the coarser recycled aggregates. Due to the same reason, the water absorption capacity of the smaller sizes recycled aggregates is higher. Recycled aggregates absorb copious amount of water, most likely into the mortar and mortar-aggregate interface components of the recycled aggregate and any fine cracks in the recycled aggregate may contribute to the increase in water absorption. For using recycled aggregates, water absorption will not present any problem if proper batching method is adopted [24]. Due to the high water absorption capacity of recycled aggregates, utilization of presoaked aggregates is suggested in the production of recycled aggregate concrete in order to maintain a uniform quality during concrete production [25].

Another major feature of recycled aggregates is the presence of foreign substances, known as contaminants, in them. These are present in recycled aggregates from the original demolition debris which gets passed on to them. They may include clay balls, bitumen chunks, and gypsum from drywalls, bricks, and organic material like wood, glass, steel pieces and other metals. Contaminants generally cause adverse effects on the mechanical properties and the durability of concrete made with recycled aggregates. Therefore, much work has to be devoted to find ways of reducing the contents of these in the waste concrete, to lessen their effects, and to find uses where these harmful effects are acceptable [19].

The strength characteristics of the recycled coarse aggregates can be tested by Los Angeles abrasion test. The value of Los Angeles abrasion changes depending on the original aggregate quality, original concrete strength, crushing process type, and adhered mortar content.

Topçu and Şengel [26] reported the properties of recycled aggregates in their study. According to their literature review, recycled concrete aggregates are crushed by means of different methods so that they could be used as concrete aggregates. Through visual observations, many experts working in this area have so far agreed that recycled concrete aggregates make proper aggregates. Waste concrete can be crushed into fine and coarse aggregates. In comparison with normal aggregate, recycled concrete aggregates have a higher water absorption ratio but a lower specific gravity. Also, Los Angeles abrasion percentage and crushing values are much higher. It was concluded that recycled concrete aggregates would contain as much as 40 percent of mortar, which would accordingly affect such deformation properties of recycled concrete aggregates as elasticity, creep, and shrinkage.

Recycled aggregates are categorized by DIN (Deutsch Institut für Normung) according to their original material composition, in year 2000. According to DIN 4226-100 (Gesteinskörnungen für Beton und Mörtel, Teil 100: rezyklierte Gesteinskörnungen) standard recycled aggregates are classified into four types, Table 3.2 shows the composition of categories for recycled aggregates in concrete and mortar [27]. According to DIN 4226-100, the minimum particle density of type 1 and 2 must be 2000 kg/m^3

compared to 1800 and 1500 kg/m³ for type 3 and 4, with a limited tolerance of ± 150 kg/m³ for the first three types and with no limits for type 4. According to the same standard, water absorption must not exceed 10, 15 and 20 percent by mass for the first three types.

Table 3.2. Composition of categories for recycled aggregates in concrete and mortar [27]

Properties		Composition [Mass %]			
		Type 1	Type 2	Type 3	Type 4
Concrete, aggregate according to DIN 4226-1		≥ 90	≥ 70	≤ 20	≥ 80
Clinker, non aerated bricks		≤ 10	≤ 30	≥ 80	
Sand lime block				≤ 5	
Other mineral properties ¹		≤ 2	≤ 3	≤ 5	≤ 20
Foreign matter	asphalt	≤ 1	≤ 1	≤ 1	
	Mineral ²	≤ 2	≤ 2	≤ 2	≤ 1
	Non mineral ³	≤ 0,5	≤ 0,5	≤ 0,5	

¹ Other mineral properties are e.g.:

Aerated bricks, light concrete, aerated concrete, plaster, mortar, aerated slag

² Mineral foreign matters are e.g.:

Glass, ceramics, non iron metal slag, plaster

³ Non mineral foreign matters are e.g.:

Rubber, plastics, metal, wood, organics, other materials

Recycled aggregate shows different characteristics due to the adhered mortar content. A recycled aggregate with zero percent adhered mortar will be the same as natural aggregate. The adhered mortar affects the water absorption and unit weight properties, while the adhered mortar was affected by original concrete strength, crushing process and recycled aggregate size. The amount of adhered mortar increases with the decrease of the size as shown in Figure 3.1. Hansen [16] reported that the other differences for the recycled aggregates are particle shape and surface texture. The recycled aggregates in general have more rough surface area compared to the natural aggregates, while the particle shape depends on the processing methods of the demolished materials.

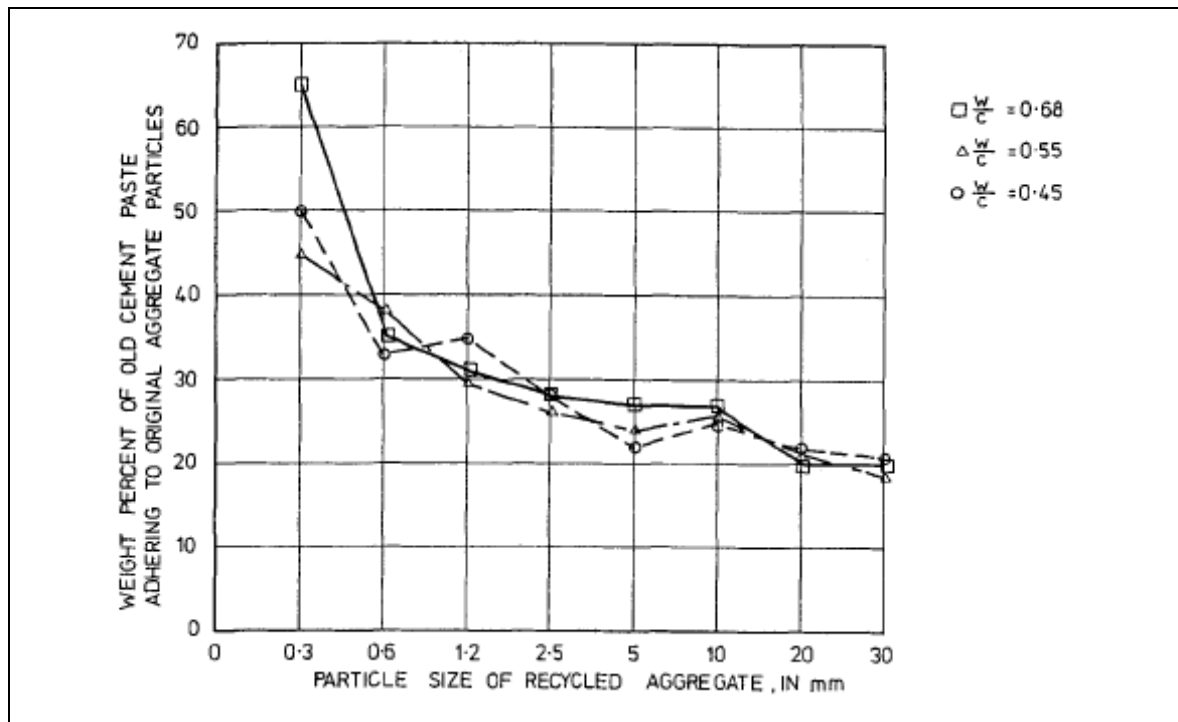


Figure 3.1. Weight percent of cement paste adhering to original aggregate particles in recycled aggregate produced from original concretes made with different water-cement ratios [16]

Chan [19] reported the applicability of recycled aggregates. According to him, the acceptance of recycled aggregates for production of new concrete will depend on whether they satisfy certain specifications. While these may be different from locality to locality, a number of general requirements are identifiable such as: being strong enough for the grade of concrete required; being dimensionally stable; being free of contaminants which could lead to adverse reactions; and having a suitable gradation. Some countries have made considerable use of recycled aggregates to mix new concrete as the conditions in those places have forced them to head in this direction. These include Japan and certain parts of Europe. Through their applications, they have demonstrated that this kind of technology is possible as long as some trade-offs are accepted. The only question is whether or not the economic market and the political environment in that place allow for such technology to be implemented.

3.3. Fresh Concrete Properties of Recycled Aggregate Concretes

The use of recycled aggregates in concrete is not simple as natural aggregates because of their high water absorption capacity which leads to difficulties in controlling the properties of fresh concrete and consequently influences the strength and durability of hardened concrete [28]. Producing the concrete with recycled aggregates needs higher amount of water or superplasticizer to achieve the required workability because of its adhered mortar as well as its rough surface. Hansen [16] reviewed the previous studies for fresh recycled aggregate concrete. According to his literature review, Mukai *et al.* found that concretes produced with coarse recycled aggregate and natural sands required approximately 5 percent more water than control concretes produced with corresponding natural aggregate in order to achieve the same slump. Approximately 15 percent more free water was required when both fine and coarse recycled aggregates were used.

Nealen and Schenk [29] examined the influence of recycled aggregate core moisture on freshly mixed and hardened concrete properties. If recycled aggregates are employed in dry conditions concrete's workability is greatly reduced due to their water absorption. The relation between flow table value, aggregate core moisture (CM) and time is shown in Figure 3.2. The recycled aggregates should be pre-wetted or saturated with water to prevent a rapid decrease in concrete workability [30]. Due to the high water absorption, the used recycled concrete aggregates were presoaked before mixing in other studies [25-26, and 31].

Poon *et al.* [32] reported that the initial slump of a concrete mixture depended on the initial free water content, while the slump loss of the mixture with time was related to the initial moisture state of the aggregates. They also reported that in the case of using recycled aggregates in the saturated surface dried state, the high water content inside the aggregate particles may result in bleeding during casting. Bleeding has negative effects on concrete properties, which is referred to the movement of water to the surface of fresh concrete a result of the settlement of solid particles. It may also increase water/cement ratio of concrete.

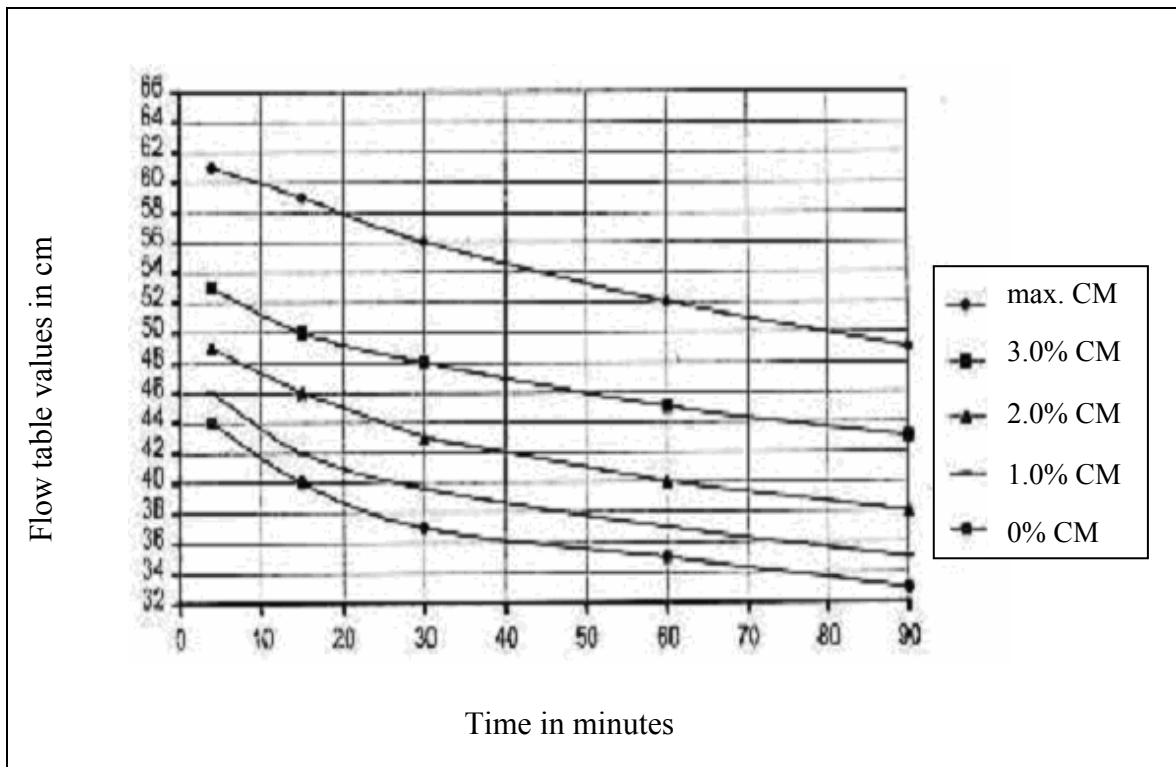


Figure 3.2. Relation between flow table value, aggregate core moisture and time [29]

Poon *et al.* [28] studied the properties of fresh concrete prepared with recycled aggregates at the replacement levels from zero to 100 percent of the total coarse aggregate. The influence of recycled aggregate on the slump and bleeding are investigated. Test results, as illustrated in Figure 3.3, showed that the initial slump increased with an increase in the percentage of recycled aggregate. The mix prepared with 100 percent recycled aggregate showed the greatest slump of 195 mm. This is due to the higher initial free water content in the concrete mixture. This higher initial free water concrete prepared with recycled aggregates was due to the higher water absorption of the recycled aggregate. The recycled aggregate was used at the air dried condition with the initial moisture content of the aggregate at mixing much lower than its absorption capacity. Additional amounts of water were added to maintain the mix proportions (designed at the saturated surface dried state). It is also shown in Figure 3.3 that the rate of slump loss was quicker within the first hour after mixing, but was lower afterward. The mix without recycled aggregate took about 130 minutes to decrease to the zero slumps; while the mix with 100 percent recycled aggregates took over three hours.

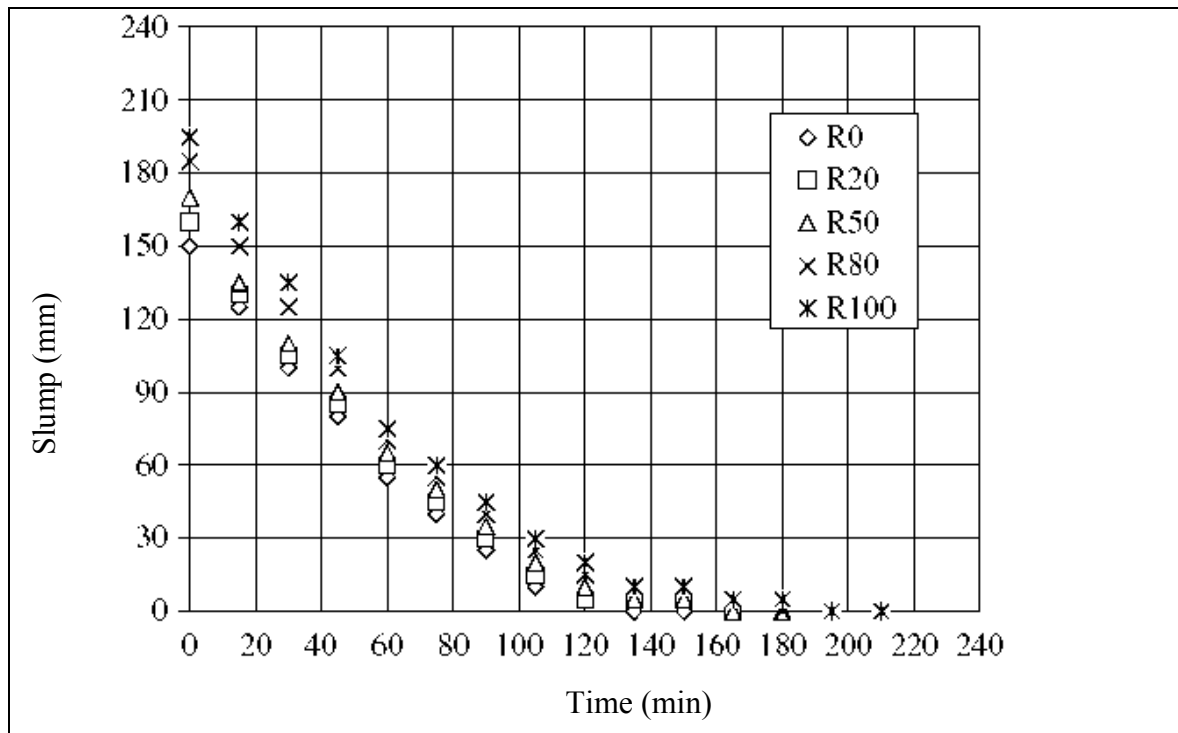


Figure 3.3. Changes of slump of concrete mixes [28]

Topçu and Şengel [26] investigated the properties of concretes produced with waste concrete aggregate. When producing new concretes, in place of natural aggregates, 30, 50, 70 and 100 percent of waste concrete aggregates were added as coarse aggregates. Fresh concrete specimen tests were carried out to determine workability of concrete with waste concrete aggregates. The results obtained showed that workability decreased in parallel to an increase in the proportion of waste concrete aggregates. The decrease amount between natural concrete and concrete with 100 percent of waste concrete aggregates was nearly 15-20 percent; the reason is that the water absorption ratio of mortar over waste concrete aggregates was much higher. The results, as illustrated in Figure 3.4, also showed that fresh unit weight decreased in concrete with waste concrete aggregate. Specific gravity of waste concrete aggregates was far lower than that of normal crushed aggregates. The reason for such a decrease is the adhered mortar with lower specific gravity existing on the surface of original aggregates.

Katz [11] tested the properties of the recycled aggregate and of the new concrete made from it, with nearly 100 percent of aggregate replacement. Fresh concrete tests showed that the bulk density of concrete made with natural aggregates was in the typical

range of normal concrete (approximately 2400 kg/m^3), whereas the concrete made with recycled aggregates was significantly lighter, approximately 2150 kg/m^3 . That lower density is the result of lower bulk specific density of the recycled aggregates. In addition, higher air content was observed, leading to an additional reduction in the density of fresh concrete. The results indicated that a normal air content for the reference concrete, but a higher one (4-5.5 percent) for the new concrete made with recycled aggregate.

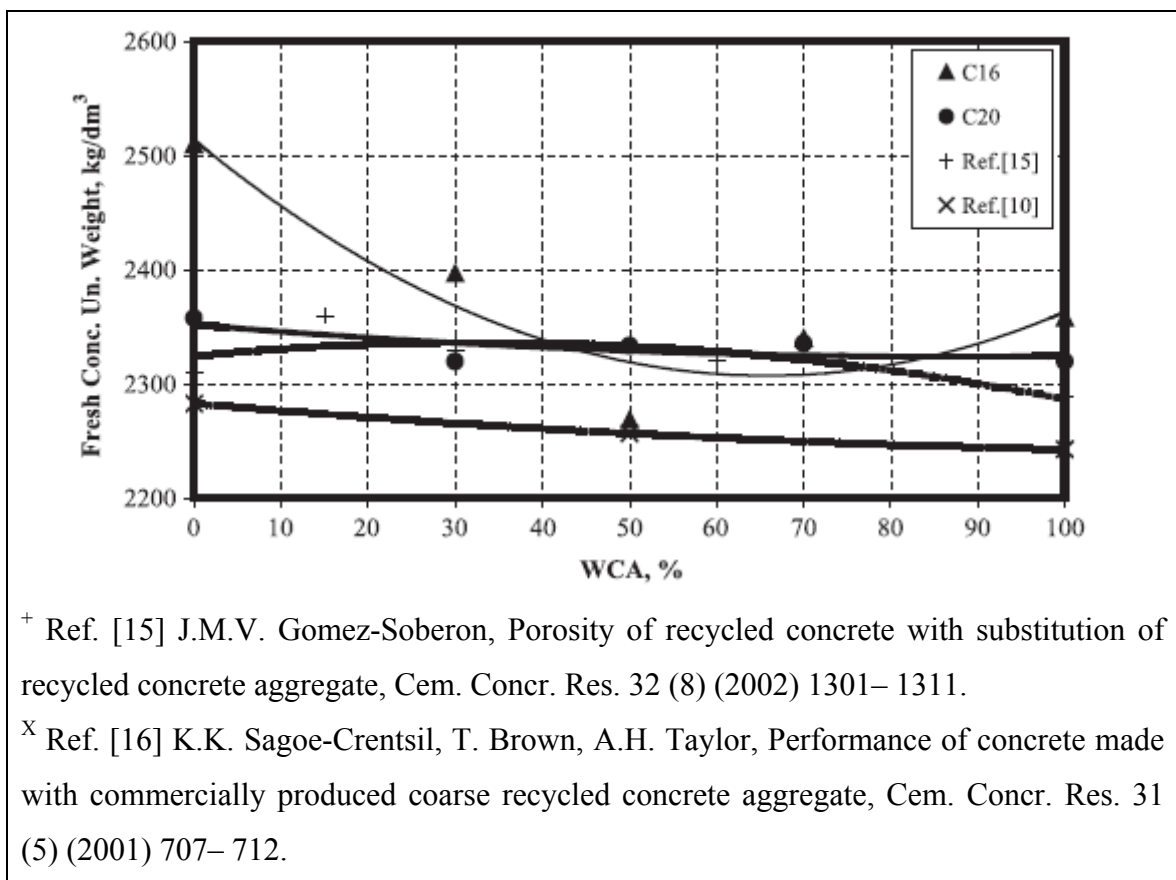


Figure 3.4. Variation of fresh concrete unit weights with waste concrete aggregate (WCA) content [26]

Due to the differences in natural and recycled aggregate properties, Khalaf and DeVenny [33] in their articles stated that standard mix design method could be used with the following modifications to design concrete mixes containing recycled aggregates:

- When designing a concrete mix using recycled aggregate of variable quality, a higher standard deviation should be employed in order to determine a target mean strength on the basis of a required characteristic strength.

- When coarse recycled aggregate is used with natural sand, it may be assumed at the design stage that the free w/c ratio required for a certain compressive strength will be the same for recycled aggregate concrete as for conventional concrete. If trial mixes show that the compressive strength is lower than required, an adjustment of the w/c ratio should be made.
- For a recycled aggregate mix to achieve the same slump, the free water content will be needed approximately 10 L/m³ higher than that of conventional concrete.
- If the free water content of recycled aggregate concrete is increased, the cement content will also need to be higher to maintain the same w/c ratio.
- Trial mixes should be made to obtain the required workability, suitable w/c ratio, and required strength.

3.4. Mechanical Properties of Recycled Aggregate Concretes

The mechanical properties of concretes produced with recycled aggregates will invariably be different than those produced with natural aggregates. Quality of concrete with recycled aggregate is generally lower than that of virgin aggregate. The main reason is that recycled aggregate with its higher water absorption property has porous mortar matrix around the virgin aggregate and hence develops an inferior bond [34]. It is generally expected that the compressive strength, tensile strength, flexural strength, shear strength, and modulus of elasticity values of the recycled aggregate concrete will be lower than those of natural aggregate concretes, while creep and shrinkage will be higher values. The actual percent difference in each of the above properties depends on a number of factors like the fraction of total aggregates that is recycled, the characteristics of the original concrete, the nature and level of contaminants present, the amount of fines, the quantity of attached mortar, etc. Investigations have aimed at determining the "optimum" combination of these factors that will economically produce a recycled aggregate concrete that is suitable [19].

In literature review, it is found that both mechanical and durability properties of concrete varies from study to study. This is because of the large variety of the aggregate combinations, regional differences, different production methods and storage conditions, as

well as other influencing factors such as different curing and test methods and used standards. In this part of the literature review, some recent investigations on mechanical properties are provided.

Xiao *et al.* [35] stated in their article that none of the previous results indicated that recycled aggregate concrete is unsuitable for structural applications. According to their literature survey, recent investigations on the performance of beams, columns, beam-column joints, and slabs made from recycled aggregate concrete gave positive results, which further supports and encourages the possibilities of applying recycled aggregate concrete in civil engineering structures.

Limbachiya *et al.* [20] examined the influences of coarse aggregates on high strength, C50 or greater, concrete performance. A series of concrete mixes were cast to assess the effect of coarse recycled concrete aggregate (RCA) on ceiling strength. RCA concrete mixes were proportioned using natural aggregate (NA) and RCA blends, with up to 100 percent coarse RCA. Tests were carried out at ages of 7, 28, 60 and 90 days. Using the test results strength versus w/c ratio plots were produced as shown in Figure 3.5. The general trends observed indicated that up to 30 percent coarse RCA had no effect on the ceiling strength and thereafter a gradual reduction occurred with increasing RCA content. At 28 days, as it is seen from Figure 3.5, the maximum attainable strength for concrete with 100 percent RCA was less than 75 MPa, compared to over 80 MPa for concrete containing up to 30 percent coarse RCA. The results showed that high levels of RCA content have an effect on the concrete strength.

Ajdukiewicz and Kliszczewicz [13] presented their findings after their experimental study. The tests were done with use of different recycled aggregates obtained from original moderate or high strength concrete, 2-7 years old, crushed up to three months before use. Following general conclusions have been derived from the analysis of test results:

- Properties of original concrete has significant influence on mechanical properties of recycled aggregate concrete; it is possible to obtain recycled concrete with higher compressive strength than the original one.

- Mix design of recycled concrete is very similar to the procedure for concrete with natural (new) aggregate; corrections in water content are necessary to obtain proper workability, but the changes in water/cement ratio may be relatively small.
- The replacement of fine fraction 0–2 mm in recycled aggregate by natural sand always changes for the better the properties of recycled concrete.
- Properties of recycled concrete may be significantly improved by admixtures of superplasticizers and silica fume, similarly to cases of concrete with natural aggregates.

Their more detailed conclusions concerning particular properties are the following:

1. Compressive strength, f_{cm28} , was achieved over 80 MPa using recycled aggregate from original concrete about 60 MPa, using reasonable content of admixtures. The characteristics $\sigma_c - \varepsilon_c$ at compression appeared very similar for concrete with natural and recycled aggregate, with slightly lower modulus of elasticity in recycled concrete.
2. Tensile strengths for mixes with new aggregates are always higher, but the differences are not greater than 10 percent for 28 days old concrete; the influence of admixtures on tensile strength is much greater than influence of introduction of recycled aggregate.
3. The values of bond stress at failure, τ_b , were measured lower for recycled aggregate concrete, particularly for plain round bars. For concrete with fully recycled aggregate the average drop was up to 20 percent while in case of replaced fine aggregate with natural sand it was up to 8 percent only.
4. Regarding problem of freezing resistance it was stated that high performance concrete made from recycled aggregate, when subjected to freeze-thaw action, had similar or better durability compared to the concrete made with natural aggregate. This conclusion is supported by standard tests with 50 cycles of freezing and thawing, with 24 h cycle (air-freezing and water-thawing).
5. The long-term tests showed the higher shrinkage and slightly lower creep in recycled concrete in comparison with concrete with natural (new) aggregate of similar strength.

The researchers finally concluded that recycled aggregates obtained from moderate or high strength concrete with original granite or basalt aggregates appeared fully useful component for high performance concrete. For many reasons the replacement of fine fraction with natural sand is advisable.

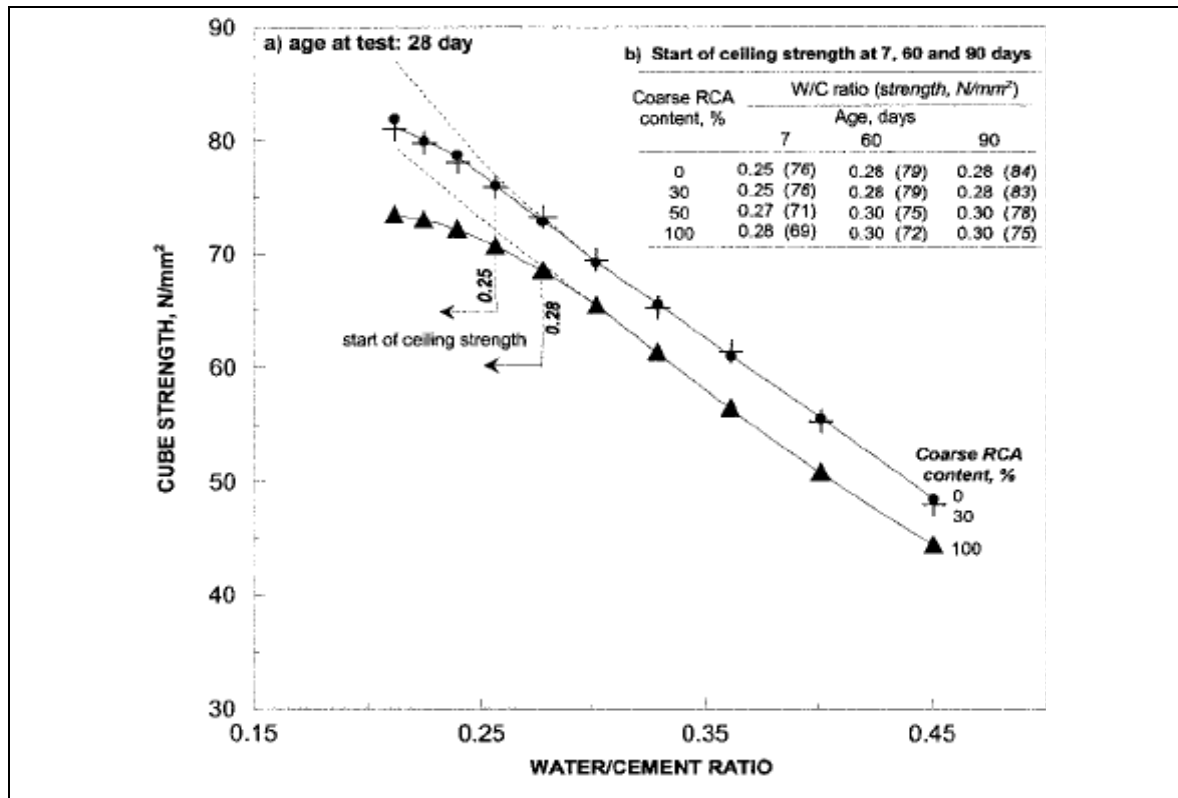


Figure 3.5. Start of ceiling strength at a) 28 days and b) 7, 60 and 90 days of RCA and NA concretes [20]

Etxeberria *et al.* [36] carried out an experimental investigation to study the possibilities of the use of concrete made with RCA as a structural material. Twelve beams made with different amounts of RCA, maintaining the same concrete compressive strength, and different amounts of transverse reinforcement have been tested to shear failure. From the observed experimental behaviour and from the comparisons with analytical predictions made, they made following conclusions:

1. The effect of the use of recycled aggregate on the beams' shear strength depends on the percentage of coarse aggregate substituted, especially for beams without

transverse reinforcement. For low percentages of substitution (less than 25 percent) it can be said that this influence is practically negligible.

2. For beams without shear reinforcement, the use of recycled coarse aggregate reduced the cracking load. This happened even though the compressive concrete strength was maintained the same for all beams by increasing the amount of cement or reducing the effective water/cement ratio if necessary. This was probably due to the fact that cracking occurs at the weakest point that is the adhered mortar on the recycled concrete aggregate.
3. For beam specimens with web reinforcement, the influence of the amount of RCA in the ultimate shear is very small. Beam specimens with web reinforcement and concrete with 50 percent and 100 percent of coarse recycled aggregate achieved approximately the ultimate shear load of conventional concrete. Only in the case of beams made with 25 percent of recycled aggregates, in which the required concrete compressive strength was reached without changing the amount of cement or the effective water/cement ratio, a 13 percent decrease in the shear strength was observed. These results suggest that the use of recycled concrete aggregate must be accompanied by a modification of the dosage (increase of cement amount and decrease of w/c ratio) in order to avoid a decrease on the shear strength.
4. In general, Code provisions for shear design overestimate the shear strength of beams made with high amounts (more than 50 percent) of RCA. However, they offer good correlation for beams made with small amounts (25 percent or less) of RCA.
5. The theoretical methods checked in the study provide good results for all beams made with conventional concrete. For beams with recycled aggregate concrete, they provide good results for beams with stirrups and for beams without stirrups with a percentage up to 25 percent of RCA.
6. Aggregate interlock and bond between concrete and the reinforcement are the mechanisms that look to work correctly in reinforced concrete beams made with RCA, even for high percentages of substitution of coarse aggregate.
7. As a general conclusion, it can be said that concrete made with up to 25 percent of RCA is suitable for structural use, provided that all measures related to dosage; compressive strength and durability aspects have been adopted. In that case, the current code provisions for shear design result adequate for their use to design RC structures with RCA.

Chen *et al.* [37] used the building rubbles as recycled aggregates. They conducted a series of experiments using recycled aggregate of various compositions from building rubble. The test results showed that the building rubble could be transformed into useful recycled aggregate through proper processing. Using unwashed recycled aggregate in concrete affected its strength. The effect was more obvious at lower water/cement ratios. When the recycled aggregate was washed, these negative effects were greatly improved. This was especially true for the flexural strength of the recycled concrete. The recycled coarse aggregate was the weakest phase at a low water/cement ratio. This effect dominated the strength of recycled concrete.

Kheder and Al-Windawi [38] studied the effect of binding mortar strength on the mechanical properties of recycled aggregate concrete (RAC) as well as reference corresponding natural aggregate concrete (NAC) mixes. Two groups of concrete mixes, NAC and RAC, were produced using natural sand; a third group of mortar mixes, with mix proportions similar to those of RAC group mixes, except that they were cast without coarse aggregate. Four main mechanical properties, namely modulus of elasticity, compressive strength, flexural strength and splitting tensile strength of the concrete and mortar mixes were studied. Test results showed that the modulus of elasticity of NAC and RAC exceeded that of corresponding mortar by about 40 percent and 10 percent respectively, for all compressive strengths investigated, in addition, the modulus of elasticity of RAC is about 20-25 percent lower than NAC. The compressive strength of RAC depends largely on the W/C ratio of the mix. The flexural strengths of both NAC and RAC were lower than that of mortar by about 5-28 percent and 20-39 percent, respectively. The difference decreased with the increase in compressive strength of the mix. The splitting tensile strength of NAC was higher than that of mortar by 13-69 percent, this difference increased by the decrease of strength, while RAC splitting strength was lower than that of mortar for mixes of high strength, and with the decrease in compressive strength, it became higher than that of mortar, the difference was between -13 to 36 percent.

Various studies [11, 16, 24, 32, 34, 39-45] showed that unit weight, Schmidt hardness [26], compressive strength, modulus of elasticity, flexural strength, splitting tensile strength values of recycled aggregate concrete decreased, while peak strain corresponding to the peak stress [31], shrinkage [46], creep [47] values of recycled

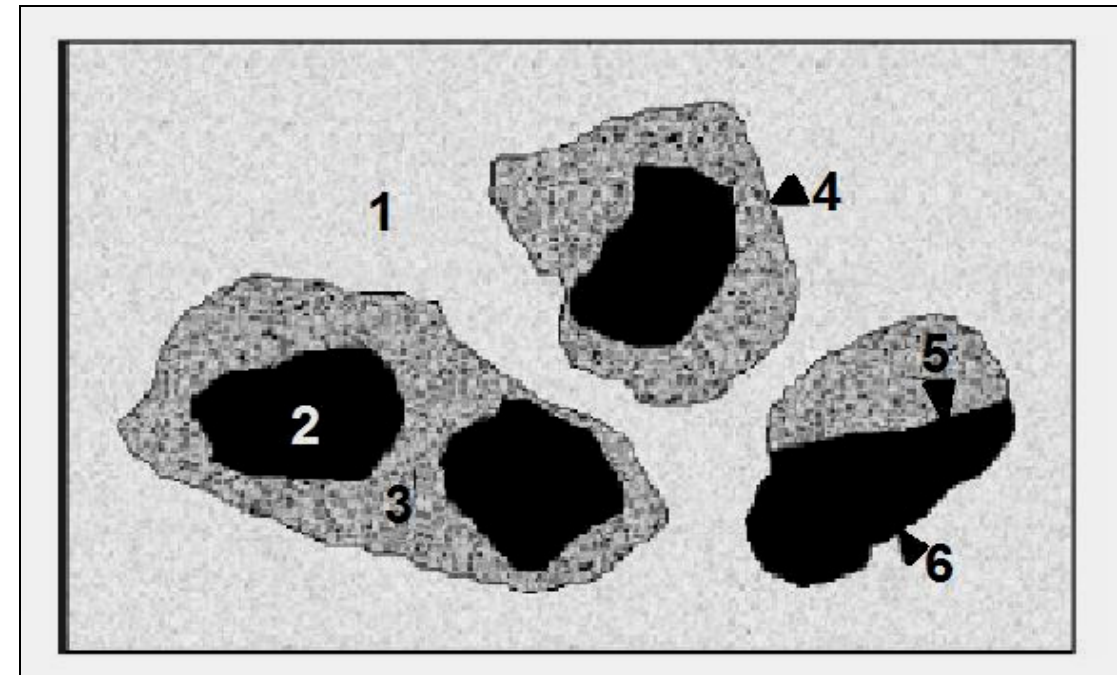


Figure 3.6. The constituents of a concrete incorporating recycled coarse aggregate [49]

The differences of recycled aggregate characteristics cause the different quality recycled aggregate concretes. However, Fukute *et al.* [43] concluded that it is possible to ensure the same quality level as normal concrete by reducing water cement ratio against normal concrete.

3.5. Durability Properties of Recycled Aggregate Concretes

A number of definitions were made for durability by researchers [50-52]. One definition was made by Neville [50] in year 2000; he means by durability that the given concrete structure will continue to perform its intended functions, that is, to maintain its required strength and serviceability, during the specified or traditionally expected service life. In year 2006, Bimel [51] gave the definition of durability made by American Concrete Institute (ACI). According to ACI (ACI 201.2R-01, “Guide to Durable Concrete”) durability of hydraulic cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. According to the article published in year 2004 [52], durability is not a characteristic of concrete in general, or of a given class or given levels of a set of properties, but it is rather a conclusion reached about a specific concrete subject to specific service environment. A concrete is durable if, in its environment, it has provided the desired service life, without excessive cost for maintenance and repair due to degradation or deterioration.

The durability search for concrete revealed that in the studies the questions are asked to answer firstly how to produce a more durable concrete secondly how to measure the durability of concrete. Like academic researchers, engineers in the industry try to answer the cited questions.

Freezing and thawing, chemical attack, abrasion resistance, corrosion of embedded metal, and alkali aggregate reaction are the important parameters to know about concrete durability [52].

RILEM Technical Committee 130-CSL in year 2000 has published [53] durability design of concrete structures. The committee summarized the degradation factors as follows:

1. Corrosion due to chloride penetration
2. Corrosion due to carbonation
3. Mechanical abrasion
4. Salt weathering
5. Surface deterioration
6. Frost attack

Additionally there exist some internal degradation processes, such as alkaline-aggregate reaction, that are not primarily caused by environmental stresses. Such degradation problems are solved by a proper selection of raw materials and an appropriate design of concrete mix. The committee also stated the main structural effects of degradation in concrete and steel as follows:

1. Loss of concrete leading to reduced cross sectional area of the concrete.
2. Corrosion of reinforcement leading to reduced cross sectional area of steel bars. Corrosion may occur at cracks and at all steel surfaces, assuming that the corrosion products are able to leach out through the pores of the concrete (general corrosion in wet conditions).
3. Splitting and spalling of the concrete cover due to general corrosion of reinforcement, leading to a reduced cross sectional area of the concrete, to a reduced bond between concrete and reinforcement and to visual unfitness.

Topics of RILEM durability design procedure are only listed here as follows:

1. Specification of the target service life and design service life
2. Analysis of environmental effects
3. Identification of durability factors and degradation mechanisms
4. Selection of a durability calculation model for each degradation mechanism
5. Calculation of durability parameters using available calculation models

6. Possible updating of the calculations of the ordinary mechanical design (*e.g.* own weight of structures)
7. Transfer of the durability parameters into the final design

Measuring the durability of concrete was another issue. A review of the current common methods for determining the chloride penetrability of concrete is presented by Stanish *et al.* [54]. They listed the existing methods in their study as follows: AASHTO T259 (salt ponding test), bulk diffusion test (Nordtest NTBuild 443), AASHTO T277 (rapid chloride permeability test), electrical migration techniques, rapid migration test (CTH test), resistivity techniques, pressure penetration techniques, sorptivity, indirect measurement techniques (water and gas permeability), diffusion measurement of propan-2-ol, and gas diffusion.

Undoubtedly the durability of recycled aggregate concretes has to be investigated as an important parameter for evaluation of the recycled aggregate concrete performance. The durability of concretes containing recycled aggregates is also affected by the factors mentioned in heading 3.4 (Mechanical Properties of Recycled Aggregate Concretes). However, the most significant one is the presence of contaminants. Many of these induce harmful reactions in the concrete that can shorten its life. For example, gypsum from drywalls, if at significant levels, causes expansive reactions which promote cracking in concrete. Another important factor affecting durability is the extent to which the original concrete has undergone harmful reactions, as these have the potential to continue in the new concrete. Furthermore, the permeability and water absorption of recycled aggregate concrete are increased due to the attached mortar; so, durability can also be influenced in this way. For these reasons, work has also been carried out to understand these effects and to reduce them since a good concrete not only has to possess good mechanical properties but must also be durable enough to serve its expected service life [19].

Limbachiya *et al.* [20] investigated the various aspects of durability relevant to the use of high strength concretes, covering initial surface absorption, air permeability, chloride diffusion, chloride induced corrosion, freeze thaw and abrasion. After they performed the durability tests, they concluded that recycled aggregate concrete mixes with 30 percent recycled aggregate content, for high strength concrete, were found to possess

durability properties, namely resistance to chloride ingress, chloride-induced corrosion, freeze thaw and abrasion, similar to the corresponding natural aggregate concrete mixes.

Zaharieva *et al.* [55] presented their research that deals with concrete containing building waste recycled as aggregates. The frost resistance was used as a durability indicator. The frost resistance of recycled aggregate concrete (RAC) was compared with the one of a reference natural aggregate concrete called NAC. The mix proportion of NAC enabled to resist to 300 freezing thawing cycles. Three types of RAC were studied: 1. concrete with coarse recycled aggregate (RA) and natural sand (RAC1), 2. concrete made entirely of RA, where RAs were water presoaked (RAC2), and 3. concrete made entirely of RA, where RAs were not presoaked (RAC3). According to the conclusion of the authors, generally, the frost resistance of saturated RAC was not satisfying, and their use in structures exposed to severe climate was not recommended. The main reason seemed to be the high total water cement ratio, inducing higher porosity and lower mechanical characteristics of RAC, as well as the frost resistance of RA themselves: first, they might contain unsound particles, which would be deteriorated by the repeated action of the freezing thawing cycles, and, second, RA could contribute to the frost damage by expelling water into the surrounding cement paste during the freezing period. Only the mixed concrete (RAC1), with a water cement ratio lower than 0.55 could be used in moderate cold climate.

Sagoe-Crenstil *et al.* [30] determined that the concrete containing recycled aggregate as the coarse aggregate fraction reduced the abrasion resistance by about 12 percent compared to the reference concrete made with basalt aggregates. Examination of water absorption and carbonation rates have shown little difference between commercially produced recycled concrete and the reference concrete having a cement content of 242 kg/m³.

Durability studies have been done by Rao *et al.* [56] in order to better understand the effect of using different qualities of recycled aggregate (RA) on the properties of the recycled aggregate concrete (RAC). Though, some studies have shown that RAC is significantly more permeable than natural aggregate concretes (NAC), it should also be noted that the durability properties can be improved by using fly ash, condensed silica

fume, etc. On the basis of carbonation test done after 6 months of curing, the carbonation depth of the recycled concrete has been found to be 1.3–2.5 times greater than that of the reference concrete. For the same water-binder ratio, the carbonation depths of RAC were slightly higher than that of NAC. This increase in the carbonation depth could be attributed to increased permeability of the RAC on account of the presence of old mortar adhering to the original aggregate, and the old interfacial transition zone (ITZ) between them. According to the authors, there was no common opinion in the literature as far as the frost resistance of RAC is concerned. In a study where the effect of mortar content adhering to the aggregate on the freezing and thawing resistance of RAC was studied, it was found that provided the quality of the concrete rubble was good, the adhering mortar may not adversely affect the performance of RAC. In another study it was found that the freezing and thawing resistance of RAC using RA made from non-air-entrained concrete was quite poor, though the RAC met the requirements of air entrainment. On the other hand, the concretes made with the recycled coarse aggregates originated from air-entrained concretes were highly frost resistant. The likely shortcomings in the performance of RAC to freezing and thawing can be attributed to the pore structure of the previously hardened cement paste that adheres to the surface of the recycled aggregate. This porous matrix absorbs water during mixing, increasing the water-cement ratio of the paste.

In another study of Zaharieva *et al.* [57] the water permeability, air permeability and surface permeability of recycled aggregate concrete (RAC) were compared with those of a control concrete made with natural aggregate. In terms of durability of RAC, the study showed that RAC was significantly more permeable than NAC. Thus their long-term performance was questionable. The replacement of natural aggregates by recycled aggregates affected the quality of the concrete cover. The carbonation of RAC was faster. This effect limits the use of recycled aggregate in the production of reinforced concrete elements. Nevertheless, based on the criteria proposed in other studies, RAC can be characterized as being of moderate quality rather than of poor quality. A possible use of admixtures such as fly ash or silica fume could decrease significantly porosity and permeability of RAC. The study showed that the permeability was more dependent than other parameters to mix-proportions and curing conditions. Mixed aggregate concrete was intermediate between RAC and NAC. It can be concluded that the main problems of durability were caused by the use of recycled sand. Therefore, the use of fine recycled

aggregate needed to be restricted. Another way of increasing the durability of RAC was to use extended curing using a moist environment.

Olorunsogo and Padayachee [58] presented their report of an investigation into the performance of concrete manufactured with recycled aggregate (RA) using durability indexes as indicators. Durability indexes, such as chloride conductivity, oxygen permeability and water sorptivity, of three different concrete mixes containing zero, 50 and 100 percent RA were monitored at ages 3, 7, 28 and 56 days. The results showed that, as shown in Figure 3.7, durability quality reduced with increase in the quantities of RA included in a mix; however, as expected, the quality improved with the age of curing. At the age of 56 days, increases in index value of a concrete mix made with 100 percent RA over that made with 100 percent natural aggregate were 86.5 percent and 28.8 percent, respectively, for chloride conductivity and water sorptivity. The corresponding value of oxygen permeability index (OPI) for the same concrete mixes was a reduction of 10 percent. For 50 percent RA concrete, the reductions in chloride conductivity and water sorptivity indexes at the curing age of 56 days compared to 3 days were 62.7 percent and 42.7 percent, respectively. The corresponding figure for OPI was an increase of 37.6 percent. The poor performance of the RA concrete was associated with the cracks and fissures, which were formed in RA during processing, so rendering the aggregate susceptible to permeation, diffusion and absorption of fluids.

Levy and Helene [59] analyzed the water absorption, total pores volume, and carbonation properties of recycled aggregate concretes. Fine and coarse recycled aggregates recovered from demolished masonry and concrete structures were utilized in the manufacture of new concrete mixtures. The recycled concrete families were created by replacing parts of the natural aggregates forming families of concrete with 0, 20, 50, and 100 percent of aggregates from recycled sources. The result show that minimum water absorption and total pore volume for the recycled aggregates concrete were observed at 20 percent replacement when using coarse recycled concrete aggregates (CRCA) and coarse recycled masonry aggregates (CRMA). These can be explained by grade aggregate factors of compacting and the ideal pozzolanic proportions. For all other conditions performed in this investigation, when water absorption and total pore volume are increased, the replacement of recycled aggregate also increases. In the recycled aggregates concrete

performed in this study, the carbonation depth decreased when the replacement was 20 percent or 50 percent. For CRMA concrete family, this better behavior also occurred when the replacement was 100 percent. This behavior shows that carbonation depth depends strongly on the chemical composition of the concrete and not only on the physical aspects. When the natural aggregate is replaced by 20 percent of the recycled aggregates from old concrete or old masonry, the resulting recycled concrete will likely present same, and sometimes better, behavior than the reference concrete made with natural aggregates in terms of the properties studied in this investigation. This fact justifies the efforts to use these concretes, which can contribute to the preservation of the environment and can achieve the same final performance with probably less cost than ordinary concretes.

The other studies showed that the durability properties of recycled aggregate concrete like water absorption [11, 60-62], sorptivity [61], water penetration depths [63], water permeability [64], skid resistance [60], abrasion [60], carbonation [11, 34, 61, 65] deterioration of concrete due to sulfate attack [66], chloride ion penetration [64], alkali silica reaction [67] increased, while freeze-thaw resistance [13, 34, 65, 68-69], concrete resistivity [70], chloride permeability [70], alkali-silica reaction [71], sorptivity [11] were approximately same, and freeze and thaw resistance [49], chemical resistance towards the expose acids salts solutions [72], decreased with increasing recycled aggregate content. Several factors like the waste concrete type, crushing age of source concrete, water cement ratio of source concrete, quality of source concrete and contaminant amount, type of crushing process, adhered mortar content of the recycled aggregate, size of the recycled aggregate, moisture state of the recycled aggregate, recycled aggregate content in recycled aggregate concrete, and of course the properties of new recycled aggregate concrete, design method, and measuring way of durability are the causes of differences in publications. However, the studies [73-75] encouraging that recycled aggregates can be used to produce a good quality structural concrete with durability properties similar to those of concrete made with natural aggregates. Good durability properties can be obtained by lowering water cement ratio of recycled aggregate concrete and using admixtures like fly ash, slag and silica fume, and also by sufficient curing.

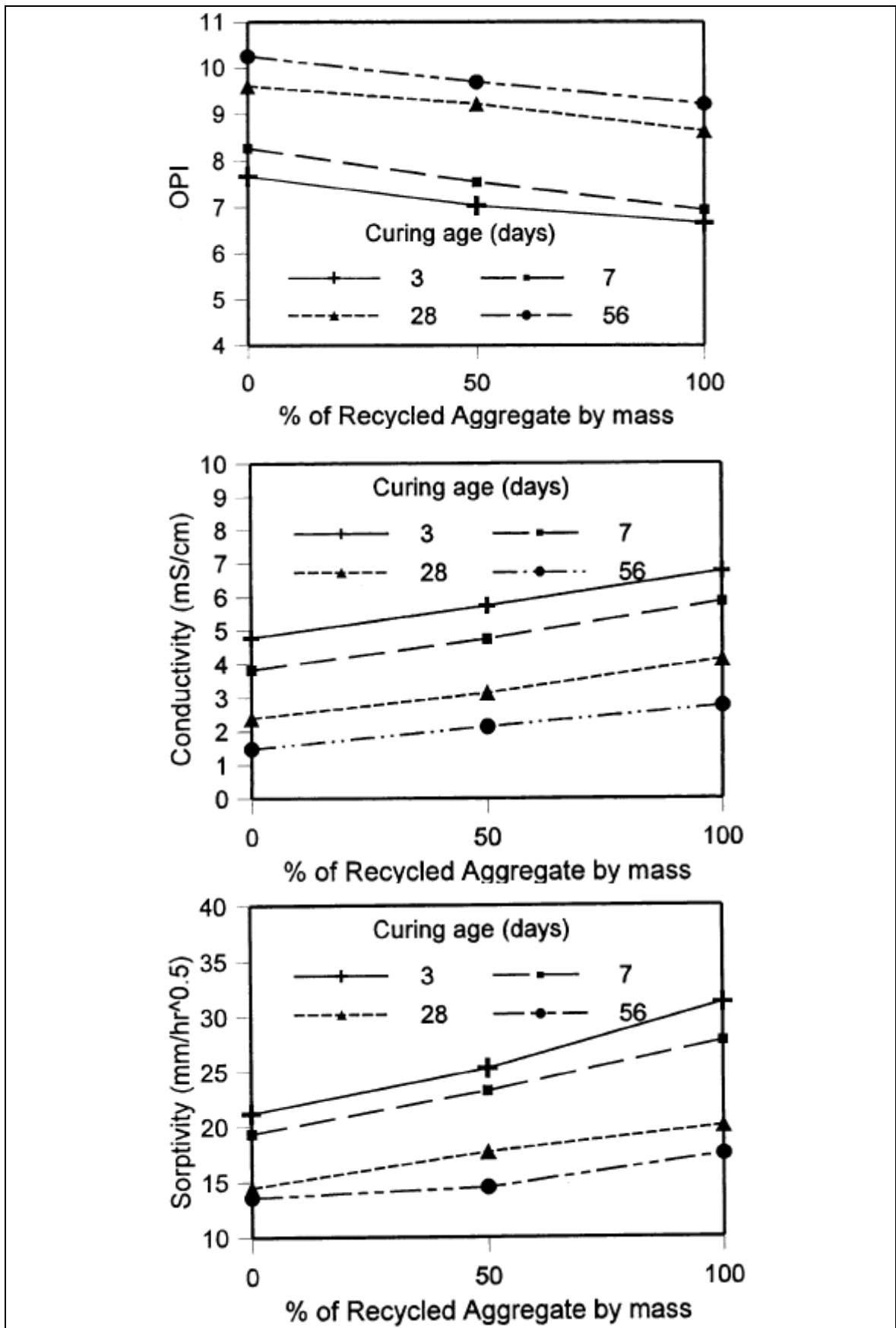


Figure 3.7. Effects of recycled aggregate content on durability properties [58]

3.6. Effects of Elevated Temperature on Concrete

Almost all concrete structures are subjected to a range of temperature imposed by environmental conditions. Occasionally, concrete structures are subjected to high temperatures, e.g., reactor vessels, thermal shock, fire, coal gasification vessels, some industrial applications [76], and jet aircraft engine blasts. When a cement-based composite material is subjected to elevated temperatures, the difference in thermal properties of the cementitious matrix and the aggregate inclusions induces thermal stresses and possibly cracking in the concrete [77]. Thermal cracking induced by thermal mismatch has been one of the problems in a cement-based composite material under elevated temperatures [78]. This type of cracking significantly reduces compressive strength and elastic modulus of concrete, also when reinforced concrete is subjected to high temperatures yield stress, ductility and tensile strength of steel decrease. As a result, the bond between them decreases [79]. Masonry walls are also affected by such elevated temperatures [76]. Volume stability [80] and durability [81] decrease besides toughness [82] and drying shrinkage [81] increase with increasing heating temperature. Due to the fire risk, the properties of all construction materials at ambient and elevated temperature conditions are of interest and therefore have received special attention in the literature [83-90].

It is a need to review the thermal damage mechanism of concrete. In literature, reasons for thermal damage of concrete are listed; these are thermal incompatibility (mismatch) [76-79, 84, 86, 89-95], decomposition of hydration products [76-80, 81-82, 85-86, 89, 91, 93, 96-99], and pore pressures [80, 84-85, 88, 90-92, 96-97, 100-104].

Cülfik [105] presented the reasons for thermal incompatibility. When concrete is exposed to temperature changes, unequal volume changes of its components cause tensile stresses and cracking of concrete, reducing its durability and strength. This phenomenon is called thermal incompatibility, and it appears because of the difference between the coefficients of thermal expansion of the cement paste and aggregate. As a result of thermal incompatibility, thermal stresses are induced between the expanding aggregate and shrinking cement paste. The induced stress results in break down of the interfacial bond between the aggregate and surrounding cement paste that results in strength loss of concrete specimens.

Decomposition of hydration products is the other reason for thermal damage. According to Gawin *et al.* [97], in heated concrete, above temperature 105 °C starts the thermal decomposition of the cement matrix, and at higher temperatures also of aggregate (depending on its type and composition). This is a consequence of several complicated, endothermic chemical reactions, called concrete dehydration. Arioz [80] reviewed the changes in chemical composition and physical structure of the concrete when they were exposed to high temperature. The dehydration such as the release of chemically bound water from the calcium silicate hydrate (CSH) becomes significant above about 110 °C. The dehydration of the hydrated calcium silicate and the thermal expansion of the aggregate increase internal stresses and from 300 °C micro cracks are induced through the material. Calcium hydroxide [Ca(OH)₂], which is one of the most important compounds in cement paste, dissociates at around 530 °C resulting in the shrinkage of concrete. The alterations produced by high temperatures are more evident when the temperature surpasses 500 °C. Most changes experienced by concrete at this temperature level are considered irreversible. CSH gel, which is the strength giving compound of cement paste, decomposes further above 600 °C. At 800 °C, concrete is usually deteriorated and above 1150 °C feldspar melts and the other minerals of the cement paste turn into a glass phase. As a result, severe micro structural changes are induced and concrete loses its strength and durability. The physicochemical changes in the concrete during heating are illustrated very well by Khoury [90] as in Figure 3.8.

The third reason for the thermal damage is pore pressure. During exposure to high temperatures, very high pore pressure may be built up as functions of temperature, heating rate, and size of the specimens [84]. It is commonly believed that the main reasons of the thermal spalling are: build up of high pore pressure close to the heated concrete surface as a result of rapid evaporation of moisture, and the release of the stored energy due to the thermal stresses resulting from high values of restrained strains caused by thermal gradients [97]. As shown in Figure 3.9, explosive spalling generally occurs under the combined action of pore pressure, compression in the exposed surface region (induced by thermal stresses and external loading) and internal cracking [90]. The pore pressure might be as high as 8 MPa, which is much higher than the concrete's tensile strength of about 5 MPa. Such a pressure could cause a dramatic type of cracking, generally known as thermal spalling of concrete, which has been reported by many researchers [84]. The pore pressure

shows its effects especially in high performance concretes. Although high performance (HPC) concrete has been shown to have a number of advantages when used in concrete structures, it suffers from one major weakness: higher brittleness. When exposed to high temperatures, HPC exhibits more serious degradation than normal concretes do, such as spalling and cracking [106].

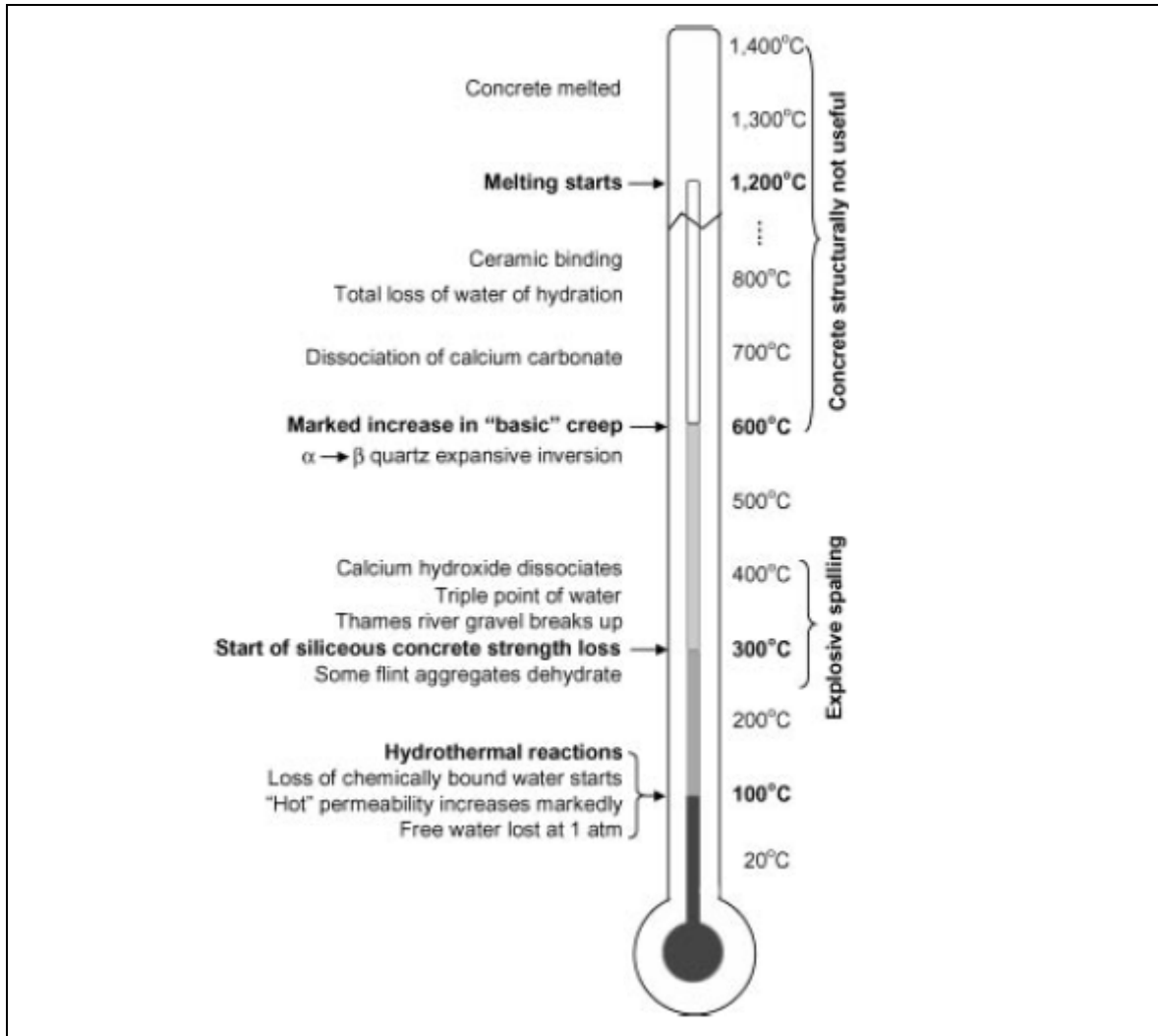


Figure 3.8. Physicochemical changes in portland cement concrete during heating [90]

The relatively low performance of the cement paste (contrary to its high durability performance at room temperature) may be attributed to the dense, low permeability structure of the paste which does not readily allow moisture to escape from the heated concrete, thus resulting in high pore pressures and the developments of microcracks [90].

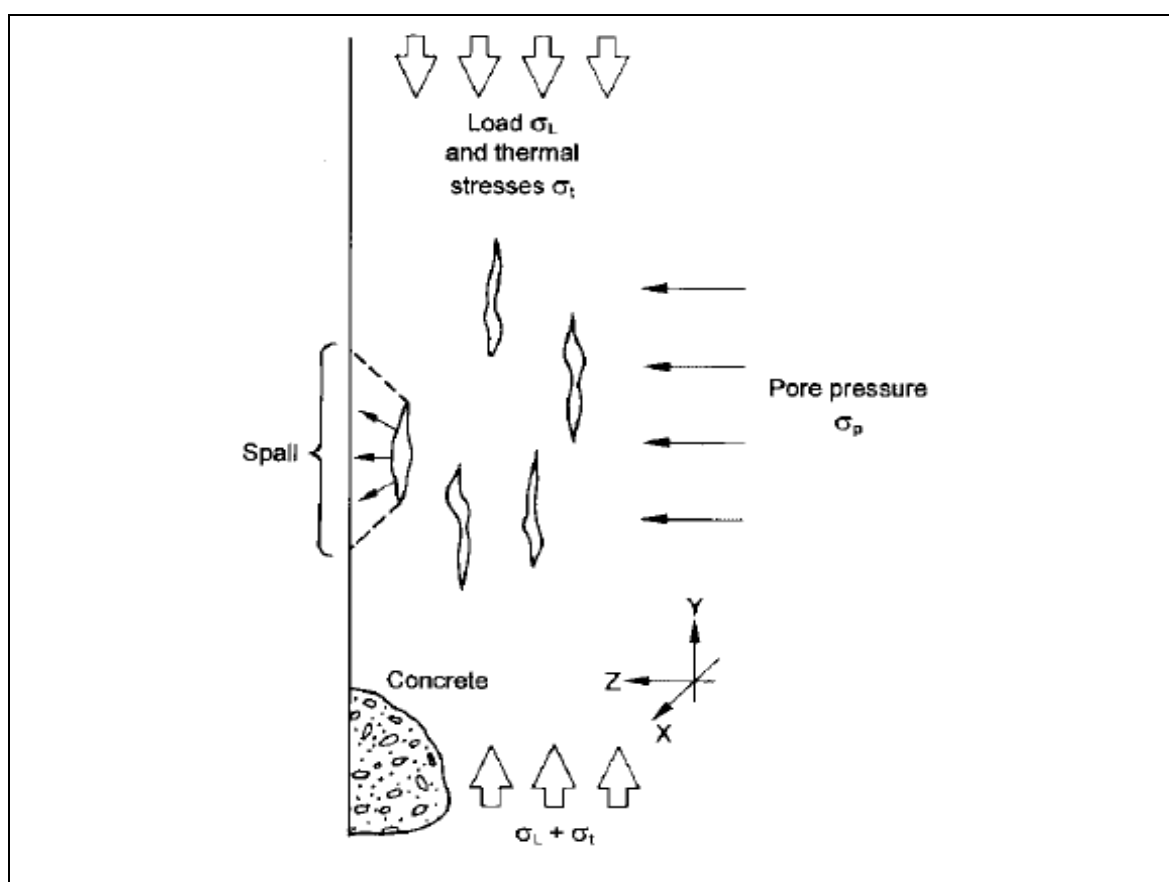


Figure 3.9. Forces acting in heated concrete [90]

Thermal incompatibilities, decomposition of hydration products and pore pressure have been examined as reasons for reduction in concrete properties. Changes in mechanical properties [76, 84, 86, 89, 92, 96, 107-110], durability properties [84, 87, 93, 96, and 109], and microstructure [81, 99, 107, and 111], have been studied by many researchers either by experimental tests or numerical modeling [78, 103] in order to better understand the effect of elevated temperature on concrete so that precautions [83, 85, 91, 96, 100, 106, 112-114] can be taken to maintain the structural integrity.

The strength and durability performance of normal and high strength pozzolanic concretes incorporating silica fume, fly ash, and blast furnace slag was compared at elevated temperatures up to 800 °C by Poon *et al.* [96]. The strength properties were determined using an unstressed residual compressive strength test, while durability was investigated by rapid chloride diffusion test. It was found that pozzolanic concretes containing fly ash and blast furnace slag give the best performance particularly at temperatures below 600 °C as compared to the pure cement concretes. Explosive spalling

occurred in most high strength concretes (HSCs) containing silica fume. The high strength pozzolanic concretes showed a severe loss in permeability related durability than the compressive strength loss. Thirty percent replacement of cement by fly ash in HSC and 40 percent replacement of cement by blast furnace slag in normal strength concrete (NSC) was found to be optimal to retain maximum strength and durability after high temperatures. The residual compressive strength test results of their research are shown in Figures 3.10 and 3.11. The results of rapid chloride diffusion test of their research are illustrated in Figures 3.12 and 3.13. As seen from figures, the test results indicate that each temperature range showed a distinct pattern of strength loss or gain. From 20 °C to 200 °C, the fly ash and blast furnace slag concretes showed increases in strength with a greater increase in fly ash concretes. The maximum strength gain was shown by the HSC containing 40 percent fly ash, which was 122 percent of the original strength. From 200 °C to 400 °C, most HSCs maintained their original strength, while a significant decrease was observed in NSCs, which was 19-26 percent of the original strength. A severe loss in strength was observed in the 400-600 °C temperature range. The average loss was 44 percent in HSC and 60 percent in NSC. At 800 °C, all the concretes showed severe deterioration. A close examination of these results indicate a clear relationship between the residual compressive strength and concrete permeability, as more permeable specimens showed more pronounced loss of compressive strength.

Cülfik and Özturan [76] investigated the effect of elevated temperatures on the residual mechanical properties of high-performance mortar, observed the rate of heating, duration of exposure to maximum temperature and finally derived the following conclusions:

1. For high-performance mortars, serious drops in compressive strength started at 300 °C, and at 900 °C specimens lost almost all of their initial compressive strengths. At 300 °C, up to 29 percent of the initial compressive strength was lost and this value reached 65 percent at 600 °C.
2. The specimens that were exposed to elevated temperatures showed higher decrease in flexural strength than compressive strength. When exposed to 300 °C, high strength mortars showed losses up to 36 percent in flexural strength, and at 600 °C,



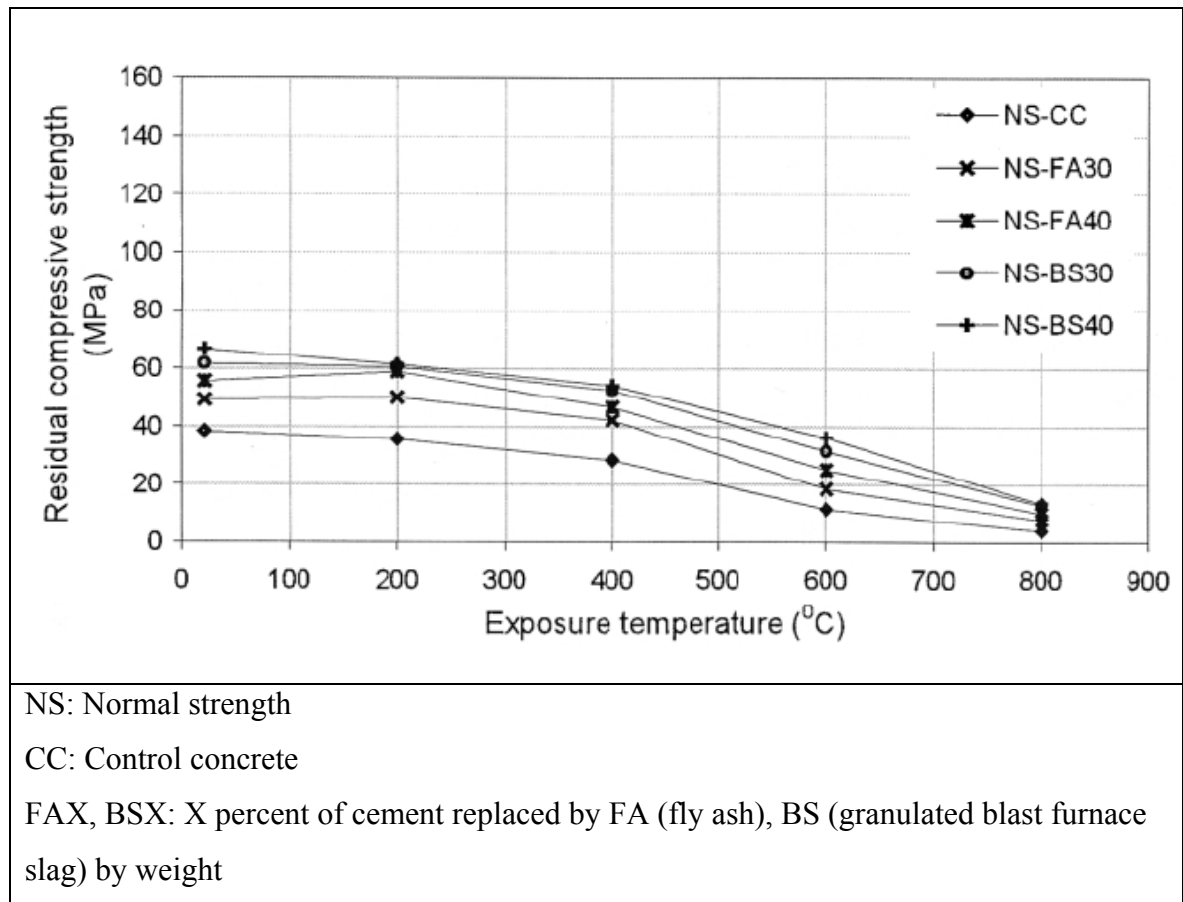


Figure 3.11. Residual compressive strength of NSCs [96]

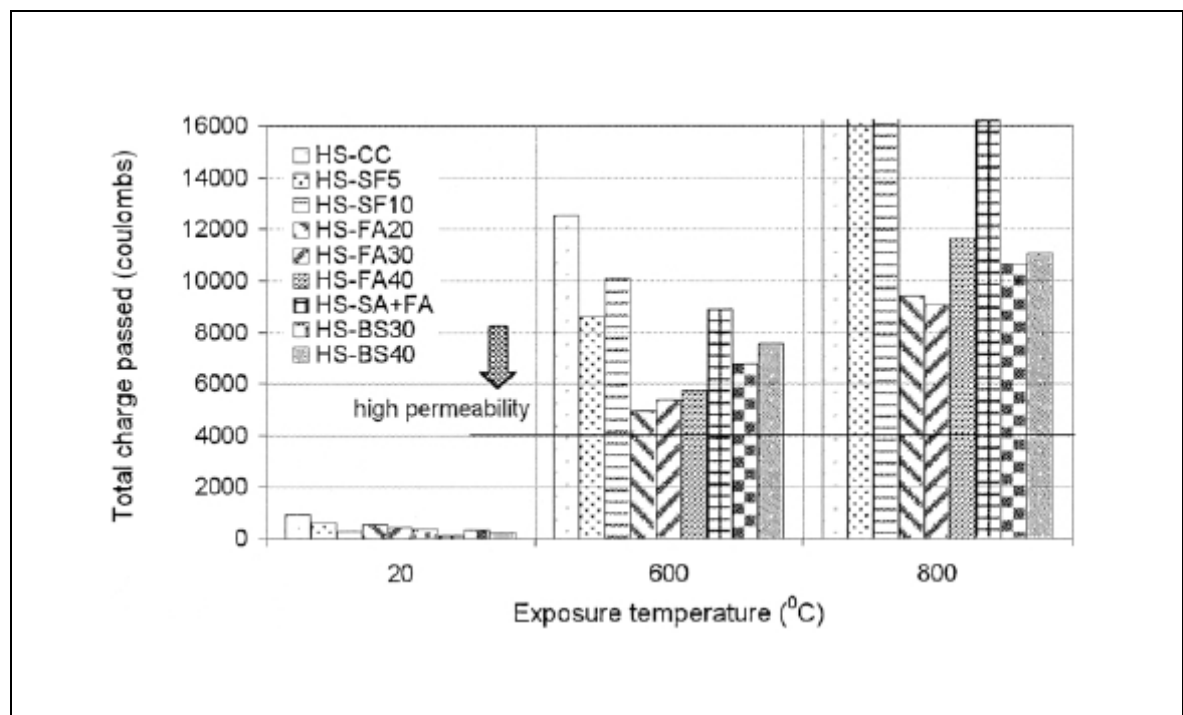


Figure 3.12. Resistance of HSC against chloride ion penetration [96]

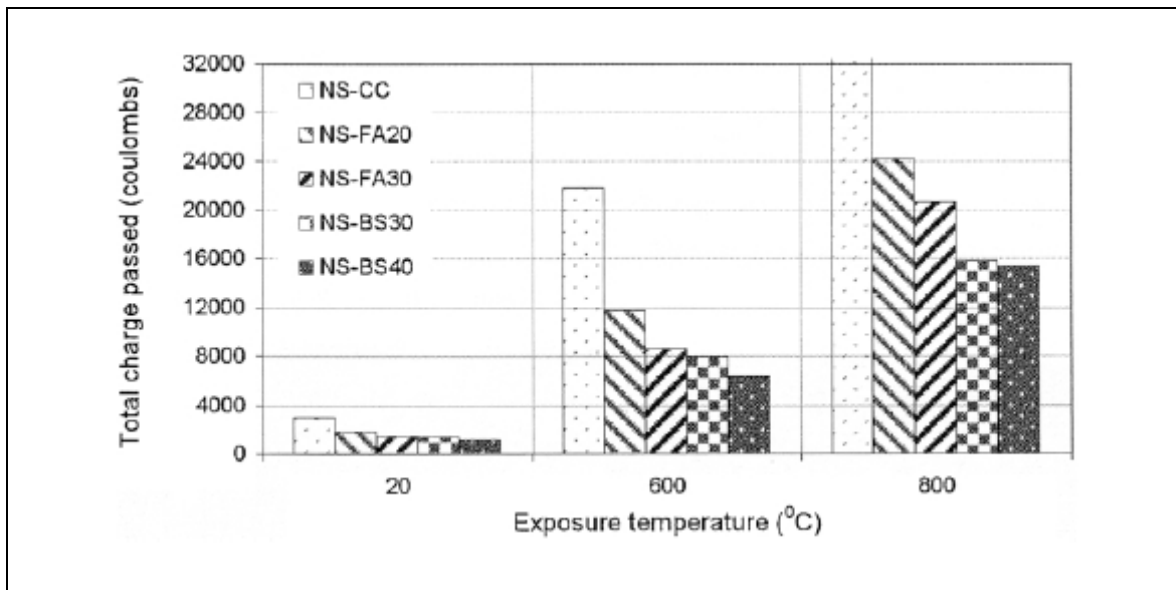


Figure 3.13. Resistance of NSC against chloride ion penetration [96]

Generally, it has been observed from the studies that the properties of concrete are adversely affected by elevated temperature exposure. The test results differ from study to study. This is obvious, because the tests were not performed under equal conditions. Factors affecting the results could be summarized as follows:

1. Mix constituents of the concrete
2. Sealing and moisture conditions
3. Heating rate
4. Degree of maximum temperature
5. Exposure time to maximum temperature
6. Rate of cooling
7. Cooling type, in air or in water
8. Test method, hot [stressed (loaded), unstressed] or cold [residual property]
9. Number of thermal cycles
10. Test specimen size and shape and values of the concrete cover

Although elevated temperature affect the mechanical and physical properties, it is important to maintain the structural integrity. For this purpose, it is seen from the articles that researchers testing concrete materials for elevated temperature service such as elevated temperature cements and high temperature aggregates and high temperature (refractory)

concrete mixes. In a study [112], gypsum free portland cement was tested at higher temperatures. Materials based on gypsum free portland cement resists better the effects of temperature over the range 20-1200 °C, because the content of Ca(OH)_2 is lower in hardened gypsum free cement concrete. Xu *et al.* [93] carried out a research to investigate the residual properties of pulverized fly ash concrete. According to their test results, they found that when pulverized fly ash was included an improvement of fire resistance as characterized by the residual compressive strength was observed. Peng *et al.* [100] reported that incorporating hybrid fiber (polypropylene fiber and steel fiber) seemed to be a promising way to enhance resistance of concrete to explosive spalling after they conducted an experimental study to explore the relationship between explosive spalling occurrence and residual mechanical properties of fiber toughened high performance concrete exposed to high temperatures.

In a study [91], after being subjected to different elevated heating temperatures, ranging between 105 °C and 1200 °C, the compressive strength, flexural strength, elastic modulus and porosity of concrete reinforced with one percent steel fiber (SFRC) have been investigated. The results show a loss of concrete strength with increased maximum heating temperature and with increased initial saturation percentage before firing. For maximum exposure temperatures below 400 °C, the loss in compressive strength was relatively small. Significant further reductions in compressive strength are observed, as maximum temperature increases, for all concretes heated to temperatures exceeding 400 °C. High performance concretes (HPC) start to suffer a greater compressive strength loss than normal strength concrete (NSC) at maximum exposure temperatures of 600 °C. It is suggested that HPC suffers both chemical decomposition and pore-structure coarsening of the hardened cement paste when C–S–H starts to decompose at this high temperature. Strengths for all mixes reached minimum values at 1000 or 1100 °C. No evidence of spalling was encountered. When steel fibers are incorporated, at one percent an improvement of fire resistance and crack resistance as characterized by the residual strengths were observed. Mechanical strength results indicated that SFRC performs better than non-SFRC for maximum exposure temperatures below 1000 °C, even though the residual strength was very low for all mixes at this high temperature. The variations with color, which occurred, are associated with maximum temperatures of exposure.

An experimental investigation [102] was conducted to evaluate the performance of metakaolin (MK) concrete at elevated temperatures up to 800 °C. Eight normal and high strength concrete (HSC) mixes incorporating zero, 5, 10 and 20 percent MK were prepared. The residual compressive strength, chloride ion penetration, porosity and average pore sizes were measured and compared with silica fume (SF), fly ash (FA) and pure ordinary Portland cement (OPC) concretes. It was found that after an increase in compressive strength at 200 °C, the MK concrete suffered a more severe loss of compressive strength and permeability related durability than the corresponding SF, FA and OPC concretes at higher temperatures. Explosive spalling was observed in both normal and high strength MK concretes and the frequency increased with higher MK contents.

Cheng *et al.* [113] investigated the effects of high temperature on the strength and stress-strain relationship of high strength concrete (HSC). Stress-strain curve tests were conducted at various temperatures (20, 100, 200, 400, 600, and 800 °C) for four types of HSC. The variables considered in the experimental study included concrete strength, type of aggregate, and the addition of steel fibers. Results from stress-strain curve tests show that plain HSC exhibits brittle properties below 600°C, and ductility above 600°C. HSC with steel fibers exhibits ductility for temperatures over 400 °C. The compressive strength of HSC decreases by about a quarter of its room temperature strength within the range of 100–400 °C. The strength further decreases with the increase of temperature and reaches about a quarter of its initial strength at 800°C. The strain at peak loading increased with temperature, from 0.003 at room temperature to 0.02 at 800 °C. Further, the increase in strains for carbonate aggregate HSC is larger than that for siliceous aggregate HSC.

In general, concrete is considered to have good properties with respect to fire resistance. The material is able to perform for a relatively long period of time, and no toxic fumes are emitted when it comes in contact with fire. Steel performs less well when subjected to fire, so concrete is often used as a protective material. In a typical fire, the temperature reaches about 500 °C in about 10 min, and 950 °C in 1 h, so the concrete must be able to withstand rapid temperature rises as well as a high final temperature. The rapid rise in temperature during a fire causes a buildup of steam pressure in the concrete voids that can cause explosive spalling of the concrete surfaces. This usually occurs during the first 30 min of exposure to heat. The concrete continues to degrade, with the gradual

separation of pieces caused by the formation of continuous fracture planes. During fires, temperatures of around 900 °C are commonplace, but only the outer layers of concrete members become drastically hot; the inner layers remain cooler, hence protecting steel reinforcement. Concrete strength is dependent on the cohesion of the cement paste, on its adhesion to the aggregate, and on the properties of the aggregate itself. Many commonly used aggregates break down physically and/or chemically when heated to higher temperatures, so the aggregate type is going to influence the ability of concrete to withstand high temperatures [33].

Eguchi *et al.* [65] investigated the characteristics of strength, durability, fire resistant property, structural performance, and workability of the recycled concrete. The water content of the recycled concrete was expected to be higher than that of ordinary concrete, due to the greater water absorption of the recycled coarse aggregate. The higher the water content, the higher the potential of the concrete spalling was expected in fire. A test was performed in order to evaluate the fire-resistant property by the authors. In the test, a cylindrical specimen of 15 cm diameter and 30 cm length was heated at the conditions specified in JIS A1304 “Method of Fire Resistance Test for Structural Parts of Building” (Japanese Industrial Standards, one hour heating up to 840 °C). The concrete specimen had water cement ratios of 0.50 and 0.60 and varying replacement ratios ranged from 0 to 100 percent. According to the test results, the water content after two months curing in air (at the time of the heating test) was 4.3–4.7 percent for the ordinary concrete. The water content of recycled concrete was higher as the replacement ratio increased and was 6.9–7.1 percent at 100 percent replacement ratio. However, spalling did not take place in any of the samples. Thus, the fire resistant property of the recycled concrete was not considered to be significantly different from that of ordinary concrete.

4. EXPERIMENTAL STUDY AND PROCEDURE

In this experimental study, the effect of quality of source concrete, by weight replacement ratio of natural coarse aggregates replaced by recycled coarse aggregates, water cement ratio, exposure temperature and age factors were investigated on natural and recycled aggregate concretes. The effects of source concrete quality on recycled aggregate concrete quality were examined in the first stage. The effects of water/cement ratio, recycled aggregate replacement ratio, exposure temperature, and age on the performance of natural plain concretes and recycled aggregate concretes were experimentally studied in the second stage in terms of their mechanical and durability properties. These properties of concretes were determined at ages of 28 and 90 days at room temperature and after exposure of concrete specimens up 105, 175 and 250 °C temperatures. Mechanical properties were determined from compressive, static modulus of elasticity and splitting tensile tests. Rapid chloride permeability, water absorption and capillary absorption (water sorptivity) tests were used to determine the durability properties. In order to determine the properties at micro structural level, mercury intrusion porosimetry test, observations and analysis on captured images of optic and scanning electron microscope were utilized.

Materials used in the civil engineering area are sometimes submitted to high level of temperature. This is particularly the case for the nuclear industry which has produced radioactive wastes characterized by the important quantity of heat that they can generate over decades. Thus, the structures designed to store these wastes may be subjected to increase in temperature up to 200 / 250 °C [87].

4.1. Materials

ASTM C125 defines the concrete and concrete aggregates. The terms used below for the materials are compatible with the mentioned standard.

4.1.1. Cement

In the production of source, natural and recycled aggregate concretes, ASTM Type I portland cement (TS EN 197-1), CEM I 42.5R (PC 42.5), was used as binder material. ASTM C150 provides for eight types of portland cement. Type I portland cement is normal, general-purpose cement suitable for all uses. The cement was supplied from Büyükçekmece Akçansa factory. The chemical analysis, physical properties and strength characteristics of the cement taken from the factory are presented in Table 4.1.

Table 4.1. Chemical composition, physical properties & strength characteristics of cement

Analysis Report	Portland Cement CEM I 42.5R
SiO ₂ (wt. percent)	20.64
Al ₂ O ₃ (wt. percent)	5.06
Fe ₂ O ₃ (wt. percent)	3.14
CaO (wt. percent)	63.98
MgO (wt. percent)	0.95
SO ₃ (wt. percent)	2.71
Na ₂ O (wt. percent)	0.31
K ₂ O (wt. percent)	0.80
Insoluble Residue (wt. percent)	0.29
Loss of Ignition (wt. percent)	1.39
Free Lime (wt. percent)	1.41
C ₃ S (wt. percent)	52.52
C ₂ S (wt. percent)	19.64
C ₃ A (wt. percent)	8.10
C ₄ AF (wt. percent)	9.56
Specific Weight (g/cm ³)	3.19
Vicat Start/Stop (hr)	2 hours:36 minutes/3h:18m
Le Chatelier (mm)	1
Specific Surface Area (Blaine) (cm ² /g)	3640
f _{cc} (2 days) (Mpa)	30.80
f _{cc} (7 days) (Mpa)	42.10
f _{cc} (28 days) (Mpa)	52.20

4.1.2. Aggregates

Natural aggregates were used in the production of source, control and recycled aggregate concretes whereas recycled aggregates were used only in the production of recycled aggregate concretes. Natural coarse aggregates were the crushed limestone no I and no II, naturally white-green (beige) color, another type of crushed limestone no I and no II, naturally blue-gray color, and natural fine aggregates were the natural sand, and crushed limestone sand (no 0). Coarse aggregates were those predominantly retained on the 4 mm sieve, and fine aggregates were those passing 4 mm sieve. Recycled aggregates were gained from crushing of source concrete in the laboratory. Crushed stones were washed for cleaning purpose of the aggregates from dusts and then spread into the laboratory to dry. After drying of crushed stones, they were sieved with 16 mm sieve. The smallest sieve opening through which the entire amount of aggregate required to pass was 16 mm, thus d_{max} , maximum aggregate size, was selected as 16 mm. Crushed sand and natural sand sieved with 4 mm sieve in order to get unpolluted sand. Coarse aggregates and fine aggregates were separated into individual size fractions by sieving. ASTM C136 test method was used for sieve analysis of fine and coarse aggregates. The physical properties and sieve analysis results of the natural aggregates are listed in Table 4.2. The specific gravity and absorption of coarse and fine aggregates were determined according to ASTM standards C127 and C128, respectively. Crushed limestone, originally beige color, was used in source concrete, crushed limestone, naturally blue-gray color, was used in natural and recycled aggregate concretes. Due to the color differences of crushed lime stones visual inspections on concrete sections could easily be done.

4.1.3. Admixture (Superplasticizer)

A chemical admixture named ADVA Flow 400, obtained from Grace Construction Products Co., was used in concrete mixtures as water reducing agent. Admixtures are generally used as an ingredient of a cementitious mixture to modify its freshly mixed, setting, or hardened properties and that is added to the batch during its mixing. Water reducing admixture either increases the slump of freshly mixed mortar or concrete without increasing the water content or maintains the slump with a reduced amount of water. The physical and chemical properties of the superplasticizer are listed in Table 4.3.

Table 4.2. Sieve analysis and physical properties of the natural aggregates

Sieve size (mm)	Sand	Crushed Sand	Crushed Limestone (beige)		Crushed Limestone (blue-gray)	
			No. I	No. II	No. I	No. II
16	100	100	100	100	100	100
8	100	100	54.6	1.1	56.5	2.1
4	100	100	1.5	0	3.7	0.1
2	99.8	66.6	0.1	0	0.2	0
1	99.3	38.2	0	0	0	0
0.50	97.2	20.3	0	0	0	0
0.25	11.8	10.3	0	0	0	0
0.125	0.6	3	0	0	0	0
Fineness modulus	1.91	3.62	6.44	6.99	6.40	6.98
Absorption (%)	2.77	2.96	1.70	1.60	2.16	2.06
Moisture content (%)	0.8	0.5	0.1	0.1	0.6	0.5
Specific gravity (dry)	2.36	2.34	2.54	2.54	2.60	2.51
Specific gravity (saturated surface dry)	2.43	2.41	2.58	2.58	2.66	2.56
Specific gravity (apparent)	2.53	2.52	2.65	2.65	2.75	2.64

Table 4.3. Physical and chemical properties of superplasticizer

Color	dark yellow-light brown
Physical state	liquid
pH value	5.8 ±1
Freezing point	-3 °C
Specific gravity	1.05 g/cm ³
Chloride content	0.0 %

4.1.4. Concrete Coloring Agent

An organic red powder named Pieri Kaolor was used in recycled aggregate concrete batches to be able to distinguish the old and new mortar. It is supplied from Grace Construction Products Co. The company informed that inclusion of the coloring agent into

concrete is allowed up to 3 percent by weight of cement and does not create any noticeable effect on the fresh and hardened concrete properties.

4.2. Production of Source Concretes

The first step of the experimental study was to produce source concretes. Source concretes were produced in order to obtain recycled aggregates. Producing recycled aggregates from crushing of source concretes was the second step and production of various natural and recycled aggregate concretes was the third step of study. Ready mixed concrete was used as source concrete in the first stage of the experiment, where laboratory produced concretes were used in the second stage as source concrete. The first stage of the experiment has been performed in order to see the effect of source concrete strength on the basic characteristics of recycled aggregate, and recycled aggregate concrete such as compressive strength and water absorption. To do that firstly recycled aggregate secondly recycled aggregate concrete were produced. Recycled aggregates were obtained from crushing of source concrete. Recycled aggregate concrete was produced by these recycled aggregates and natural aggregates plus a new matrix. In the first stage, three different source concretes with low, medium and high design strengths, namely concretes of strength class C16, C25, and C35, were used. These three source concretes were delivered from Samandra Betonsa Ready Mix Concrete Plant at the same day. Three distinct source concretes were selected in order to cover the real demolished concretes in a wide range with regard to their changing mixture compositions and qualities. The mixture proportions of the source concretes provided by Ready Mix Plant manager are presented in Table 4.4.

Laboratory produced concretes with a water cement ratio of 0.75 were utilized in the second stage of the study as source concrete. In the production of the source concretes, portland cement (CEM I 42.5R) was used as binder. The aggregates, crushed limestone no I and no II (beige), natural sand and crushed limestone sand, were used in concrete production, which satisfied the aggregate grading for TS 706. The most efficient particle size distribution to prevent segregation of the mixture recommended by TS 706 is between A16 and B16 curves, for this purpose aggregates for concrete batch were combined in the proper proportions to produce the desired grading. The grading curve of the mixture is

shown in Figure 4.1. A total of 340 dm³ concrete were produced to cast the 100x200 mm (Φ 100mm and height 200 mm) and 150x300 mm cylindrical specimens. By casting into two forms of cylinder moulds, the production time of source concretes shortened. Molds for specimens in contact with the concrete were made of steel. At the end of source concrete production the numbers of 100x200 mm and 150x300 mm cylindrical specimens were 96 and 36, respectively. Superplasticizer was included into the mixtures as solution in the mixing water in order to achieve adequate consistency. Slump was 14 cm in the source concrete, using a superplasticizer could control workability of concrete very well. The mixture proportion of the source concrete is given in Table 4.5. Superplasticizer content was 0.77 percent of the cement weight, the entrained air content of the source concrete was 4.01 percent and unit weight of the source concrete was 2184 kg/m³.

Table 4.4. Mixture proportions of the plant source concretes (kg/m³)

Batch	C16	C25	C35
Water	183	191	189
Cement (PC42.5)	195	240	300
Fly ash	50	55	65
Sand (0-3 mm)	680	619	545
Sand (0-5 mm)	252	262	199
Crushed stone # I	524	506	526
Crushed stone # II	485	487	545
Superplasticizer	1.225 (SIKA MR70)	1.475 (SIKA MR70)	4.38 (SIKA NP)

Table 4.5. Mixture proportion of laboratory produced source concretes (in kg/m³)

Mixture	Source Concrete
Water	218.0
Cement	290.7
Natural sand	163.4
Crushed limestone sand	671.2
Crushed limestone No. I (beige)	419.5
Crushed limestone No. II (beige)	419.5
Superplasticizer	2.25

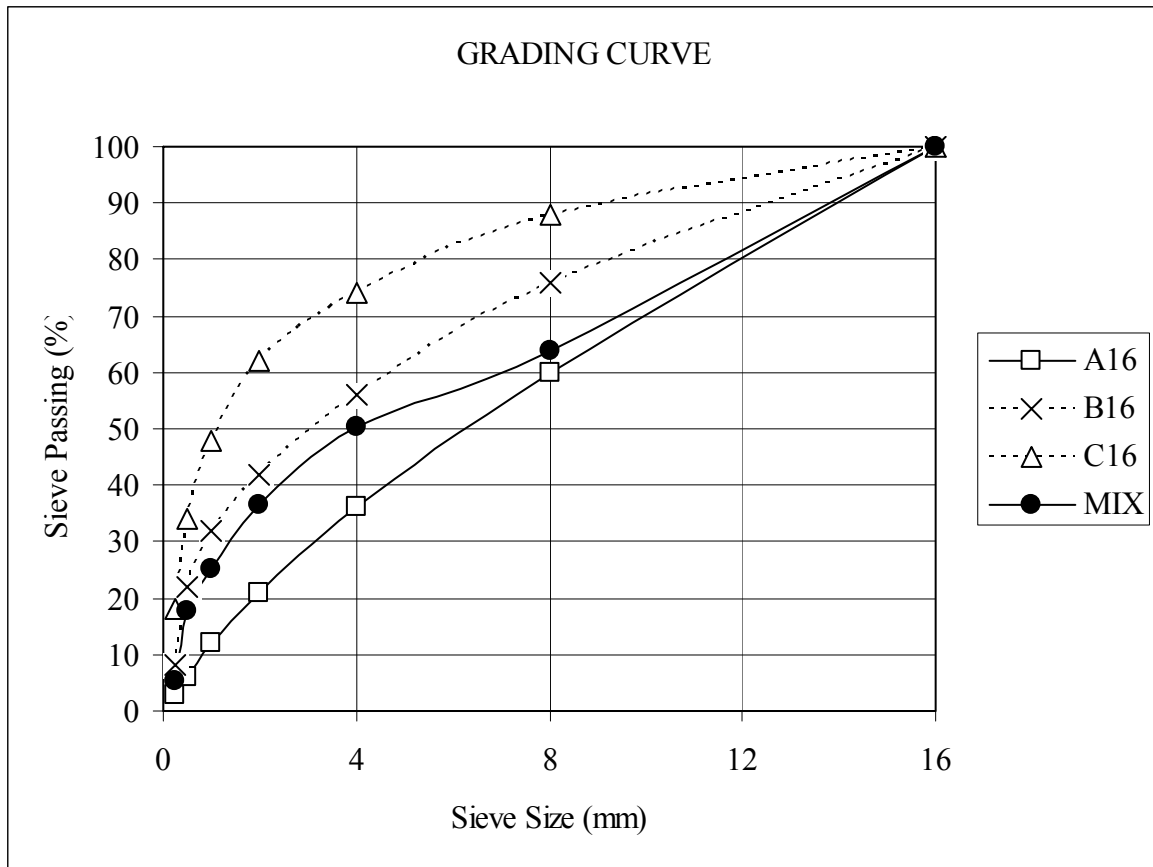


Figure 4.1. Grading curve of laboratory produced source concrete

4.3. Casting and Curing of Source Concretes

For the first stage of the study, cube moulds with dimension 150x150x150 mm were utilized for casting the C16, C25 and C35 source concretes. For each strength class a total of 10 moulds were casted at the ready mix plant before concrete transported to the site. Specimens were removed from the molds after 24 hours at the plant and then carried to the university laboratory and finally put into the curing tank and stored there for 27 days.

For the second stage, source concretes were produced at Boğaziçi University Civil Engineering Department Materials Laboratory. Concrete batches were mixed in a power-driven revolving pan mixer. Making and curing the test specimens of source concretes in the laboratory were completed in accordance with the specification of ASTM C192. Cement was sieved by 1 mm sieve to remove all lumps and then added to the mixer after addition of aggregates. After the addition of the ingredients into the mixer, the mixture was

blended for sufficient time in order to get a homogeneous mixture, and then cylinder moulds with dimension of 100x200 mm and 150x300 mm were casted. Slump was measured immediately after mixing in accordance with test method ASTM C143. The density (unit weight) and air content of fresh concretes were also determined in accordance with test method ASTM C138. The specimens were casted in three layers and compacted by the aid of vibration table, a steel trowel was used for finishing purpose. Usually sufficient vibration has been applied as soon as the surface of the concrete becomes relatively smooth and large air bubbles cease to break through the top surface. The casted specimens were covered with a plastic sheet to prevent evaporation of water from unhardened concrete for initial curing. The specimens were removed from the molds 24 hours after casting and were put into the tank filled with water for curing. The source concrete specimens were cured in 23 ± 2 °C water until 28 days age.

4.4. Production of Recycled Aggregates

After 27 days of water curing the concretes were removed from the curing tank, then all the source concretes were transported to Division of Mineral and Coal Processing Laboratory located at ITU Mine Engineer Department for crushing and recycling. In order to obtain the recycled aggregates smaller than the desired size, the original source concretes were conveyed to the jaw crusher. The details of the methodology for production of the recycled aggregates explained here is only for second stage of the experimental study. Although the methodology was same for recycling, the grading of aggregates was dissimilar for two stages. In the study, for the first stage the maximum size for the recycled aggregate was selected as 20mm and for the second stage the maximum size of the recycled aggregates was decided as 16 mm. Recycling process was finished after three stages, in which three special crushers were used. In the first stage of the recycling process, the crushed concrete particles were reduced in size less than about 90 mm. After the primary crushing, all particles were conveyed to the second jaw crusher; this second jaw crusher was smaller than the first jaw crusher. Jaw crusher crushes the material by direct impact pressing. After the secondary crushing, the aggregates were reduced in size less than about 40 mm. At the end of the second stage, the recycled aggregates were sieved by 16 mm sieve. After classifying the aggregates by sieving, the aggregates between 16 mm

and 40 mm were conveyed to the third crusher. This third crusher crushes the aggregates by rotational impact pressing. After completing these three steps of recycling process, the maximum size of the aggregates was 16 mm. At the end of the recycling process, the coarse fraction (4-8 mm and 8-16 mm) and the fine fraction (under 4 mm) were classified by sieving. The flow of producing recycled aggregates is shown in Figure 4.2. The photographs demonstrating different stages of the recycled aggregate production process are presented in Figure A.1 of Appendix A. At the second stage of the study only recycled coarse aggregates (4-16 mm) were used while in the first stage of study both recycled fine and coarse aggregates (2-20 mm) were used. The percentage of recycled aggregates for the first stage (0-2 mm) and (2-20 mm) by weight is shown in Table 4.6, and for the second stage the percentage of recycled fine aggregate (0-4 mm) and recycled coarse aggregates (4-8 mm and 8-16 mm) by weight is seen in Table 4.7.

Table 4.6. Percentage of recycled aggregates by weight (first stage)

Recycled Aggregates	0-2 mm	2-20 mm
RA16*	16.2	83.8
RA25*	16.0	84.0
RA35*	15.7	84.3

*RA16, RA25, RA35: Recycled aggregates obtained by crushing of C16, C25 and C35, respectively.

Table 4.7. Percentage of recycled fine and coarse aggregates by weight (second stage)

Fine (0-4 mm)	Coarse (4-8 mm)	Coarse (8-16 mm)
35.2%	27.5%	37.3%
Fine (0-4 mm)	Coarse (4-16 mm)	
35.2%	64.8%	

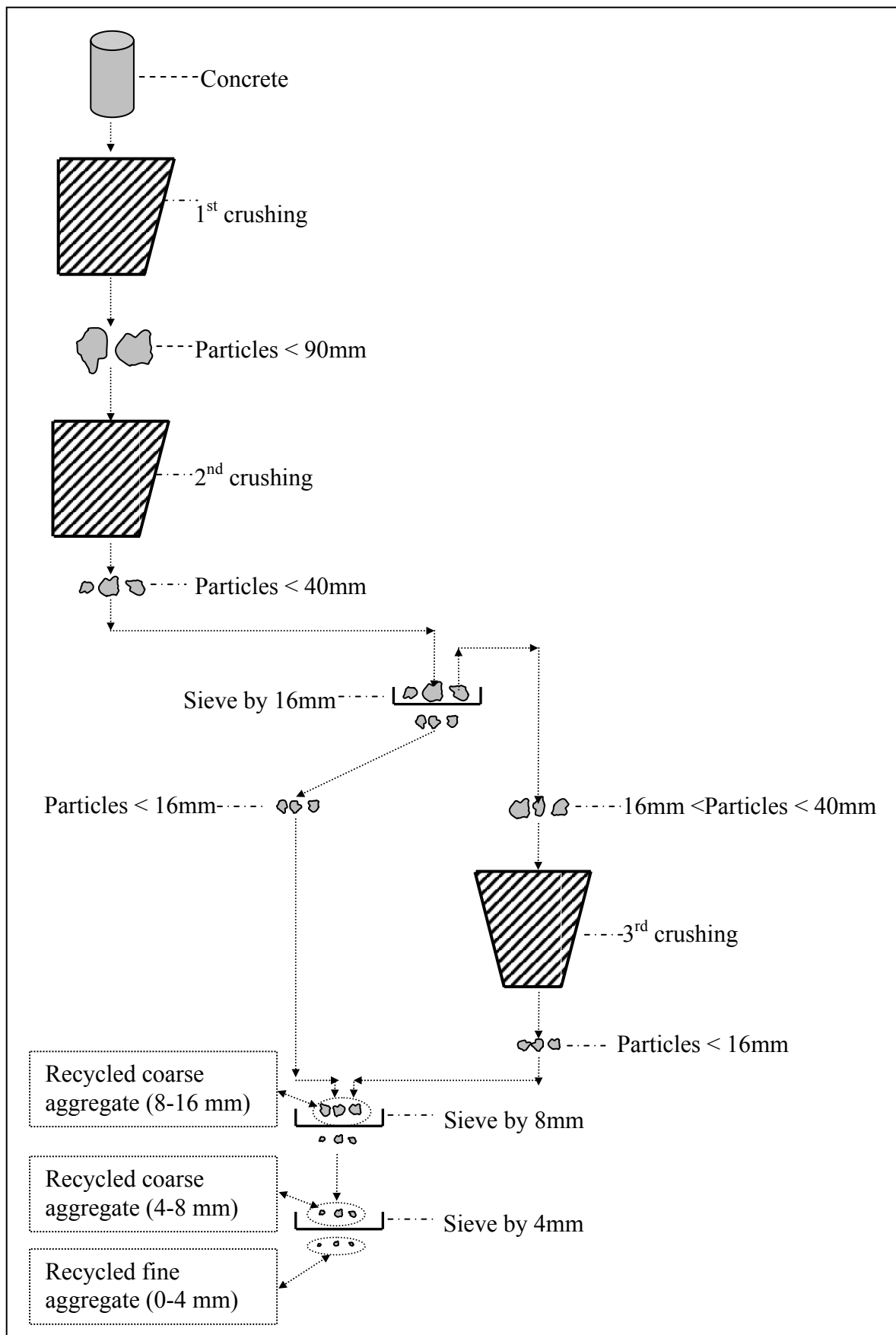


Figure 4.2. Flow of producing recycled aggregates

4.5. Properties of Recycled Aggregates

In the first stage, there were three kinds of recycled aggregates; these three recycled aggregates were labeled as RA16, RA25, and RA35. In the notation of the recycled aggregates, source concrete strength was taken as reference. RA16 was the recycled aggregate which was obtained by crushing of C16 concrete; and RA25 was obtained from C25 concrete, and lastly RA35 was obtained from C35 concrete. The saturated surface dry densities of the recycled aggregates, RA16, RA25, and RA35 were in that order 2.42, 2.45, and 2.46 g/cm³. The sieve analysis results of the recycled aggregates were given in Table 4.8.

Recycled coarse aggregates were classified into two groups at the second stage, namely recycled coarse aggregates 4-8 and 8-16 mm. The physical properties and sieve analysis results of these recycled aggregates are listed in Table 4.9.

Table 4.8. Sieve analysis of the recycled aggregates (first stage)

Sieve size (mm)	RA16	RA25	RA35
16	100	99	100
8	54	73	91
4	50	22	23
2	0	0	0
Fineness modulus	1.96	2.06	1.86

4.6. Tests and Measurements for Aggregates

4.6.1. Specific Gravity and Absorption Tests of Coarse Aggregates

The determination of specific gravity (relative density) and the absorption of natural and recycled coarse aggregates were completed in accordance with ASTM C127. Specific gravity, a dimensionless quantity, is expressed as oven dry, saturated surface dry, or as apparent relative density. Oven dry specific gravity was determined after drying the

aggregates, and saturated surface dry specific gravity and absorption were determined after soaking the aggregates in water for a period of 24 hours. The aggregates were firstly dried in the oven to constant mass at a temperature of 105 °C and then cooled in air at room temperature for 2 hours. Before immersing the aggregate in water, the mass of the aggregates were determined in oven dry condition. The test samples were immersed in water at room temperature for a period of 24 hours, and after a prescribed duration the test samples were removed from the water and wrapped between two pieces of absorbent clothes until all visible films of water removed. After the occurrence of saturated surface dry condition the mass and apparent mass of the aggregates were determined in air and in water (the volume of the sample was determined by the displacement of water method). Absorption is the increase in mass of aggregates due to water penetration into the pores of the particles, and expressed as a percentage of the dry mass as follows:

$$\text{Absorption, \%} = \frac{B-A}{A} \times 100 \quad (4.1)$$

where:

A= mass of oven dry test sample

B= mass of saturated surface dry test sample in air

Specific gravity is the ratio of density of a material to density of distilled water. Specific gravity on the basis of oven dry and saturated surface dry aggregates was calculated as follows:

$$\text{Specific Gravity, oven dry} = \frac{A}{B - C} \quad (4.2)$$

$$\text{Specific Gravity, saturated surface dry} = \frac{B}{B - C} \quad (4.3)$$

where:

A= mass of oven dry test sample

B= mass of saturated surface dry test sample in air

C= apparent mass of saturated surface dry test sample in water

Apparent specific gravity was calculated as follows:

$$\text{Apparent Specific Gravity} = \frac{A}{A - C} \quad (4.4)$$

Table 4.9. Sieve analysis and physical properties of the recycled coarse aggregates
(second stage)

Sieve size (mm)	Recycled fine aggregate	Recycled coarse aggregate (4-8 mm)	Recycled coarse aggregate (8-16 mm)
16	100	100	100
8	100	100	0
4	100	0	0
2	61	0	0
1	40.7	0	0
0.50	26.2	0	0
0.25	13.5	0	0
Fineness modulus	2.59	5.00	6.00
Absorption (%)	7.92	7.65	7.16
Moisture Content (%)	4.17	4.77	3.09
Specific gravity (dry)	2.16	2.19	2.22
Specific gravity (saturated surface dry)	2.33	2.35	2.38
Specific gravity (apparent)	2.61	2.63	2.64

4.6.2. Specific Gravity and Absorption Tests of Fine Aggregates

ASTM C128 test method was used to determine the specific gravity and the absorption of natural fine aggregates. The aggregate sample was immersed in water for 24 hours to essentially fill the pores. It was then removed from the water, the water was dried from the surface of the particles, and the mass was determined. Then, the sample was placed in a measure (graduated container) and the volume of the sample was determined by

the gravimetric method. Finally, the sample was oven dried and the mass determined again. Using the mass values obtained thus and formulas according to the test method, specific gravity and absorption were calculated as follows:

$$\text{Specific Gravity, oven dry} = \frac{A}{B + S - C} \quad (4.5)$$

$$\text{Specific Gravity, saturated surface dry} = \frac{S}{B + S - C} \quad (4.6)$$

$$\text{Apparent Specific Gravity} = \frac{A}{B + A - C} \quad (4.7)$$

$$\text{Absorption, \%} = \frac{S - A}{A} \times 100 \quad (4.8)$$

where:

A= mass of oven dry sample

B= mass of measure filled with water to calibration mark

C= mass of measure filled with sample and water to calibration mark

S= mass of saturated surface dry sample

4.6.3. Total Evaporable Moisture Content of Aggregate by Drying

The standard test method according to ASTM C 566 was used to determine the total evaporable moisture content of aggregate by drying. The mass of the aggregates were determined before they were dried and after they were dried to a constant mass in the oven at 105 °C. Then the total evaporable moisture content was calculated as follows:

$$p = (W - D) / D \quad (4.9)$$

where:

p= total evaporable moisture content of sample

W= mass of original sample

D= mass of dried sample

4.6.4. Sieve Analysis of Fine and Coarse Aggregates

The standard test method, ASTM C136, was used for sieve analysis, in other words particle size distribution, of fine and coarse aggregates. By sieving, a sample of dry aggregate of known mass was separated through a series of sieves of progressively smaller openings, and then the particle size distribution was determined. After sieving, percentages passing, total percentages retained and the fineness modulus were calculated.

4.6.5. Adhered Mortar Content of Recycled Coarse Aggregates

The determination of the adhered mortar content of recycled coarse aggregate was carried out through a method represented in the literature [25]. The method uses the hydrochloric acid dissolution to separate out the mortar and old natural aggregate from each other. In order to determine the adhered mortar content of the recycled coarse aggregate, the recycled coarse aggregates were put into hydrochloric acid (HCl) solution with a concentration of 10 percent. Hydrochloric acid dissolves the adhered mortar; it also dissolves the original coarse aggregate. That is why original coarse aggregates were also put into HCl solution for correction process.

In this study, a new method was applied to determine the adhered mortar content. This new method, thermal shock, deals with heating for separating out the adhered mortar and virgin aggregate. The aim was to determine the adhered mortar content by physical reaction other than chemical reaction. During heating, internal forces were produced due to thermal mismatch, which finally broke the adhered mortar. For this purpose, the recycled aggregates and natural coarse aggregates were heated up to 1200 °C and exposed to this temperature for two hours. Natural coarse aggregates were heated for correction procedure. The oven was shut off at the end of two hours exposure period, and the aggregates were left to cool down for 24 hours, after that they were taken out of oven. At the end of this heating and cooling process the original coarse aggregates were easily separated out from the mortar by sieving, so the mass of the aggregate and mortar obtained. Later the adhered

mortar content was calculated by dividing the mass of the adhered mortar to the total mass of adhered mortar and aggregate. The photographs demonstrating the HCl dissolution method and heating method for determination of adhered mortar content are presented in Figures B.1-B.2 of Appendix B.

An equation was also developed in the study of adhered mortar content. As illustrated in Figure 4.3, a recycled aggregate particle is composed of the original coarse aggregate and the adhered mortar.

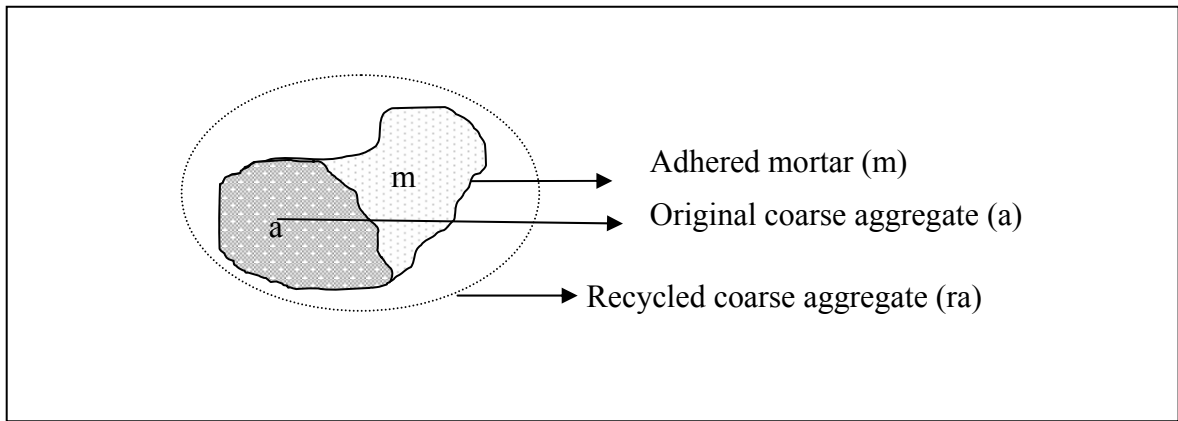


Figure 4.3. Recycled aggregate and its composition

The mass of the recycle aggregate is the sum of original coarse aggregate and adhered mortar masses. The adhered mortar content and other expressions are calculated as follows:

$$m_{ra} = m_a + m_m \quad (4.10)$$

$$r_a = \frac{m_a}{m_{ra}} \quad (4.11)$$

$$r_m = \frac{m_m}{m_{ra}} \quad (4.12)$$

Dividing the both sides of Equation 4.10 by m_{ra} yields the following equation:

$$r_a + r_m = 1 \quad (4.13)$$

where:

m_{ra} = mass of recycled aggregate

m_a = mass of original coarse aggregate

m_m = mass of adhered mortar

r_a = original coarse aggregate content, mass fraction

r_m = adhered mortar content, mass fraction

Since the recycled aggregate is the composition of two solids with masses m_a and m_m and volumes V_a (volume of original coarse aggregate) and V_m (volume of adhered mortar), the density of the recycled aggregate could be written as follows:

$$d_{ra} = \frac{m_a + m_m}{V_a + V_m} \quad (4.14)$$

Manipulation of this equation gives the following:

$$\begin{aligned} d_{ra} &= \frac{1}{\frac{V_a + V_m}{m_a + m_m}} = \frac{1}{\frac{V_a}{m_a + m_m} + \frac{V_m}{m_a + m_m}} \\ &= \frac{1}{\frac{m_a}{m_a + m_m} \left(\frac{V_a}{m_a} \right) + \frac{m_m}{m_a + m_m} \left(\frac{V_m}{m_m} \right)} = \frac{1}{\frac{r_a}{d_a} + \frac{r_m}{d_m}} \end{aligned} \quad (4.15)$$

where:

d_{ra} = density of recycled aggregate

d_a = density of original coarse aggregate

d_m = density of adhered mortar

By using the Equations 4.13 and 4.15 the following equation can be derived for adhered mortar content:

$$r_m = \frac{(d_a - d_{ra}) \cdot d_m}{(d_a - d_m) \cdot d_{ra}} \quad (4.16)$$

4.7. Design of Experiments

For the first stage of the study, four mixtures were designed in order to investigate the basic recycled aggregate concrete properties such as compressive strength and water absorption. Water/cement ratio was kept at 0.50 for all four mixtures.

At the second stage of the experimental study the effect of water/cement ratio, by weight replacement ratio of natural coarse aggregates replaced by recycled coarse aggregates, exposure temperature and age were investigated on natural and recycled aggregate concretes. Water cement ratio, w/c, was the first factor and lower limit of this factor was 0.35 and upper limit was 0.60. The second factor was the replacement ratio, the lowest limit was zero percent, and the highest limit was 100 percent. Replacement ratio zero percent refers to the concretes made with natural coarse aggregates fully and replacement ratio 100 percent refers to the concretes produced fully with recycled coarse aggregates. The third factor was the exposure temperature after which the concretes were tested at room temperature. Four levels of the third factor, 22, 105, 175 and 250 °C were utilized. The fourth factor was the age of the concretes. Concretes were tested at the ages of 28 and 90 days, and these two testing ages were the two levels of the fourth factor. The factors of the experiment were independent variables, and the variable's levels were set to investigate the effect of that factor mainly. If all of the four factors were included in the study with their all levels (2 levels of w/c ratio (0.35, 0.60), 5 levels of replacement ratio (0, 25, 50, 75, 100%), 4 levels of temperature (22, 105, 175, 250 °C) and 2 levels of age (28, 90 days)) the number of runs for full factorial test would be 80 ($2 \times 5 \times 4 \times 2$). A computer program named Design Expert was used to lower the number of runs with statistical and mathematical techniques, thus saving was made on materials and time. The statistical design of experiment was the process of planning experiments so that appropriate data would be collected, the minimum number of experiments would be performed to acquire the necessary technical information, and suitable statistical methods would be used to analyze the collected data. The software uses several methods for designing the experiments and analyzing the test results. Mixture design, factorial design and response surface design are all efficient ways of design of experiments. For this study, response surface methodology, RSM, was selected for the modeling and analysis of tests in which the responses such as compressive strength, modulus of elasticity, split tensile strength,

etc., influenced by numerous factors and the objective was to optimize the response. Moreover, response surface methodology description is a collection of statistical techniques that are useful for the modeling and analysis of problems in which one or more responses of interest are influenced by several variables and the objective is to find the relationship between the response(s) and several variables, and to optimize the response(s). By the aid of program and response surface method, rotatable 3D plots for visualization of response surfaces, and interactions of factors on 2D graphics could easily be drawn.

The D optimal criterion, one of several alphabetic optimalities, was used in response surface methodology to select design points in a way that minimizes the variance associated with the estimates of the model coefficients. D optimal designs are one of form of design provided by a computer algorithm. D optimal design minimizes the determinant of the $(X'X)^{-1}$ matrix, equivalently D optimality maximizes the determinant $(X'X)$ which is called the information matrix. The key is that D optimal design built algorithmically to provide the most accurate estimates of the model coefficients. The model is the empirical mathematical model that is fit to the data. Polynomial models are used in Design Expert software. Quadratic model is a polynomial model containing linear, two factor terms, and/or squared terms. Quadratic model is adequate for most designs. The factors of our study are four, and a response function, Y, depends on four factors with quadratic model is expressed as below equation:

$$Y = \beta_0 + \sum_{i=1}^{k=4} \beta_i x_i + \sum_{i=1}^4 \beta_{ii} x_i^2 + \sum_{i=1}^3 \sum_{j=i+1}^4 \beta_{ij} x_i x_j + \varepsilon \quad (4.17)$$

where:

Y: response function

β : coefficients

k: number of factors

x: factors

ε : error

Finding the coefficients, constructing the matrix X, finding the minimum number of design points that are needed for experiment and then constructing the design are all done

by a series of algorithm created by statisticians and mathematicians who are still under survey to get any possible small number of runs for designs. Design Expert software uses the algorithm to find the points of the design space; the algorithm used to find an approximately D optimal design is inscribed in Appendix C. In order to run the software and get the design points from response surface D optimal design, first the factors with their levels were entered into the software. Water cement ratio and replacement ratio were the numeric (continuous) factors, and the temperature and age factors were the categorical factors, which have a number of levels. For numeric factors the low and high limits and for categorical factors all levels were entered into the design. After selecting the quadratic model for the design, the software generated the candidate set of points based on the following geometric properties:

- Vertices (factorial points), the corners of the design space
- Centers of edges, midpoints between adjacent vertices
- Check points, average of centroid and vertices
- Interior points, average of centers of edges and with the overall centroid
- Overall centroid, center of design space

The software gives the total number of experiments. The total number of experiments was the sum of minimum number of design points for the “model”, which is equal to the number of coefficients of quadratic model, “lack of fit” and “replicate” points. “Lack of fit” points were added to the design, due to the subsequent explanation: The result of the experimentation is a model which adequately predicts the response within the design space. If all of the points in the design are needed to calculate the model coefficients, there will be no points left over to see how well the model fits. The model may fit the design points, but not be very good predictor at other points. Extra design points, beyond those needed for determining the model coefficients, need to be added to check the model fit. Besides “replicate” points were also added to the design in order to guarantee desired statistical accuracy. The program outputs, which show all of the runs of the experiments of our study, are seen in Table 4.10.

Table 4.10. The design layout of the experiments of the study

Run	Factor 1	Factor 2	Factor 3	Factor 4	Point Type
	A:W/C	B: Replacement Ratio (%)	C: Temperature ($^{\circ}$ C)	D: Age (days)	
1	0.35	0	22	90	Vertex
2	0.35	0	105	28	Vertex
3	0.35	0	105	28	Vertex
4	0.35	0	175	28	Vertex
5	0.35	0	250	28	Vertex
6	0.35	50	22	28	Edge center
7	0.35	50	105	90	Edge center
8	0.35	100	22	90	Vertex
9	0.35	100	105	28	Vertex
10	0.35	100	175	90	Vertex
11	0.35	100	250	90	Vertex
12	0.4125	25	250	90	Axial CB
13	0.475	0	175	90	Edge center
14	0.475	0	175	90	Edge center
15	0.475	25	22	28	Interior
16	0.475	50	250	28	Center
17	0.475	100	105	90	Edge center
18	0.475	100	175	28	Edge center
19	0.5375	25	175	28	Axial CB
20	0.5375	50	22	90	Interior
21	0.5375	75	250	90	Axial CB
22	0.60	0	22	28	Vertex
23	0.60	0	105	90	Vertex
24	0.60	0	105	90	Vertex
25	0.60	0	250	90	Vertex
26	0.60	50	105	28	Edge center
27	0.60	50	105	28	Edge center
28	0.60	100	22	28	Vertex
29	0.60	100	175	90	Vertex
30	0.60	100	175	90	Vertex
31	0.60	100	250	28	Vertex

4.8. Production of Normal and Recycled Aggregate Concretes

At the first stage of study, four mixtures with a water cement ratio of 0.50 were produced at Boğaziçi University Civil Engineering Department Materials Laboratory by using portland cement CEM I 42.5R, natural aggregates namely crushed limestone (blue-gray color) as coarse aggregate and sand as fine aggregate, recycled aggregate, superplasticizer as water reducing agent, and red powder as coloring agent. Recycled aggregates were the RA16, RA25, and RA35. Four mixtures were casted with the following aggregate combinations by weight:

- Mixture I: Natural aggregate (control mix)
- Mixture II: 50% natural aggregate & 50% recycled aggregate RA16
- Mixture III: 50% natural aggregate & 50% recycled aggregate RA25
- Mixture IV: 50% natural aggregate & 50% recycled aggregate RA35

In mixtures II-IV, 50 percent of natural aggregate was replaced by weight with recycled aggregate, which means that the replacement ratio of recycled aggregate was 50 percent. Superplasticizer was added to the concrete mixes to provide a sufficient workability. Slump was kept at 14 ± 2 cm. The mixture proportions of concretes are given in Table 4.11.

Table 4.11. Mixture proportions of concretes of first stage (in kg/m³)

Mixture No	I	II	III	IV
w/c	0.50	0.50	0.50	0.50
Water	177.1	177.1	177.1	177.1
Cement	354.2	354.2	354.2	354.2
Sand	361.4	194.7	194.7	194.7
Crushed Sand	178.8	177.0	177.0	177.0
Crushed Stone No. I	214.6	106.2	106.2	106.2
Crushed Stone No. II	1037.3	584.1	584.1	584.1
RA16	-	707.9	-	-
RA25	-	-	707.9	-
RA35	-	-	-	707.9
Red powder	5.3	5.3	5.3	5.3
Superplasticizer	4.25	4.25	4.25	4.25

31 concrete mixtures with the following water cement and replacement ratios were produced at the second stage of the study:

- Mixtures 1-5: $w/c=0.35$, replacement ratio=0% (natural coarse aggregates)
- Mixtures 6-7: $w/c=0.35$, replacement ratio=50% (both natural and recycled coarse aggregates)
- Mixtures 8-11: $w/c=0.35$, replacement ratio=100% (recycled coarse aggregates)
- Mixture 12: $w/c=0.4125$, replacement ratio=25% (both natural and recycled coarse aggregates)
- Mixtures 13-14: $w/c=0.475$, replacement ratio=0% (natural coarse aggregates)
- Mixture 15: $w/c=0.475$, replacement ratio=25% (both natural and recycled coarse aggregates)
- Mixture 16: $w/c=0.475$, replacement ratio=50% (both natural and recycled coarse aggregates)
- Mixtures 17-18: $w/c=0.475$, replacement ratio=100% (recycled coarse aggregates)
- Mixture 19: $w/c=0.5375$, replacement ratio=25% (both natural and recycled coarse aggregates)
- Mixture 20: $w/c=0.5375$, replacement ratio=50% (both natural and recycled coarse aggregates)
- Mixture 21: $w/c=0.5375$, replacement ratio=75% (both natural and recycled coarse aggregates)
- Mixtures 22-25: $w/c=0.60$, replacement ratio=0% (natural coarse aggregates)
- Mixtures 26-27: $w/c=0.60$, replacement ratio=50% (both natural and recycled coarse aggregates)
- Mixtures 28-31: $w/c=0.60$, replacement ratio=100% (recycled coarse aggregates)

A total of 31 series of concretes were produced by keeping cement paste volume about 315 dm^3 at Boğaziçi University Civil Engineering Department Materials Laboratory and tested at 28 and 90 days; 15 out of 31 series were tested at 28 days, and the rest 16 series tests were performed at 90 days. Eleven mixtures were designed with only natural coarse aggregates, 10 mixtures were designed with only recycled coarse aggregates, and 10 mixtures were designed with both natural and recycled coarse aggregates. All concrete

mixtures were produced with portland cement, CEM I 42.5R. Crushed limestone no I and no II (blue-gray), natural sand, crushed limestone sand, and recycled coarse aggregates were used in these concrete mixtures. Aggregates were combined such that they obey the replacement ratio of the run and assure TS 706 granulometry. A 25% replacement ratio means that 25% of natural coarse aggregates were replaced by recycled coarse aggregates, by weight, which was in the range of 4-8 mm, and 25% of natural coarse aggregates were replaced by recycled coarse aggregates, by weight, which was in the range of 8-16 mm. The grading curve of the mixture is shown in Figure 4.4. For each series, eight cylindrical specimens with dimensions 100x200 mm (Φ 100mm and height 200 mm) were casted for several testing purposes. In order to attain sufficient consistency, superplasticizer was incorporated to the mixtures in solution in the mixing water. Slump was kept 12 ± 2 cm in the fresh concrete. The mixture proportions of concretes and measured fresh concrete properties are given in Table 4.12 and Table 4.13. The photographs belonging to the production of concretes were displayed in Figure D.1 of Appendix D.

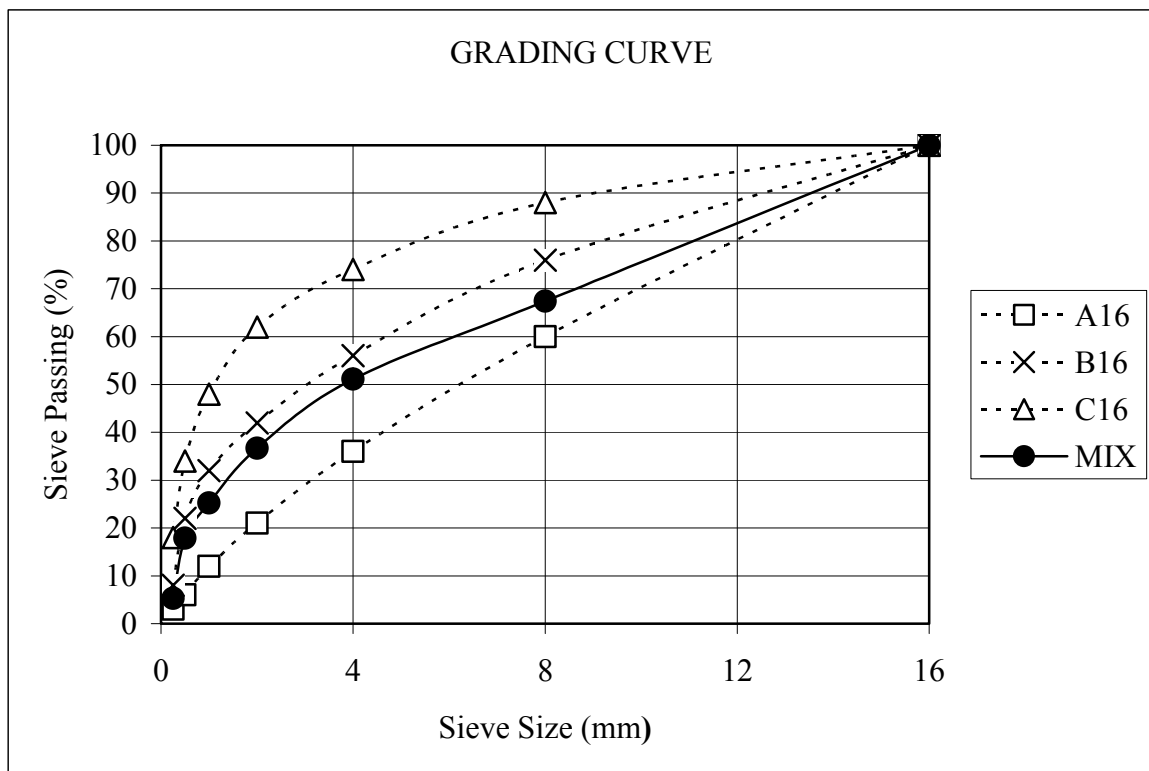


Figure 4.4. Grading curves of concrete mixtures produced with natural aggregates

Table 4.12. Mixture proportions of concretes of second stage

M ⁽¹⁾	w/c	RR ⁽²⁾ (%)	Materials (kg/m ³)									
			W ⁽³⁾	C ⁽⁴⁾	FA ⁽⁵⁾		CA ⁽⁶⁾		RCA ⁽⁷⁾		RP*	SP**
					S ⁽⁸⁾	CLS ⁽⁹⁾	No I	No II	4-8 mm	8-16 mm		
1-5	0.35	0	152.2	434.9	169.5	695.0	527.0	358.0	0.0	0.0	0.0	18.1
6-7	0.35	50	153.5	438.5	165.0	676.6	247.0	174.1	138.9	282.2	6.6	15.3
8-11	0.35	100	154.0	440.0	159.6	654.5	0.0	0.0	268.8	546.0	6.6	12.6
12	0.4125	25	172.0	417.1	167.4	686.6	367.1	258.7	68.8	139.8	6.3	8.4
13-14	0.475	0	187.5	394.7	166.2	681.7	516.9	351.2	0.0	0.0	0.0	7.7
15	0.475	25	186.4	392.5	166.0	680.6	363.9	256.5	68.2	138.6	5.9	6.3
16	0.475	50	186.8	393.3	163.3	669.5	238.6	168.2	134.2	272.6	5.9	9.1
17-18	0.475	100	189.0	397.9	159.3	653.3	0.0	0.0	261.9	532.1	6.0	7.8
19	0.5375	25	197.7	367.8	163.3	669.5	366.7	258.4	68.7	139.6	5.5	6.3
20	0.5375	50	198.8	369.9	163.1	669.0	238.5	168.1	134.1	272.4	5.5	4.9
21	0.5375	75	196.7	365.9	158.4	649.7	115.8	81.6	195.4	396.8	5.5	3.5
22-25	0.60	0	204.2	340.3	159.3	681.3	516.6	351.0	0.0	0.0	0.0	5.6
26-27	0.60	50	205.1	341.9	162.9	668.1	238.1	167.8	133.9	272.0	5.1	3.5
28-31	0.60	100	207.6	346.0	159.0	652.1	0.0	0.0	261.4	531.0	5.2	2.8

⁽¹⁾M: Mixture⁽²⁾RR: Replacement Ratio⁽³⁾W: Water⁽⁴⁾C: Cement⁽⁵⁾FA: Fine Aggregate⁽⁶⁾CA: Coarse Aggregate (blue-gray)⁽⁷⁾RCA: Recycled Coarse Aggregate⁽⁸⁾S: Sand⁽⁹⁾CLS: Crushed Limestone Sand

*RP: Red Powder

**SP: Superplasticizer

Table 4.13. Fresh concrete properties of second stage

Mixture	w/c	Replacement Ratio (%)	Slump (cm)	Density (kg/m ³)	Air content (%)
1-5	0.35	0	12.5	2369.7	4.2
6-7	0.35	50	10.0	2313.0	2.9
8-11	0.35	100	13.5	2257.2	2.8
12	0.4125	25	14.0	2305.0	2.9
13-14	0.475	0	14.0	2316.6	3.6
15	0.475	25	12.0	2275.5	3.1
16	0.475	50	13.5	2252.1	3.0
17-18	0.475	100	14.0	2218.1	2.1
19	0.5375	25	13.5	2252.1	3.3
20	0.5375	50	14.0	2233.1	2.8
21	0.5375	75	13.0	2177.8	4.0
22-25	0.60	0	12.5	2265.0	4.3
26-27	0.60	50	13.0	2205.4	3.4
28-31	0.60	100	12.5	2172.0	2.5

4.9. Casting and Curing of Normal and Recycled Aggregate Concretes

For the first and second stages of the study, cylinder moulds with dimension of 100x200 mm were casted. Before the production of mixtures, recycled aggregates were immersed in water for 24 hours; this immersing process is just a precaution for slump arrangement. After removing the recycled aggregates from water, the aggregates were wrapped between two pieces of clothes till saturated surface dry condition of the aggregates occur. Concrete batches were blended in a power-driven revolving pan mixer. Making and curing the test specimens of the concretes in the laboratory were completed in accordance with the specification of ASTM C192. Cement was sieved by 1 mm sieve to remove all lumps and then added to the mixer after addition of aggregates. After the addition of the ingredients into the mixer, the mixture was blended for three minutes followed by a three minutes rest, followed by a two minutes final mixing in order to get a homogeneous mixture, and then cylinder moulds with dimension of 100x200 mm were casted. Slump was measured immediately after mixing in accordance with test method

ASTM C143. The density (unit weight) and air content of fresh concretes were also determined in accordance with test method ASTM C138. The specimens were casted in three layers and compacted by the aid of vibration table, a steel trowel was used for finishing purpose. Usually sufficient vibration has been applied as soon as the surface of the concrete becomes relatively smooth and large air bubbles cease to break through the top surface. The casted specimens were covered with a plastic sheet to prevent evaporation of water from concrete for initial curing. The specimens were removed from the molds 24 hours after casting and put into the water tank for curing. The concrete specimens were cured in 23 ± 2 °C water until test day.

4.10. Tests and Measurements for Fresh Concrete

The properties of fresh concrete were investigated for the mixes produced with natural and recycled aggregates. The measurements of density (unit weight), and air content of fresh concrete were determined in accordance with test method ASTM C 138. The slump of hydraulic cement concrete in the laboratory was determined with the standard test method of ASTM C 143. The slump test is the simplest and most commonly used test for workability. According to the ASTM C143, the slump of concrete was determined by the following method: a sample of freshly mixed concrete was filled in a mold shaped as a cone in three layers and compacted by rodding each layer with 25 strokes, the mold was then raised in a vertical direction and concrete was allowed to subside, and then the slump was measured immediately by determining the vertical difference between the top of the cone and the displaced original center of the top surface of the specimen.

4.10.1. Density (Unit Weight) and Air Content of Concrete

The determination of the density of freshly mixed concrete and calculation of the air content were done after the application of succeeding procedure: concrete was placed in the measure in two layers and vibrated for consolidation. After consolidation, the top surface of the concrete was struck off and finished smoothly with the flat strike of plate to leave the measure just level full. Subsequent to strike off, all excess concrete was cleaned

from the exterior of the measure and the mass of the concrete and measure were determined. The density and air content of concrete were calculated by using the mass values obtained and formulas according to the test method as follows:

$$D = \frac{M_c - M_m}{V_m} \quad (4.18)$$

$$A = \frac{\left(\frac{M}{D} - V \right)}{\frac{M}{D}} \quad (4.19)$$

where:

D= density of concrete

M_c= mass of measure filled with concrete

M_m= mass of measure

V_m= volume of the measure

A= air content (percentage of voids) in concrete

M= total mass of all materials batched

V= total absolute volume of the component ingredients in the batch

4.11. Tests for Determination of Mechanical Properties of Hardened Concrete

Compressive strength, static modulus of elasticity and splitting tensile strength tests were conducted on three 100x200 mm cylinder hardened concrete specimens at 28 and 90 days. The standard test methods ASTM C 39, C 469, and C 496 were used to determine the compressive strength, static modulus of elasticity and splitting tensile strength, respectively. The specimens were tested at room temperature and after the heating and cooling cycle according to the design conditions. The specimens were heated in an electric furnace up to temperatures of 105, 175 and 250 °C. Each peak temperature was maintained for three hours to achieve the thermal steady state. At the end of three hours exposure to maximum temperature the power was turned off and the specimens were remained in the furnace until the furnace cooled down naturally to room temperature. The temperatures

were recorded at every 30 minutes; the temperature time curves for the furnace at each temperature are given in Figures 4.5-4.7. The photographs displaying the concrete specimens located into the oven prior to heating and tested for compressive strength, static modulus of elasticity and split tensile after heating are presented in Figure E.1 of Appendix E.

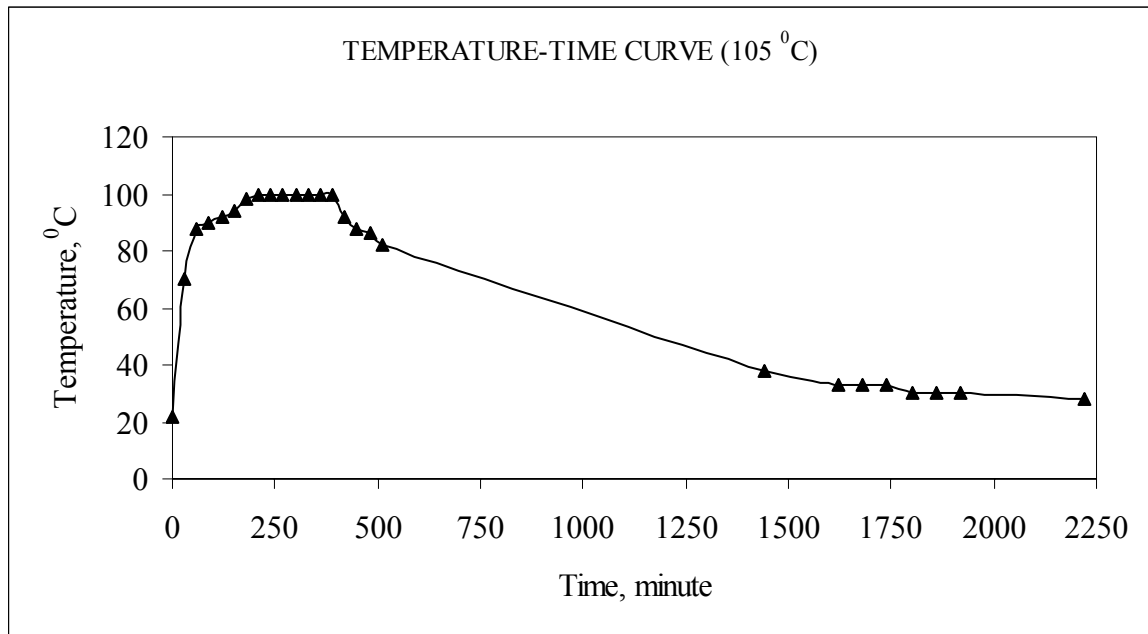


Figure 4.5. Temperature-time curve of concrete specimens exposed to 105 °C

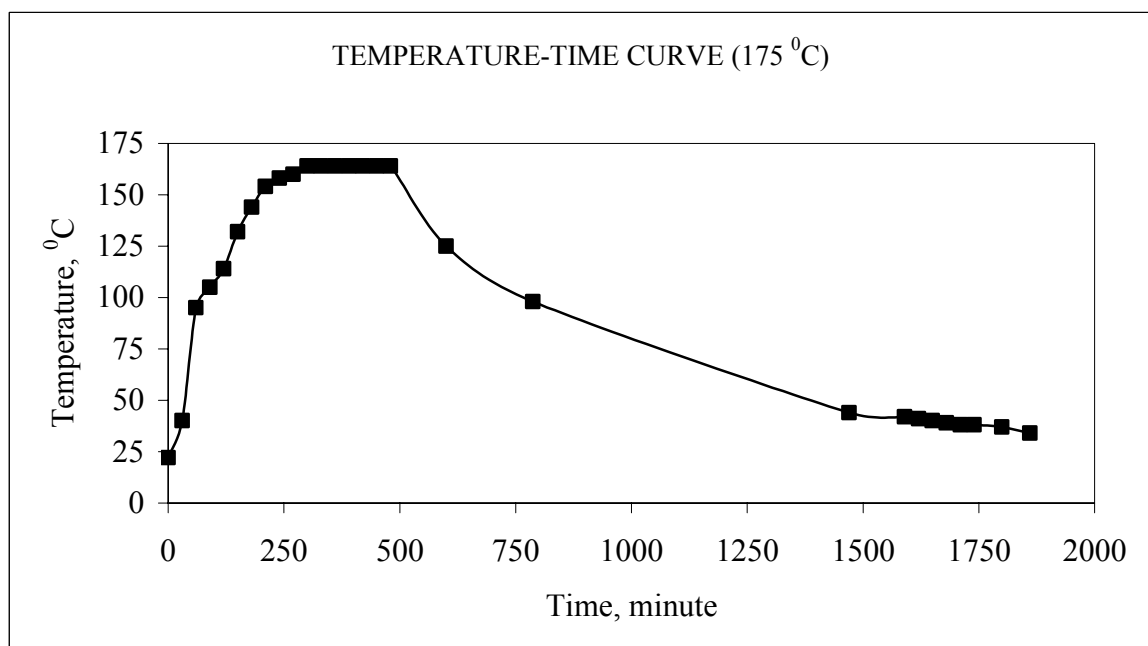


Figure 4.6. Temperature-time curve of concrete specimens exposed to 175 °C

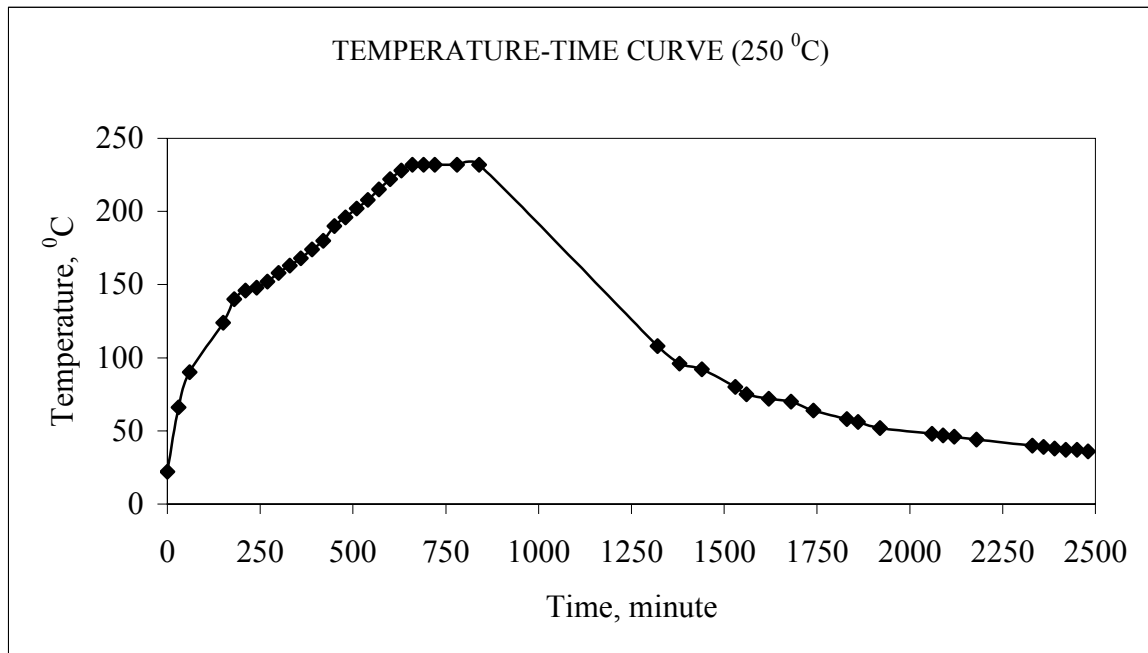


Figure 4.7. Temperature-time curve of concrete specimens exposed to 250 °C

4.11.1. Compressive Strength and Static Modulus of Elasticity

The compressive strength of cylindrical concrete specimens was calculated by dividing the maximum load attained during the test by the cross sectional area of specimen. Compressive strength test method consisted of applying a compressive axial load to 100x200 mm cylinders at a rate of 2 kN/sec until failure occurred. In order to prevent stress concentrations during the test, the specimens were capped with sulfur mortar prior to the test. Capping of cylindrical concrete specimens was performed as recommended by ASTM C 617. The specimens tested for compressive strength provided the data as well for determination chord modulus of elasticity (Young's modulus). The longitudinal deformations were measured for strain calculations by the linear variable displacement transducer (LVDT) equipments. The modulus of elasticity was calculated from the stress strain curve. The chord modulus of elasticity was the slope of the line drawn between two fixed points on the stress strain curve. The lower point was the point where the strain was 0.000050 that was established for eliminating the effect of cracks on the initial portion of the curve. The upper point was the point where the stress was equal to 40 percent of the ultimate stress. The modulus of elasticity was calculated as follows:

$$E = \frac{S_2 - S_1}{\varepsilon_2 - 0.000050} \quad (4.20)$$

where:

E= chord modulus of elasticity

S₂= stress corresponding to 40 percent of ultimate load

S₁= stress corresponding to a longitudinal strain, ε_1 , of 50 millionths

ε_2 = longitudinal strain produced by stress S₂

4.11.2. Splitting Tensile Strength

Splitting tensile strength test method was used to determine tensile strength of concrete. This test method consisted of applying a diametrically compressive force along the length of 100x200 mm cylindrical concrete specimen at a rate of 0.7 kN/sec until failure occurred by splitting. Metal strips were used so that the load was applied uniformly along the length of the cylinder. The splitting tensile strength was calculated as follows:

$$T = \frac{2P}{\pi DL} \quad (4.21)$$

where:

T= splitting tensile strength

P= maximum applied load

D= diameter of the specimen

L= length of the specimen

4.12. Tests for Determination of Durability Performance of Hardened Concrete

Water absorption, capillary absorption (sorptivity) and rapid chloride permeability tests were conducted on cylinder slices that were cut from 100x200 mm cylindrical specimens at 28 and 90 days. The heated and unheated 100x200 mm cylindrical specimens were cut into four pieces with dimensions 100x75mm, 100x50mm, 100x50mm and 100x25

mm as shown in Figure 4.8. First slice was used to determine water absorption, voids in hardened concrete and density in accordance with ASTM C 642, the second slice that was the mid portion of the 100x200 mm cylinder specimen utilized for determination of the electrical conductance of concrete to provide a rapid indication of its resistance to the penetration of chloride ions according to ASTM C 1202 and the third slice was used to measure the rate of absorption (sorptivity) of water by ASTM C 1585 test method. The photographs demonstrating the rapid chloride permeability test are presented in Figure F.1 of Appendix F.

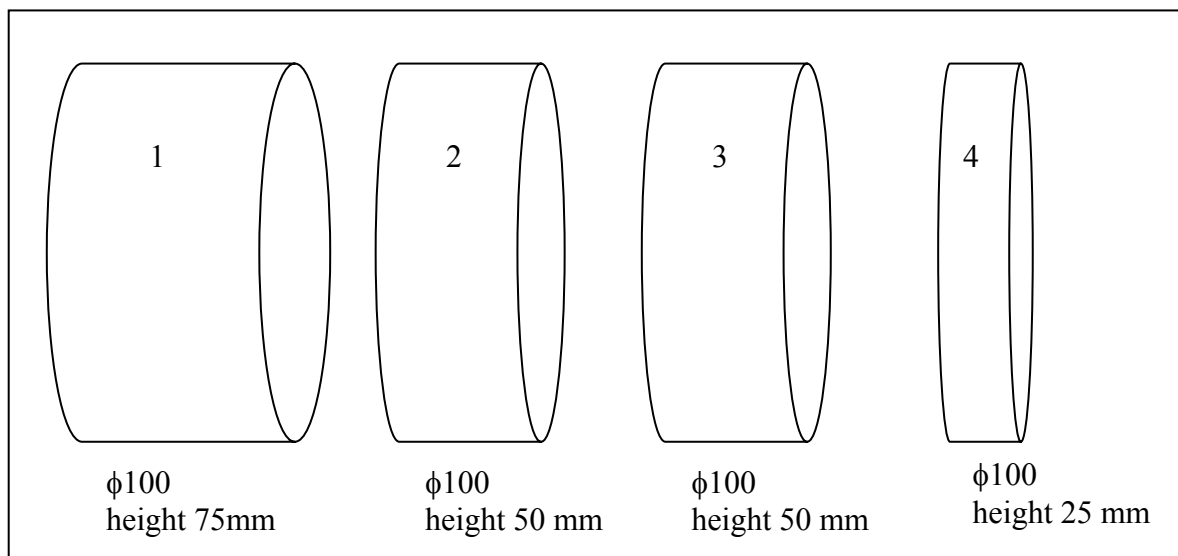


Figure 4.8. A cylindrical concrete specimen after cutting

4.12.1. Density, Absorption and Voids in Hardened Concrete

In order to determine the density, absorption, and voids in hardened concrete, the concrete slices were dried in oven for 72 hours at 105 °C. After the specimens had been dried and cooled to room temperature, the mass of the concretes were determined. Then the specimens were immersed in water for 72 hours. After immersing period, the specimens were removed from water and wrapped between two pieces of towel until all moisture removed from the surface. Then the mass of the specimens were determined for saturated surface dry condition. The apparent masses of these specimens were determined in water, by suspending them with a wire. By using the values for mass determined in accordance with the above procedure, the following calculations were made:

$$\text{Absorption after immersion \%} = \frac{B-A}{A} \times 100 \quad (4.22)$$

$$\text{Bulk density, dry} = \frac{A}{B-D} \quad (4.23)$$

$$\text{Bulk density, saturated} = \frac{B}{B-D} \quad (4.24)$$

$$\text{Apparent density} = \frac{A}{A-D} \quad (4.25)$$

$$\text{Volume of permeable pore space (voids)} = \frac{B-A}{B-D} \quad (4.26)$$

where:

A= mass of oven dry test sample in air

B= mass of surface dry test sample in air

D= apparent mass of sample in water after immersion

4.12.2. Measurement of Rate of Absorption of Water in Hydraulic Cement Concrete

The rate of absorption of water (sorptivity) by concrete was determined by measuring the increase in the mass of a specimen resulting from absorption of water as a function of time when only one surface of the specimen was exposed to water. The exposed surface of the specimen was immersed in water and water ingress of unsaturated concrete governed by capillary suction during initial contact with water. In order to determine the rate of absorption, the concrete slices were first conditioned in the temperature and humidity controlled cabinet at a temperature of 50 °C and relative humidity of 80 percent for 72 hours. After 72 hours, concrete slices were placed inside the cabinet and stored there at 22 °C and relative humidity of 60 percent for 15 days. The specimens were removed from the cabinet after 15 days, and the side surfaces of each specimen were sealed with epoxy in order to assure that the water was absorbed only

through the bottom surface, but not the side surfaces. The top surface, which was not exposed to water, was sealed by using a loosely attached plastic sheet. The plastic sheet was secured using an elastic band. The mass of the sealed specimens were measured before their immersion into water. The supports were placed at the bottom of the pan after that the pan filled with water so that the water level was 3 mm above the top of the supports. Then the test surfaces of the specimens were placed on the supports. At the time intervals of 1, 5, 10, 20, 30, and 60 minutes and 2, 3, 4, 5, 6, 25, 52, 74, 120, 146, 172 and 192 hours, the specimens were removed from the pan, the stopwatch stopped, and the mass of the specimens were measured after the excess water on the bottom surface removed with towel. Then the specimens were replaced on the supports and the stopwatch was restarted. The absorption was the change in mass divided by the cross sectional area of the test specimen and the density of water as shown in below equation; the density of water was taken as 0.001 g/mm³.

$$I = \frac{m_t}{a / d} \quad (4.27)$$

where:

I= absorption

m_t= the change in the specimen mass at the time *t*

a= the exposed area of the specimen

d= the density of the water

The gain in mass per unit area over the density of water was plotted versus square root of the elapsed time. The initial rate of absorption is defined as the slope of the line that is the best fit to I plotted against the square root of time. This slope was obtained by using least squares, linear regression analysis of the plot I versus time^{1/2}. For the regression analysis, all the points were used from one minute to six hours. The secondary rate of absorption was the slope of the line that was the best fit to I plotted against the square root of time using all the points from one day to seven days. Least square linear regression analysis was used to determine the slope. A correlation coefficient of less than 0.98 means that data between one minute and six hours do not follow a linear relationship, in that case the rate of absorption could not be determined. An example for absorption (I) versus

time^{1/2} is seen in Figure 4.9. A schematic of the capillary absorption (sorptivity) test setup is illustrated in Figure 4.10.

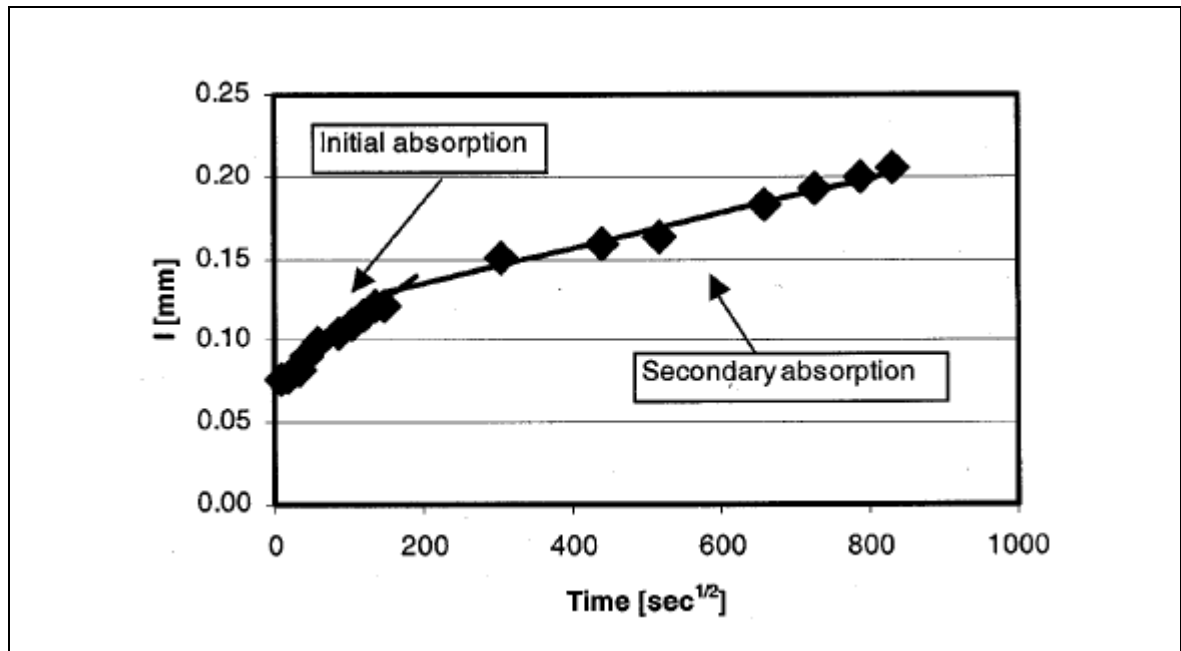


Figure 4.9. Absorption (I) versus time^{1/2}

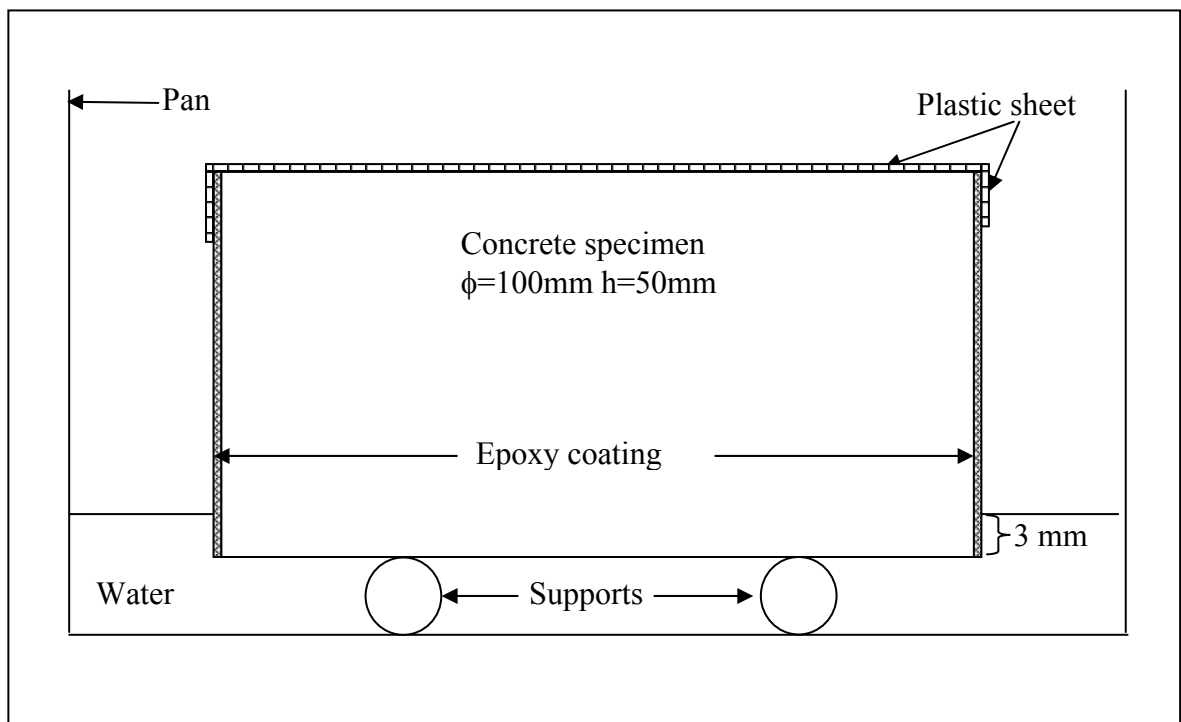


Figure 4.10. Capillary absorption test setup

4.12.3. Chloride Ion Permeability

The electrical conductance of concrete to provide a rapid indication of its resistance to the penetration of chloride ions was determined by rapid chloride permeability test. This test method consisted of monitoring the amount of electrical current passed through 50 mm thick concrete slices of 100 mm diameter cylinders during a six hours period. A potential difference of 60 Volts direct current (dc) was maintained across the ends of the specimen, one of which was immersed in a sodium chloride solution, the other in a sodium hydroxide solution. The setup of the rapid chloride permeability test is illustrated in Figure 4.11. Two specimens for each concrete batch were conditioned before testing. The side surfaces of the specimens were coated with rapid drying epoxy by brush and placed onto the suitable support to dry. After drying of specimens, they were placed in glass beaker filled with drinking water such that both end faces of specimens exposed to water. Then the beaker cap closed and sealed by oil. The vacuum pump was turned on and the specimens were vacuumed for three hours. After three hours of vacuum procedure, the specimens were saturated under water in the beaker for 18 hours. After the conditioning procedure described above, the specimens were removed from water and transferred to the test cells. The specimens were fixed into the cells and covered with plastic sheet to restrict moisture movement. The silicon as a sealant was applied around specimen-cell boundary. The two halves of the test cell were clamped together and then specimen-cell boundary sealed again for any water loss. One side of the cell was filled with three percent NaCl solution and connected to the negative terminal of the power supply. The other side of the cell was filled with 0.3 N NaOH and connected to the positive terminal of the power supply. After the electrical connections were made to voltage application and data readout apparatus, the power supply was turned on and set to 60 Volts. The initial voltage and voltages during the periods of six hours were recorded at every one minute time intervals by data acquisition system. Six hours later, the test was terminated and specimens were removed. The area beneath the current versus time curve gives the total coulomb value of charge passed during six hours test period. The total charge passed was a measure of the electrical conductance of the concrete during the period of the test. These obtained numerical results were compared with the qualitative values listed in Table 4.14 for evaluation of test results. ASTM C1202 classifies the chloride ion permeability of concretes into five groups as high, moderate, low, very low and negligible based on charged passed.

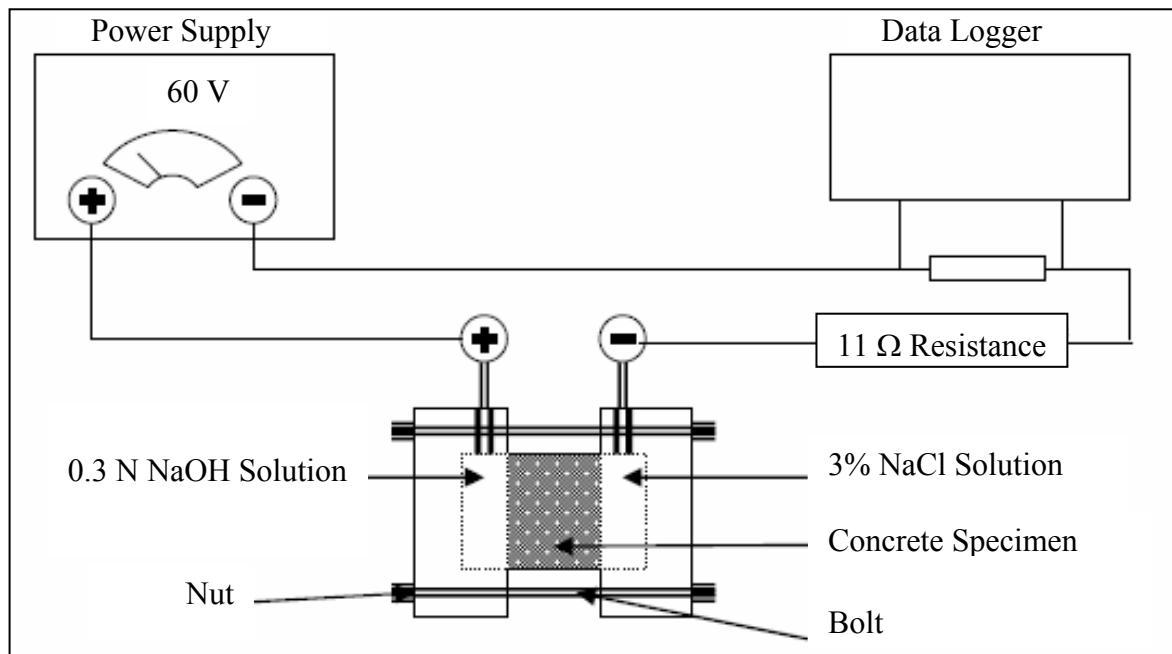


Figure 4.11. Rapid chloride permeability test setup

Table 4.14. Chloride ion penetrability based on charge passed

Charge passed (coulombs)	Chloride ion penetrability
>4000	high
2000-4000	moderate
1000-2000	low
100-1000	very low
<100	negligible

4.13. Microstructural Studies of Hardened Concrete

Until recently, the knowledge of nature and behaviour of concrete was restricted to a few properties. In the past few decades, concrete technologists have focused their attention on the internal structure of concrete [115]. The internal structure of concrete plays a decisive role in influencing the mechanical and durability properties of concrete. Most of the properties of the hardened concrete are related to the quantity and the characteristics of pores in concrete [116]. The interfacial transition zone is usually regarded as the weakest region in concrete influencing both mechanical properties and durability [25]. Interfaces occur in cement based materials in many forms: within the hydration medium, between

cement and pozzolanic admixtures, between cement and reinforcing steel or fibers, and between cement and aggregates [105].

Pores present in cement solid matrix or in concrete can be distinguished into air voids due to entrained air and air mixed with materials and entrapped during the mixing and, pores due to mixed water. Pores due to mixed water can be divided into gel pores and capillary pores, which, respectively, increase and decrease according to the hydration process of cement, resulting finally in a reduction of the total volume of pore at the end of the process. Especially, since the structure of the pores in cement matrix depends significantly on the types of cement and admixtures, the maturity that is the degree of hydration of concrete and the composition of concrete, producing dense cement matrix will contribute effectively on improving the strength as well as the durability of concrete [117]. Same study [117] defines micro-pores to be smaller than $0.01\ \mu\text{m}$ (micrometer), meso-pores ranging from 0.01 to $0.05\ \mu\text{m}$, and macro-pores as pores with sizes between 0.05 and $10\ \mu\text{m}$ as shown in Figure 4.12. Classification is also made for the pores in the cement matrix as gel pore, inter-gel particle pore, capillary pore and entrained air respectively for diameters smaller than $0.003\ \mu\text{m}$, between 0.003 and $0.2\ \mu\text{m}$, between 0.003 and $10\ \mu\text{m}$, and larger than $10\ \mu\text{m}$, respectively.

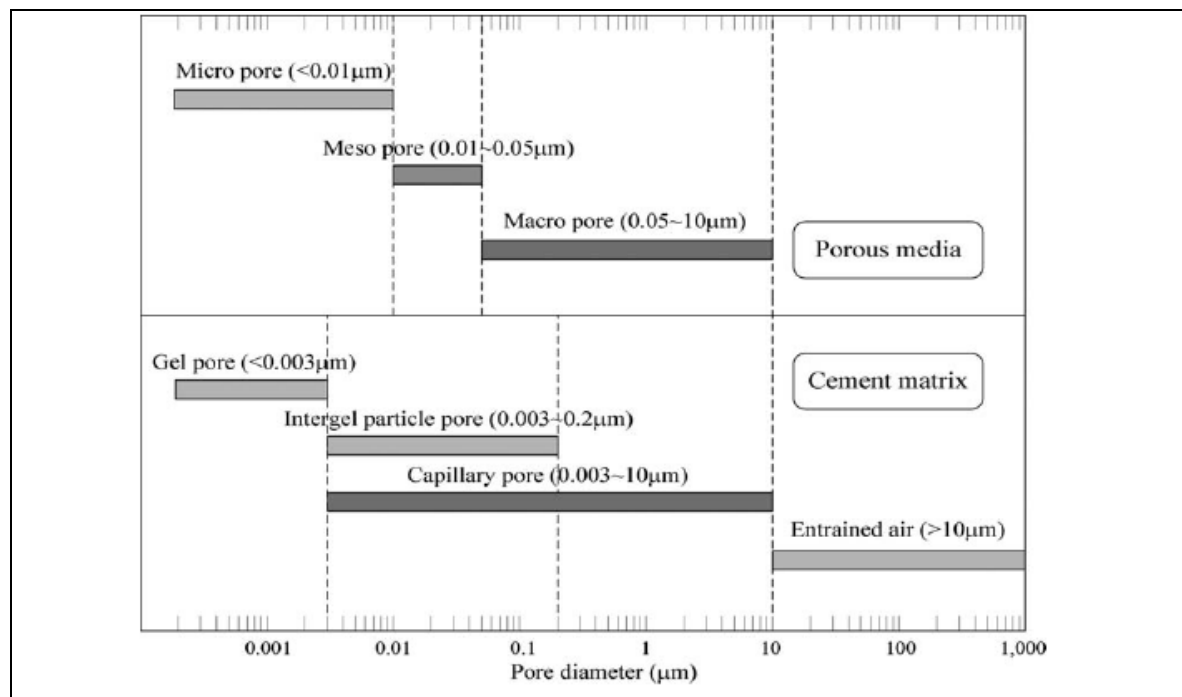


Figure 4.12. Classification of pore size [117]

The alterations in pore structure of natural and recycled aggregate concretes subjected to elevated temperatures are also of great concern. Due to the importance of microstructure, studies were performed on hardened concrete in order to examine the pore structure and interfacial transition zone (ITZ). Porosity was measured using mercury intrusion porosimetry (MIP) and ITZ was studied using optic microscope and scanning electron microscope (SEM).

4.13.1. Determination of Porosity by Mercury Intrusion Porosimetry

Mercury intrusion porosimetry (MIP) test was conducted to identify the change in pore structure of natural and recycled aggregate concrete subjected to temperatures of 22 °C, 105 °C, 175 °C and 250 °C. MIP is widely adopted for the study of porosity and pore size distribution of cement based composites such as cement paste, mortar and concrete [118].

Kumar and Bhattacharjee [115] studied some factors affecting the results in the use of MIP method in concrete research. According to their review, in the past, pore structure study of concrete has been carried out using porosimetry test performed on either cement paste or mortar extracted from concrete. The pore size distribution of cement paste measured by MIP depends on the method of preparation of the paste. Pore size distribution of plain paste prepared independently is different than that of a paste separated from fresh mortar of concrete, even though the pastes were identical otherwise. Cement paste present in mortar and concrete is more porous than the plain paste prepared independently. The mortar adhered with coarse aggregate is more porous than the mortar devoid of coarse aggregate taken from the same concrete specimen. This is due to the fact that the transition zone present between the boundary of coarse aggregate and the bulk mortar matrix is also included in the sample containing the coarse aggregate. This transition zone is more porous than the bulk mortar present in the interior concrete. Thus, it was concluded that mercury porosimetry test performed on concrete should be conducted using concrete sample, i.e., mortar adhered with coarse aggregate, rather than plain mortar devoid of coarse aggregate extracted from the parent concrete.

In the study, MIP test was performed on small concrete pieces extracted by saw machine from the interior of cylinder specimens cast with mixes 1-31 ensuring the presence of natural and/or recycled coarse aggregate. The extracted small concrete pieces was then reduced to size about $\phi = 8 \text{ mm} \times h = 10 \text{ mm}$ by grinder machine. The MIP tests were done at Istanbul Technical University Metallurgical and Materials Engineering Department Laboratory on a Quantachrome Poremaster, which subjected the samples to a maximum pressure of up to 227.5 MPa (33000 psi). The photograph of Quantachrome Poremaster is shown in Figure 4.13. The samples were dried in an oven at a temperature of 105°C before testing in order to empty the pores of any existing fluid. The samples were then transferred to a chamber, which were then evacuated and mercury was introduced to surround the specimen. Then, pressure was applied to the mercury in progressive increments, and the intrusion of mercury at each step was monitored. The relationship between cylindrical pore diameter and the pressure was evaluated by Washburn equation [119] as follows:

$$d = \frac{-4\gamma \cos \theta}{P} \quad (4.28)$$

where:

d = diameter of cylinder being intruded

γ = surface tension of mercury

θ = contact angle of mercury on the solid

P = applied pressure

The contact angle and surface tension values of mercury were adopted from available literature [118, 120-122] to be 117° and 0.484 N/m. According to the Equation 4.28 the measurable pore size range is from 6.16 nm (nanometer), when pressure is 227.5 MPa (33000 psi), to 136.1 μm , when pressure is 0.0103 MPa (1.5 psi).



Figure 4.13. Quantachrome Poremaster equipment for MIP

4.13.2. Study of the Interfacial Transition Zone (ITZ) Using Optic Microscope and Scanning Electron Microscope

Interfacial transition zone (ITZ) is shown to significantly influence the properties of concrete. ITZ is generally the weakest link of the chain and the strength limiting phase in concrete. It is because of the presence of ITZ that concrete fails at a considerably lower stress level than the strength of either of the two components (aggregate and mortar matrix). In concrete, the ITZ serves as a bridge between the two components: the mortar matrix and the coarse aggregate particles. Even when the individual components are of high stiffness, the stiffness of the concrete may be low because of the broken bridges (i.e., voids and microcracks in the ITZ), which do not permit stress transfer. This can be seen in Figure 4.14, which shows the fracture surface of a concrete specimen that failed under compression. It can be clearly seen that the failure cracks propagate to the ITZ around aggregate surface [61].

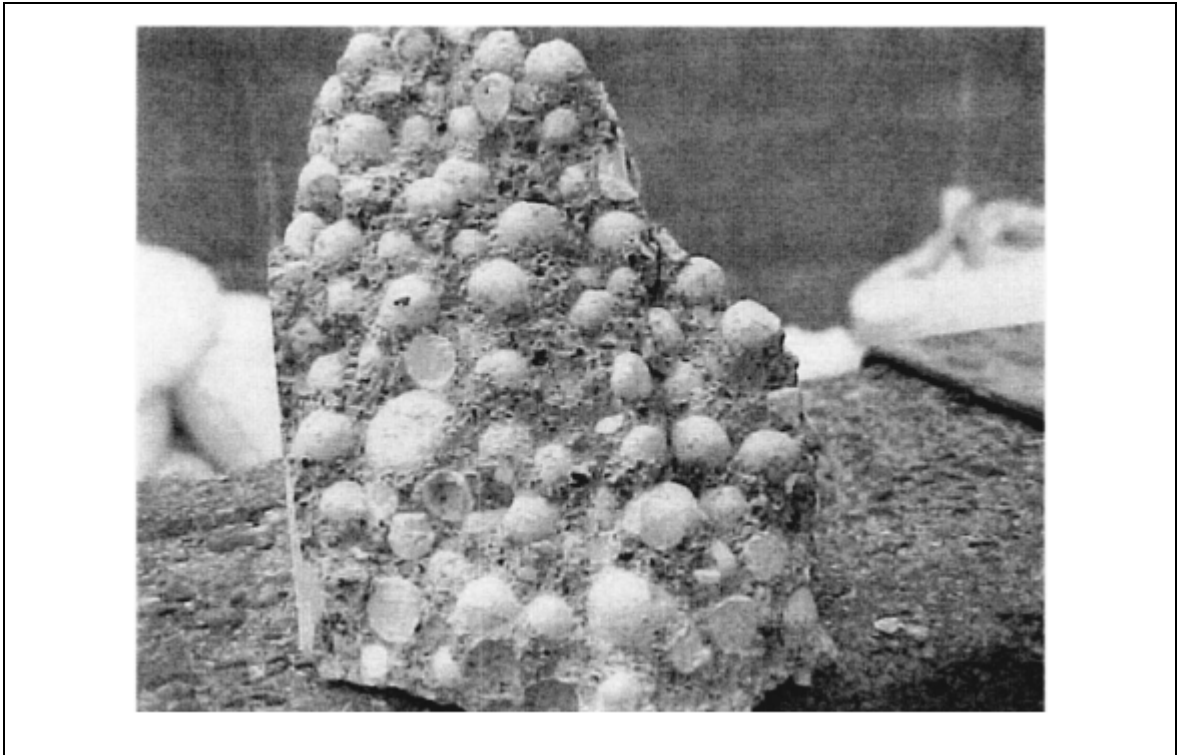


Figure 4.14. Fracture surface of concrete specimen

Since ITZ affect the strength and durability of concrete, it should be considered when natural and recycled aggregate concretes are evaluated. The width of the interfacial transition zone is about 50 μm [84]. Principally, in recycled aggregate concrete, there are different kinds of ITZs, not like in natural aggregate concrete. Natural aggregate concrete has only one kind of ITZ between natural aggregate and mortar matrix, whereas in recycled aggregate concrete there are four interfaces (if $0 < \text{replacement ratio} < 100$ percent), as seen in Figure 4.15. The interfaces in the Figure 4.15 were numbered according to the following representation:

- Interface 1: interface between natural aggregate and new mortar
- Interface 2: interface between adhered mortar and original coarse aggregate
- Interface 3: interface between adhered mortar and new mortar
- Interface 4: interface between original coarse aggregate and new mortar

Optical microscope and scanning electron microscope evaluations were used effectively to examine the ITZ in the study. Optical microscope, shown in Figure 4.16, offers a wide field of view which in turn gives a quick understanding of the concrete

microstructure, besides it is an economic examination. The microscopic studies on concrete pieces of mixes 1-31 were carried out at Boğaziçi University Soil Mechanics Laboratory. Software, DT Acquire by Data Translation Inc., was used to capture the images when microscope set at a magnification of 75 times and 150 times. The locations of the images were randomly selected; only specific attention was given to collect the images at all existing interfacial transition zones.

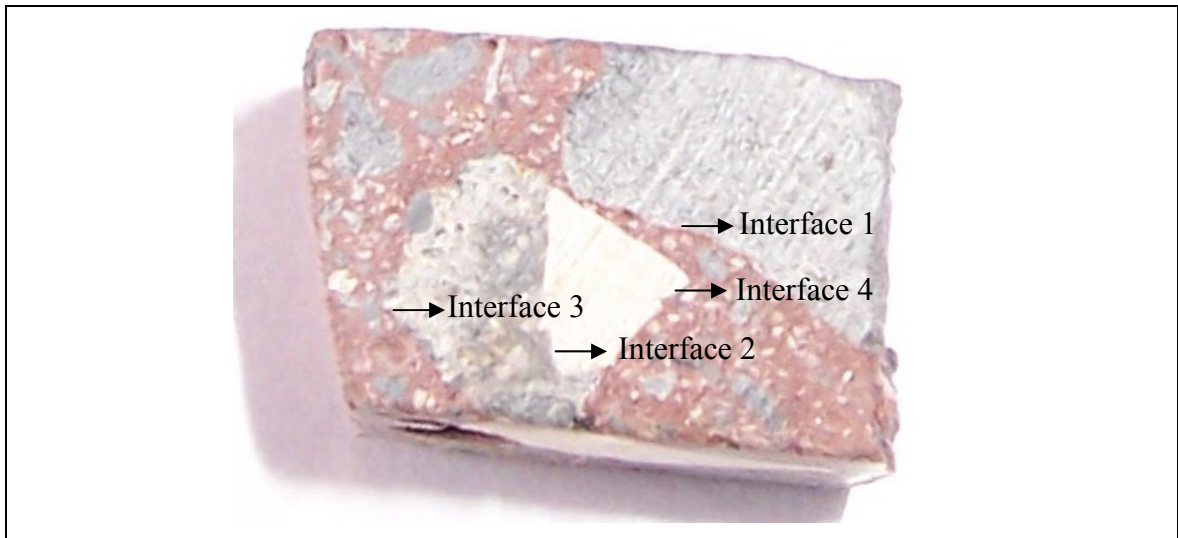


Figure 4.15. Interfaces in recycled aggregate concrete (sample from mix 6)

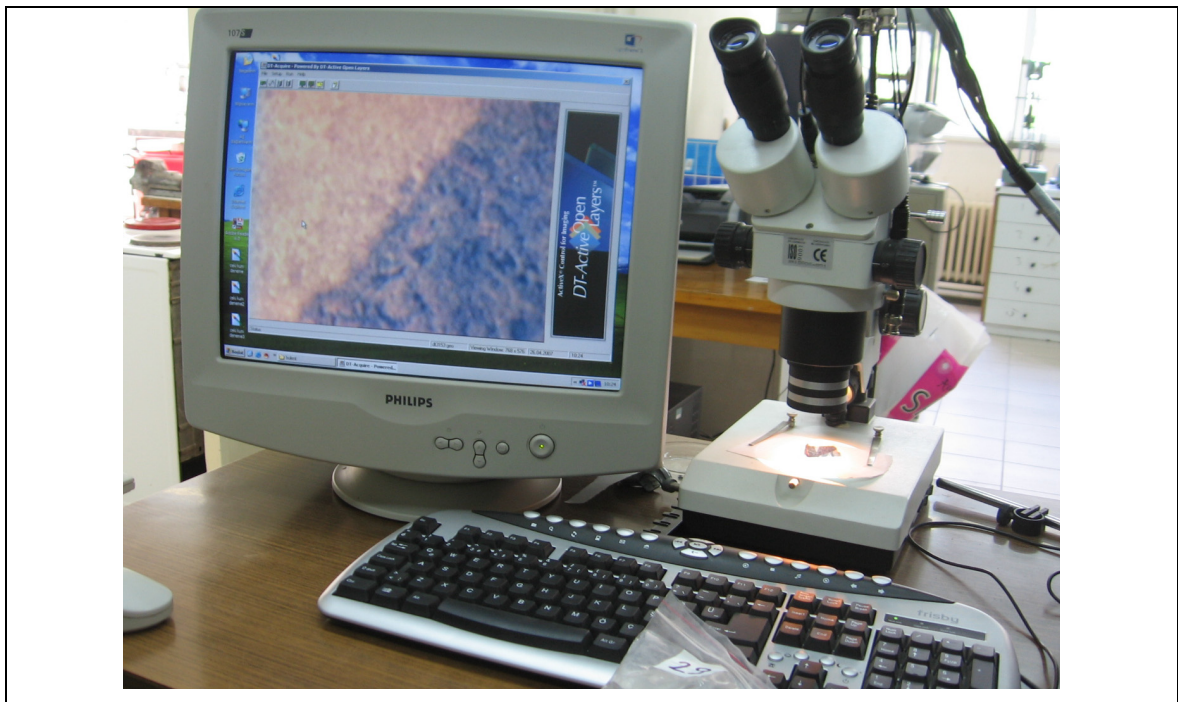


Figure 4.16. Optical microscope setup

Scanning electron microscope (SEM) was employed also to analyze the interfacial transition zone of the samples from mixes 1-31. Imaging the natural aggregate concrete pieces were performed at Turkish Cement Manufacturers' Association located at Ankara on SEM equipment, whereas imaging the recycled aggregate concrete pieces were completed at Istanbul Technical University Metallurgical and Materials Engineering Department Electron Microscopy Laboratory on Field Emission Scanning Electron Microscope (FESEM) provided by Oxford Instruments Incax-sight. The model of the FESEM instrument was JEOL JSM – 7000F. The photograph of FESEM is shown in Figure 4.17. The concrete samples were coated with a very thin layer of gold in order to make the surface electrically conductive. After the samples had been properly prepared, they were placed in the electron microscope viewing chamber, and signals were collected. An image resolution of 1024 pixels x 768 pixels was used at magnifications of about 500 times. The selected magnification was the one suggested in before study [105] to analyze the ITZ. The greater the magnification and resolution, the smaller the pore size can be detected.

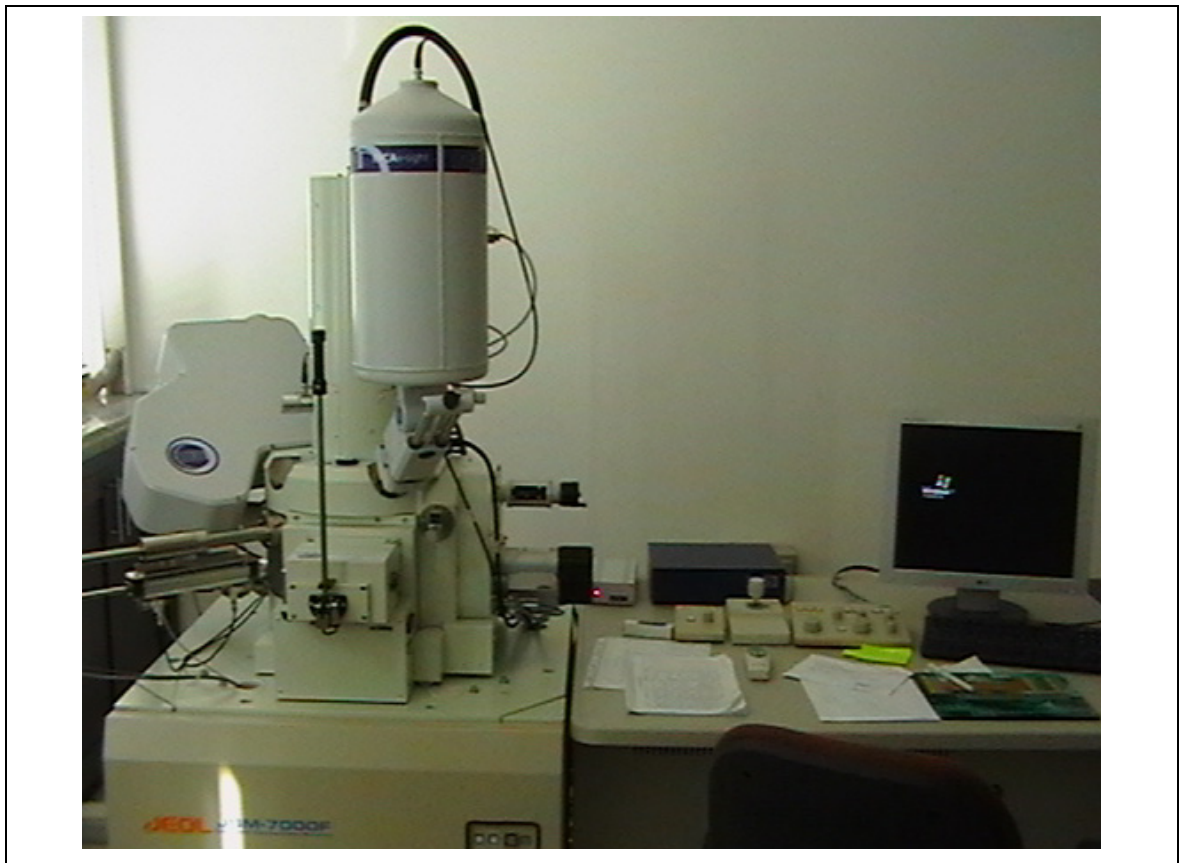


Figure 4.17. Field emission scanning electron microscope setup

4.13.3. Determination of Porosity by Image Analysis

Image analysis consists of nondestructive analysis of surface and/or inner characteristics of structures and materials using various image capturing techniques. Image analysis is used in applications including engineering and science as well as medicine and various other fields to evaluate and characterize the properties and condition of materials. Common methods used for obtaining images of materials include optical and thermal techniques. In microscopy analysis (optical analysis), images can be obtained by optical light microscope, scanning electron microscope (SEM), transmission electron microscope (TEM), confocal microscope, and atomic force microscope (AFM). Digital video recorders and cameras are also used to capture images of materials. Magnifications of approximately 1/1 to 1/1.000.000 can be achieved using optical methods. Optical methods are generally used to investigate surface and inner characteristics of materials such as texturing, molecular structures, defects, porosity of materials, etc. [123].

In this study, electron microscope pictures were used in image analysis to evaluate the concrete porosity. Free software, Image J 1.37v (available at <http://rsb.info.nih.gov/ij/>) developed by USA National Institutes of Health was used in thresholding the image and then to evaluate the pore structure. The images were converted to gray scale 8-bit type, and then black & white threshold tool adjusted with upper and lower threshold values so that the pores were assigned one color while the solid part was assigned another color. After thresholding done by the aid of analyze tool in the software, measurements for the porosity were computed.

5. TEST RESULTS AND EVALUATION

Tests and measurements for aggregates and fresh concrete were explained extensively in chapter four where the results of the tests and measurements were also included. In this chapter fresh concrete properties of natural and recycled aggregate concretes of second stage were evaluated. The test results and analysis of the test results of hardened concrete properties of source concretes and natural and recycled aggregate concretes, adhered mortar content of recycled aggregates, and micro structural studies were indicated also in this chapter.

Design Expert software was used for analysis of the test results. The test results were entered into the program and by the help of analysis tools the test results were analyzed. In order to analyze the results (responses), the appropriate model was chosen to fit the data. The model is the empirical mathematical model that is fit to data; polynomial models are used in the software. Design Expert software uses the statistical analysis to choose the best model by regression calculations among the linear, 2FI (two factor interaction), quadratic and cubic models. The program calculates the effects for all models. It produces statistics such as F-values, lack of fit and R-squared values for comparing the models. Based on the statistical report, the program gives reports to help user select the most suitable model for a given result (response). The F distribution is a probability distribution used to compare variances by examining their ratio. Low standard deviation and R-squared near one are the best. The program also makes analysis of variance (ANOVA) of polynomial model coefficients and case statistics for analysis of residuals. ANOVA is the statistical method that is used for making simultaneous comparisons between two or more means, which yields values that can be tested to determine whether a significant difference exists between variables. ANOVA outputs are helpful to comment on the individual factors. The definition of Prob>F is the probability of seeing the observed F value if the null hypothesis is true (there is no factor effect), the probability equals the proportion of the area under the curve of the F distribution that lies beyond the observed F value; shortly this means that if the Prob>F value is very small (less than 0.05) then the terms in the model have a significant effect on the response. The program also allows the user to inspect various

diagnostic plots to statistically validate the model. Among the model diagnostic plots “Actual versus Predicted” helps the user detect a value, or group of values, that are not easily predicted by the model, it is a graph of the actual response values versus the predicted response values. If all checks of models are good enough, then model graphs are generated for interpretation.

5.1. Mechanical Properties of Source Concretes

Compressive strength test results of the first stage source concrete specimens casted in a ready mixed plant at 28 days are presented in Table 5.1. The compressive strengths of the source concretes were obtained from three 150x150x150 mm cube specimens. C16 refers to the concrete with compressive strength of 16 MPa when it is tested with 150x300 mm cylindrical specimens at 28 days; the equivalent cube strength of C16, C25, and C35 are 20 MPa, 30 MPa, and 45 MPa, respectively. According to the test results listed in Table 5.1, the compressive strengths of the C16, C25, and C35 source concretes were in that order 23.5, 35.9, and 50.3 MPa.

Table 5.1. Compressive strength of first stage source concretes at 28 days

Batch	f_c^* , 28days (MPa) 1 st sample	f_c , 28days (MPa) 2 nd sample	f_c , 28days (MPa) 3 rd sample	f_c , 28days (MPa) average
C16/20	22.7	23.0	24.9	23.5
C25/30	36.1	35.5	35.9	35.9
C35/45	49.7	49.0	52.0	50.3

* f_c : compressive strength

In the second stage, compressive strength tests were carried out on hardened concrete specimens at 28 days. Compressive strength was determined on three 100x200 mm and three 150x300mm cylinder specimens. In addition, three 100x200 mm cylinder specimens were tested for splitting tensile strength at 28 days, at room temperature. The compressive and splitting tensile strength test results of the second stage source concretes casted in laboratory with a water/cement ratio of 0.75 (C20) are given in Tables 5.2 and 5.3.

Table 5.2. Compressive strength of second stage source concretes at 28 days

Source Concrete	$f_{c, 28\text{days}}$ (MPa) 1 st sample	$f_{c, 28\text{days}}$ (MPa) 2 nd sample	$f_{c, 28\text{days}}$ (MPa) 3 rd sample	$f_{c, 28\text{days}}$ (MPa) average
100x200 mm specimens	21.9	22.7	21.6	22.1
150x300 mm specimens	24.2	24.7	24.2	24.4

Table 5.3. Splitting tensile strength of second stage source concretes at 28 days

Source Concrete	$f_t^*, 28\text{days}$ (MPa) 1 st sample	$f_t, 28\text{days}$ (MPa) 2 nd sample	$f_t, 28\text{days}$ (MPa) 3 rd sample	$f_t, 28\text{days}$ (MPa) average
100x200 mm specimens	2.17	2.16	1.91	2.08

* f_t : splitting tensile strength

5.2. Fresh Concrete Properties of Natural and Recycled Aggregate Concretes

The mixture proportions of natural and recycled aggregate concretes of mixes 1-31 were listed in previous chapter in Table 4.12. As it is seen from that table at the same w/c ratio, the free water content of natural and recycled aggregate concretes were about equivalent, also at the same w/c ratio, the cement content was kept same for natural and recycled aggregate concretes to maintain the equal w/c ratio. The free water content of recycled aggregates was increased due to the high water absorption capacity of recycled aggregates by some researchers [16, 33]. Instead of increasing the free water content due to the high water absorption of recycled aggregates, they were presoaked in our study before mixing as suggested by others [25-26]. As seen from Table 4.12, superplasticizer content was lower in recycled aggregate concretes than natural aggregate concrete. Although recycled aggregates were employed in saturated surface dry conditions, absorbed water and water film on rough surfaces of recycled aggregates increased workability. Poon *et al.* [32] reported that the high water content inside the recycled aggregate particles may result in bleeding. Certainly, superplasticizer content was decreased in parallel to an increase in w/c

ratio. Moreover, fresh unit weight decreased in natural and recycled aggregate concretes with an increase in w/c ratio as given in Table 4.13. Fresh unit weight measurements showed also that at the same w/c ratio, the density of recycled aggregate concretes was lower than natural aggregate concrete. The concrete containing 50 percent recycled aggregate as the coarse fraction reduced the fresh unit weight by 2.5 percent compared to the natural aggregate concrete, when w/c ratio was 0.35. For the same w/c ratio, the concrete incorporating 100 percent replacement ratio (fully) recycled coarse aggregates decreased the fresh unit weight by 5 percent compared to the natural aggregate concrete. When the replacement ratio was 50 and 100 percent, the fresh unit weight was calculated as 2.7 and 4.3 percent lower than that of the natural aggregate concrete at w/c ratio of 0.60. That lower density was the result of lower density of the recycled aggregates due to adhered mortar. The properties of recycled aggregate and concrete made with natural aggregate and recycled aggregate obtained in this study are consistent and within the range of values obtained by other researchers [11, 26].

5.3. Mechanical Properties of Natural and Recycled Aggregate Concretes

5.3.1. Compressive Strength

Compressive strength test results at 28 and 90 days of the first stage concrete specimens made with natural aggregates (mixture I) and recycled aggregates (mixture II-IV) are given in Tables 5.4 and 5.5, and average values of compressive strengths are depicted in Figure 5.1. Test results showed that the effect of source concrete strength which means the quality of recycled aggregate did not change significantly the recycled aggregate concrete strength. The decrease in 28 days compressive strength of mixture II and mixture IV were 8 and 4 percent with respect to mixture I (control mix), respectively; the increase in compressive strength of mixture III is 4 percent with respect to the control mix. Ninety days test results also showed that the influence of recycled aggregate quality did not noticeably affect the recycled aggregate concrete within the test results investigated here. In a study [25] it has been also reported that the original concrete strength can influence the compressive strength of the new one but does not determine the result totally

even if recycled coarse aggregate has a large amount of adhered mortar. In another study [49], the authors stated that there was no noticeable difference in freezing and thawing durability of the recycled aggregate concretes with regard to strength level of the source concrete. Twenty-eight and 90 days test results also showed that the strength gain in recycled aggregate concretes were higher than natural aggregate concretes, as the strength increase in mixture I was 15 percent whereas the strength increases were 24, 21 and 17 percent for mixtures II, III and IV, respectively. The presence of unhydrated cement in recycled concrete aggregate was reported by some researchers [11, 124-125], which affected the properties of recycled aggregate concretes. Higher strength increase in recycled aggregate concretes after 28 days might be attributed to further cementing action of these unhydrated cement particles in the recycled aggregates.

Table 5.4. Compressive strength of first stage natural and recycled aggregate concretes at 28 days

Mixture No	$f_{c, 28\text{days}}$ (MPa) 1 st sample	$f_{c, 28\text{days}}$ (MPa) 2 nd sample	$f_{c, 28\text{days}}$ (MPa) 3 rd sample	$f_{c, 28\text{days}}$ (MPa) average
I	29.6	34.7	31.9	32.1
II	27.9	30.8	30.2	29.6
III	30.3	35.7	33.6	33.2
IV	34.7	29.7	27.7	30.7

Mixture I: Natural aggregate (control mix)

Mixture II: by weight, 50% natural aggregate & 50% recycled aggregate RA16

Mixture III: by weight, 50% natural aggregate & 50% recycled aggregate RA25

Mixture IV: by weight, 50% natural aggregate & 50% recycled aggregate RA35

Table 5.5. Compressive strength of first stage natural and recycled aggregate concretes at 90 days

Mixture No	$f_{c, 90\text{days}}$ (MPa) 1 st sample	$f_{c, 90\text{days}}$ (MPa) 2 nd sample	$f_{c, 90\text{days}}$ (MPa) 3 rd sample	$f_{c, 90\text{days}}$ (MPa) 4 th sample	$f_{c, 90\text{days}}$ (MPa) average
I	37.3	29.4	41.7	39.0	36.8
II	31.8	41.6			36.7
III	37.4	39.4	38.6	45.4	40.2
IV	30.8	39.0	36.6	36.7	35.8

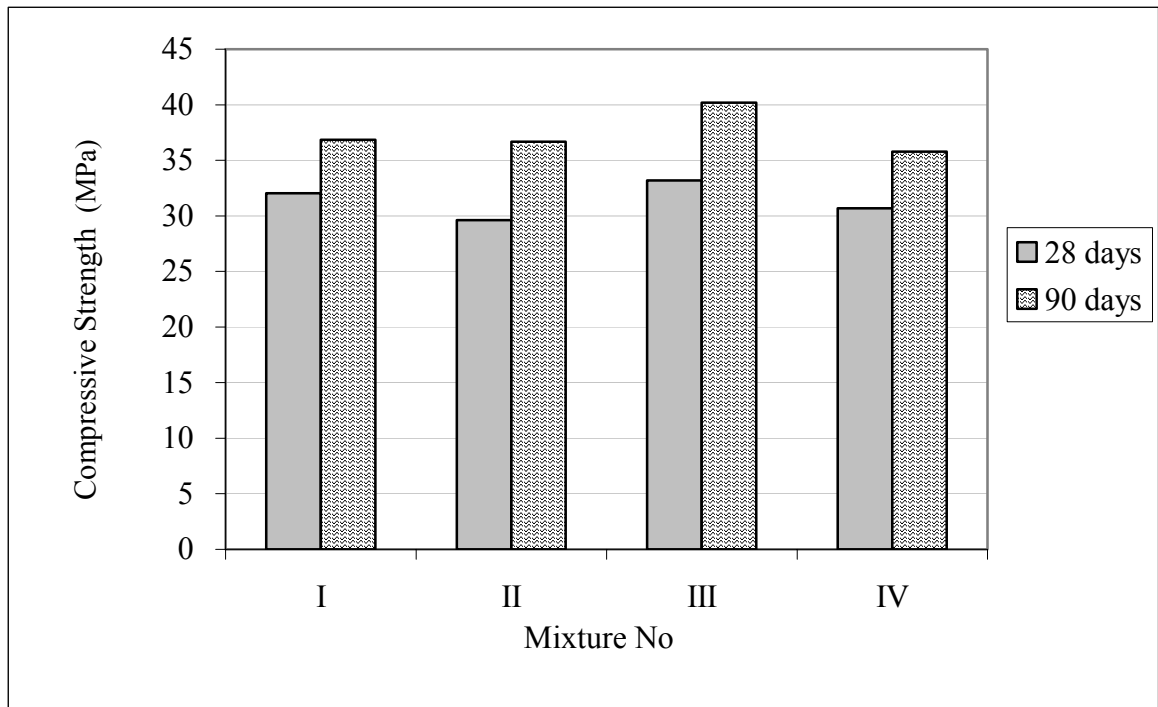


Figure 5.1. Compressive strength of first stage natural and recycled aggregate concretes

The compressive strength test results of second stage concretes, mixes 1-31, produced with various water/cement and replacement ratios and tested at 28 and 90 days at room temperature and after exposure to elevated temperatures are given in Table 5.6. The mean, standard deviation and coefficient of variation of compressive strengths of mixes are included in Table 5.6. Standard deviation is the most common measure of statistical dispersion, measuring how widely spread the values in a data set are. If the data points are all close to the mean, then the standard deviation is close to zero. If many data points are far from the mean, then the standard deviation is far from zero. If all the data values are equal, then the standard deviation is zero. In probability theory and statistics, the coefficient of variation (CV) is a measure of dispersion of a probability distribution. It is defined as the ratio of the standard deviation (σ) to the mean (μ). As suggested from Design Expert software, 2FI model was selected from analysis tool to analyze and draw 3D view of the average compressive strength values of second stage concretes. The standard deviation and the R-squared values of the model were 2.04 and 0.97. 2FI model was as follows:

$$R = I + c_1A + c_2B + c_3C + c_4D + c_5AB + c_6AC + c_7AD + c_8BC + c_9BD + c_{10}CD \quad (5.1)$$

Table 5.6. Compressive strength of second stage natural & recycled aggregate concretes

Run	w/c ⁽¹⁾	RR ⁽²⁾ (%)	T ⁽³⁾ (°C)	Age (days)	f _{c1} (MPa)	f _{c2} (MPa)	f _{c3} (MPa)	$\bar{f}_c$ (MPa)	σ	CV
1	0.35	0	22	90	39.3	45.3	35.6	40.0	4.00	10.0%
2	0.35	0	105	28	39.7	31.8	30.0	33.8	4.21	12.5%
3	0.35	0	105	28	31.9	29.3	28.6	29.9	1.43	4.8%
4	0.35	0	175	28	50.5	36.8	47.0	44.8	5.83	13.0%
5	0.35	0	250	28	50.5	49.1	57.0	52.2	3.42	6.6%
6	0.35	50	22	28	34.0	38.0	41.1	37.7	2.90	7.7%
7	0.35	50	105	90	41.7	39.6	36.6	39.3	2.09	5.3%
8	0.35	100	22	90	44.4	38.8	36.9	40.0	3.20	8.0%
9	0.35	100	105	28	43.0	35.5	33.1	37.2	4.20	11.3%
10	0.35	100	175	90	40.2	42.3	39.2	40.6	1.30	3.2%
11	0.35	100	250	90	37.8	45.2	45.2	42.7	3.49	8.2%
12	0.4125	25	250	90	46.9	39.7	48.1	44.9	3.70	8.2%
13	0.475	0	175	90	32.8	36.7	34.1	34.6	1.63	4.7%
14	0.475	0	175	90	32.4	33.8	32.8	33.0	0.57	1.7%
15	0.475	25	22	28	32.7	32.4	31.6	32.2	0.48	1.5%
16	0.475	50	250	28	42.3	37.5	36.2	38.7	2.61	6.8%
17	0.475	100	105	90	31.8	33.1	27.2	30.7	2.54	8.3%
18	0.475	100	175	28	32.6	28.5	34.9	32.0	2.67	8.3%
19	0.5375	25	175	28	26.0	28.0	29.1	27.7	1.28	4.6%
20	0.5375	50	22	90	30.4	33.3	30.1	31.3	1.43	4.6%
21	0.5375	75	250	90	33.0	33.8	33.3	33.3	0.34	1.0%
22	0.60	0	22	28	25.4	22.7	20.6	22.9	1.98	8.6%
23	0.60	0	105	90	25.4	25.6	23.3	24.8	1.03	4.2%
24	0.60	0	105	90	21.2	22.2	25.8	23.0	1.96	8.5%
25	0.60	0	250	90	29.7	35.7	32.2	32.5	2.45	7.5%
26	0.60	50	105	28	23.5	24.9	20.9	23.1	1.66	7.2%
27	0.60	50	105	28	26.7	24.3	25.6	25.5	0.96	3.8%
28	0.60	100	22	28	28.0	26.8	24.0	26.3	1.67	6.3%
29	0.60	100	175	90	26.7	29.5	28.9	28.4	1.20	4.2%
30	0.60	100	175	90	23.2	25.6	27.9	25.6	1.92	7.5%
31	0.60	100	250	28	25.9	25.0	23.4	24.8	1.04	4.2%

⁽¹⁾w/c: water/cement ratio, ⁽²⁾RR: replacement ratio, ⁽³⁾T: temperature

f_{c1}, f_{c2}, f_{c3}: compressive strength of specimens no 1, 2 and 3

$\bar{f}_c$: mean of f_{c1}, f_{c2}, f_{c3} & σ : standard deviation & CV: coefficient of variation

where:

R= response (compressive strength value)

I= intercept

A= water-cement ratio, w/c

B= replacement ratio

C= temperature

D= age

c₁, c₂... c₁₀= coefficients

The equation of the compressive strength created by the software in terms of coded factors was as follows:

$$\begin{aligned}
 \text{Compressive Strength (MPa)} = & 33.77 - 7.35 * A - 0.25 * B - 0.92 * C[1] - 3.40 * C[2] \\
 & - 0.25 * C[3] + 0.83 * D + 0.82 * AB + 1.06 * AC[1] \\
 & + 2.22 * AC[2] - 1.29 * AC[3] + 0.38 * AD + 0.95 * BC[1] \quad (5.2) \\
 & + 2.59 * BC[2] - 0.017 * BC[3] - 0.10 * BD \\
 & + 0.48 * C[1]D + 0.40 * C[2]D - 0.39 * C[3]D
 \end{aligned}$$

In the above equation the factors were used as coded. Design descriptions and analyses were best done with coded factors. Coding was a way to simplify calculations by converting the low and higher levels to -1 and +1 respectively. Coding allowed the calculations to be performed independent of the units for each factor. It also reduced the range of each factor to a common scale -1 to +1, regardless of its relative magnitude. The coding table of each factor is seen in Tables 5.7-5.10. By using the values in the coding tables the compressive strength response could be predicted. The predicted and actual values of the compressive strength, residual which is the difference of actual and predicted values and errors calculated by dividing the absolute value of residual to actual value are seen in Table 5.11, and graphed in Figure 5.2; and predicted values of compressive strengths are shown in Figures 5.3-5.10. The R-squared value of 0.97 shows that 2FI model is good enough for compressive strength response. The results of the ANOVA for response surface 2FI model are presented in Table 5.12. As seen from ANOVA table the model terms A, C, AC, and BC are significant, because the Prob>F values are less than 0.05. This means that water/cement ratio (A) and the exposure temperature (C) have important effects

on compressive strength. An interaction occurs when the response is different depending on the settings of two factors. Plots make it easy to interpret two factor interactions. They will appear with two non-parallel lines, indicating that the effect of one factor depends on the level of the other. For the response compressive strength, the relations between water/cement and replacement ratios are given in Figures 5.11-5.12; the interaction graphs of water/cement ratio and temperature are illustrated in Figure 5.13; the interaction graphs of replacement ratio and temperature are seen in Figure 5.14; and the interaction graphs of temperature and age are presented in Figure 5.15.

As seen from Equation 5.2 the biggest coefficient belonging to “A” (water/cement ratio) was 7.35, this highest coefficient and analysis of variance (ANOVA) showed the main factor for compressive strength was the water/cement ratio. It is clearly seen from 3D surface view figures that compressive strength increased with the decrease of water/cement ratio at all ages, exposed temperatures and replacement ratios, as shown in Figures 5.3-5.10. Twenty eight days test results showed that at zero replacement ratio, when water/cement ratio decreased from 0.60 to 0.35 the raise in compressive strength was 65 percent at 22 °C, 62 percent at 105 °C, 84 percent at 175 °C, and 68 percent at 250 °C; and the increase was 50, 47, 72, and 60 percent at 22, 105, 175, and 250 °C, respectively, when 90 days test results were compared. When the replacement ratio was 100 percent, 28 days test results showed that the compressive strength increased 44 percent at 22 °C, 35 percent at 105 °C, 66 percent at 175 °C, and 70 percent at 250 °C as water/cement ratio decreased from 0.60 to 0.35; and the increase was 34 percent at 22 °C, 26 percent at 105 °C, 57 percent at 175 °C, and 61 percent at 250 °C when the testing age was 90 days.

In Equation 5.2 the coefficient of “B” (replacement ratio) was only 0.25 which means the individual effect of replacement ratio on compressive strength of recycled aggregate concrete was very small compared to the factors water/cement ratio (A) and the temperature (C). As shown in Figures 5.12-5.13, the interaction graphs of replacement ratio at 28 and 90 days, and at every water/cement ratio ranging from 0.35 to 0.60 and at all exposed temperatures showed also the slight effect of replacement ratio. At w/c ratio 0.35, when the replacement ratio increased from zero to 100 percent, there was no quite change in compressive strength at 22 °C, at both 28 and 90 days; 10 and 8 percent increase, respectively at 28 and 90 days was calculated in compressive strength after exposure to

105 °C; the decrease in compressive strength after being exposed to 175 °C was 5 percent both at 28 and 90 days; and 21 and 22 percent decrease at 28 and 90 days, respectively, was calculated in compressive strength after exposure to 250 °C. At w/c ratio 0.60, when the replacement ratio increased from zero to 100 percent, the increase in compressive strength at 22 °C was, respectively, 14 and 11 percent at 28 and 90 days; 32 and 26 percent increase at 28 and 90 days, respectively, was calculated in compressive strength after exposure to 105 °C; the increase in compressive strength after exposure to 175 °C was 6 and 4 percent, respectively, at 28 and 90 days; and 22 and 23 percent decrease at 28 and 90 days was calculated respectively in compressive strength after exposure to 250 °C.

As reported in analysis of variance table as listed in Table 5.12 and shown in temperature interaction figures as shown in Figures 5.13-5.15 the effect of temperature factor was significant. ANOVA results showed that there exist an interaction between AC and BC; these interactions are seen from Figures 5.13-5.14. After exposed to 105 °C heat the compressive strength decreased (with respect to 22 °C test results) 15 and 14 percent respectively at 28 and 90 days when the w/c and replacement ratios were 0.35 and zero percent, respectively. When the w/c and replacement ratios were 0.35 and 100 percent respectively the decrease was 5 percent for both 28 and 90 days; 13 and 12 percent decrease was calculated when the w/c and replacement ratios were 0.60 and zero percent, respectively; and very small increase was observed in the amount of two and one percent when the w/c and replacement ratios were 0.60 and 100 percent. The compressive strength increased when the w/c ratio was 0.35, and a decrease was observed at 0.60 w/c ratio after the concrete was exposed to 175 °C. The amount of increase at 0.35 w/c ratio was 13 and 8 percent at 28 and 90 days, respectively, at zero percent replacement ratio, and 8 and three percent at 28 and 90 days, respectively, and at 100 percent replacement ratio. The amount of decrease at 0.60 w/c ratio was 6 percent at 90 days at zero percent replacement ratio, and 8 and 13 percent at 28 and 90 days at 100 percent replacement ratio. At zero percent replacement ratio, the compressive strength was affected very much by exposing the specimens to 250 °C. The increase was 37 and 30 percent at 28 and 90 days and at w/c ratio 0.35, and it was 34 and 22 percent at 28 and 90 days and at w/c ratio 0.60. At 100 percent replacement ratio the compressive strength increased at 0.35 w/c ratio and decreased at 0.60 w/c ratio when the concretes were tested after exposure to 250 °C. The

increase was 13 and 8 percent at 28 and 90 days and at 0.35 w/c ratio, and the decrease was 4 and 11 percent at 28 and 90 days and at 0.60 w/c ratio.

In Equation 5.2 the coefficient of “D” (age) was 0.83, this small coefficient affected the compressive strength of recycled aggregate concrete slightly. Test results and interaction figures of age showed that age has a positive effect on compressive strength, as shown in Figure 5.15. At 22 °C and 0.35 w/c ratio, the gain in compressive strength from 28 to 90 days was 5 and 4 percent at zero and 100 percent replacement ratios; it was 15 and 12 percent at zero and 100 percent replacement ratio and at 0.60 w/c ratio. After exposure to 105 °C the increase in the compressive strength was 6 and 4 percent at zero and 100 percent replacement ratios and at 0.35 w/c ratio and it was 17 and 11 percent at zero and 100 percent replacement ratios and at 0.60 w/c ratio. After exposure to 175 °C the increase in the compressive strength was one percent at 0.35 w/c ratio and at zero percent replacement ratio and it was 8 and 6 percent at zero and 100 percent replacement ratios and at 0.60 w/c ratio. After exposure to 250 °C there was almost no change in compressive strength at zero and 100 percent replacement ratios and at 0.35 w/c ratio and the increase was 5 percent at both zero and 100 percent replacement ratios and at 0.60 w/c ratio.

From the test results and findings presented in the above paragraphs, the following conclusions for the compressive strength of concrete are drawn:

1. The incorporation of recycled aggregate influenced the compressive strength of concretes. Compressive strength of recycled aggregate concretes ranged from 25 to 43 MPa, whereas the range was larger in natural aggregate concretes, which was 20 to 52 MPa. That range was because of w/c ratio, exposed temperature and ages of natural and recycled aggregate concretes.
2. The test results showed that each temperature range indicated a different pattern of strength increase or decrease. From 22 to 105 °C, compressive strength of recycled aggregate concretes generally indicated higher strength than natural aggregate concretes, but no more than 14 percent. Compressive strength test results of recycled aggregate concretes exposed to 175 °C was lower than natural aggregate concretes by no more than 5 percent when the w/c ratio was 0.35, while the strength of recycled aggregate concretes was higher than natural aggregate concretes by 4-6 percent when

the w/c ratio was 0.60. A considerable strength difference was observed between natural and recycled aggregate concretes exposed to 250 °C; compressive strength of recycled aggregate concretes were lower than natural aggregate concretes, which was 21-23 percent. The recycled aggregate concretes did not lose their strength after 175 °C, but the strength increase was not as much natural aggregate concrete, the 21-23 percent difference was due to the high strength gain of natural aggregate concretes after 175 °C. Lastly, it could be said that recycled aggregate concretes may serve for the strength purpose with a compressive strength of 43 MPa until 250 °C if the concretes were produced with w/c ratio of 0.35.

3. For the natural aggregate concretes, generally the compressive strength decreased at 105 °C, recovered to 22 °C strength at 175 °C and then increased till 250 °C. For the recycled aggregates no general comment could be drawn. But test results indicated that compressive strength of recycled aggregate concretes were less affected with temperature elevations when compared with natural aggregate concretes.
4. Compressive strength of natural and recycled aggregate concretes increased with time, as expected. The highest increase of compressive strength with time was observed in natural aggregate concretes produced with 0.60 w/c ratio and tested at 105 °C.
5. When 28 days test results were compared, compressive strength of natural aggregates concretes increased 62-84 percent with decrease of w/c ratio from 0.60 to 0.35, whereas for the same range of w/c ratio the increase was 35-70 percent for recycled aggregate concretes. This result showed that changes in w/c ratio affected the natural aggregate concretes more than recycled aggregate concretes. On analyzing the 90 days result, same consequence was figured out. It seems that, apart from w/c ratio, the moisture condition of the recycled aggregate also appears to affect the compressive strength [56]. On the other hand, the properties of coarse aggregate such as particle shape and surface texture are also found to play an important role in controlling the strength of concrete [38]. To finish, the compressive strength of natural and recycled aggregate concretes largely depended on the w/c ratio of the mix.

Though most researchers have reported a reduction in strength in recycled aggregate concretes, some recent studies [25, 48] showed that the strength of recycled aggregate

concrete is comparable to that of natural aggregate concrete, which is valid for our study. If a brief evaluation is made then it can be said that the effect of replacement ratio factor on compressive strength was lower than the water/cement ratio and temperature factors. Concretes that will be subjected to temperatures even as high as 250 °C could be produced safely with recycled aggregates by lowering the water/cement ratio if the compressive strength is in concern.

Table 5.7. Coding of w/c ratio factor

Coded value	Actual value
-1	0.35
-0.5	0.4125
0	0.475
0.5	0.5375
1	0.60

Table 5.8. Coding of replacement ratio factor

Coded value	Actual value (%)
-1	0
-0.5	25
0	50
0.5	75
1	100

Table 5.9. Coding of temperature factor

Coded values	Temperature (°C)
C[1]=1, C[2]=0, C[3]=0	22
C[1]=0, C[2]=1, C[3]=0	105
C[1]=0, C[2]=0, C[3]=1	175
C[1]=-1, C[2]=-1, C[3]=-1	250

Table 5.10. Coding of age factor

Coded values	Age (days)
-1	28
1	90

Table 5.11. Actual and predicted values of compressive strength

Run	w/c	RR (%)	T (°C)	Age (days)	Actual value (MPa)	Predicted value (MPa)	Residual (MPa)	Error
1	0.35	0	22	90	40.0	40.3	-0.2	0.6%
2	0.35	0	105	28	33.8	33.0	0.8	2.3%
3	0.35	0	105	28	29.9	33.0	-3.1	10.4%
4	0.35	0	175	28	44.8	43.1	1.7	3.7%
5	0.35	0	250	28	52.2	52.2	0.0	0.0%
6	0.35	50	22	28	37.7	38.2	-0.5	1.3%
7	0.35	50	105	90	39.3	36.4	2.9	7.5%
8	0.35	100	22	90	40.0	39.9	0.2	0.5%
9	0.35	100	105	28	37.2	36.3	1.0	2.6%
10	0.35	100	175	90	40.6	41.0	-0.5	1.1%
11	0.35	100	250	90	42.7	43.0	-0.2	0.6%
12	0.4125	25	250	90	44.9	45.3	-0.4	0.9%
13	0.475	0	175	90	34.6	34.3	0.2	0.6%
14	0.475	0	175	90	33.0	34.3	-1.3	4.1%
15	0.475	25	22	28	32.2	31.1	1.1	3.3%
16	0.475	50	250	28	38.7	38.0	0.7	1.7%
17	0.475	100	105	90	30.7	33.9	-3.1	10.2%
18	0.475	100	175	28	32.0	32.9	-0.9	2.9%
19	0.5375	25	175	28	27.7	28.5	-0.7	2.7%
20	0.5375	50	22	90	31.3	31.2	0.1	0.2%
21	0.5375	75	250	90	33.3	32.5	0.9	2.6%
22	0.60	0	22	28	22.9	23.2	-0.3	1.4%
23	0.60	0	105	90	24.8	23.8	1.0	3.8%
24	0.60	0	105	90	23.0	23.8	-0.8	3.3%
25	0.60	0	250	90	32.5	32.8	-0.2	0.7%
26	0.60	50	105	28	23.1	23.6	-0.5	2.3%
27	0.60	50	105	28	25.5	23.6	1.9	7.4%
28	0.60	100	22	28	26.3	26.5	-0.2	0.9%
29	0.60	100	175	90	28.4	26.2	2.2	7.7%
30	0.60	100	175	90	25.6	26.2	-0.6	2.3%
31	0.60	100	250	28	24.8	25.4	-0.6	2.6%

Table 5.12. Results of ANOVA for compressive strength response surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	839.02	1	839.02	202.41	<0.0001
B	0.19	1	0.19	0.04	0.8357
C	291.73	3	97.24	23.46	<0.0001
D	17.69	1	17.69	4.27	0.0611
AB	7.85	1	7.85	1.89	0.1940
AC	51.24	3	17.08	4.12	0.0318
AD	2.03	1	2.03	0.49	0.4975
BC	89.88	3	29.96	7.23	0.0050
BD	0.16	1	0.16	0.04	0.8454
CD	5.21	3	1.74	0.42	0.7428

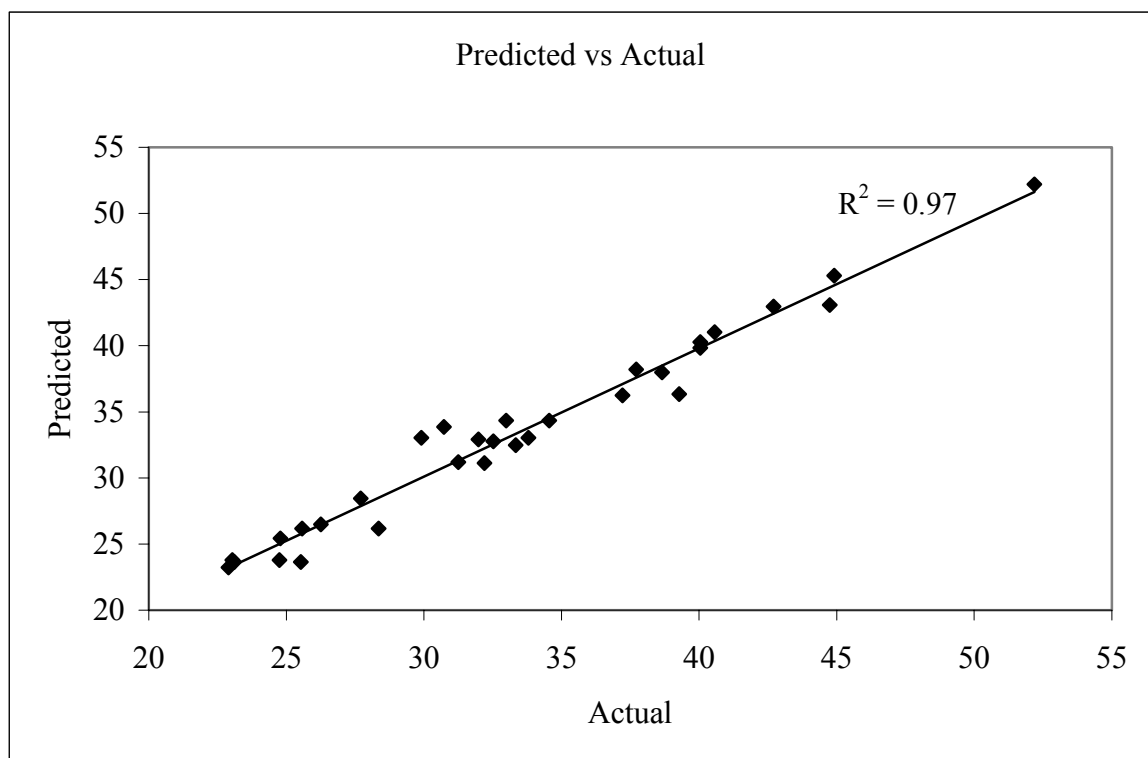


Figure 5.2. Predicted versus actual values of compressive strengths (MPa)

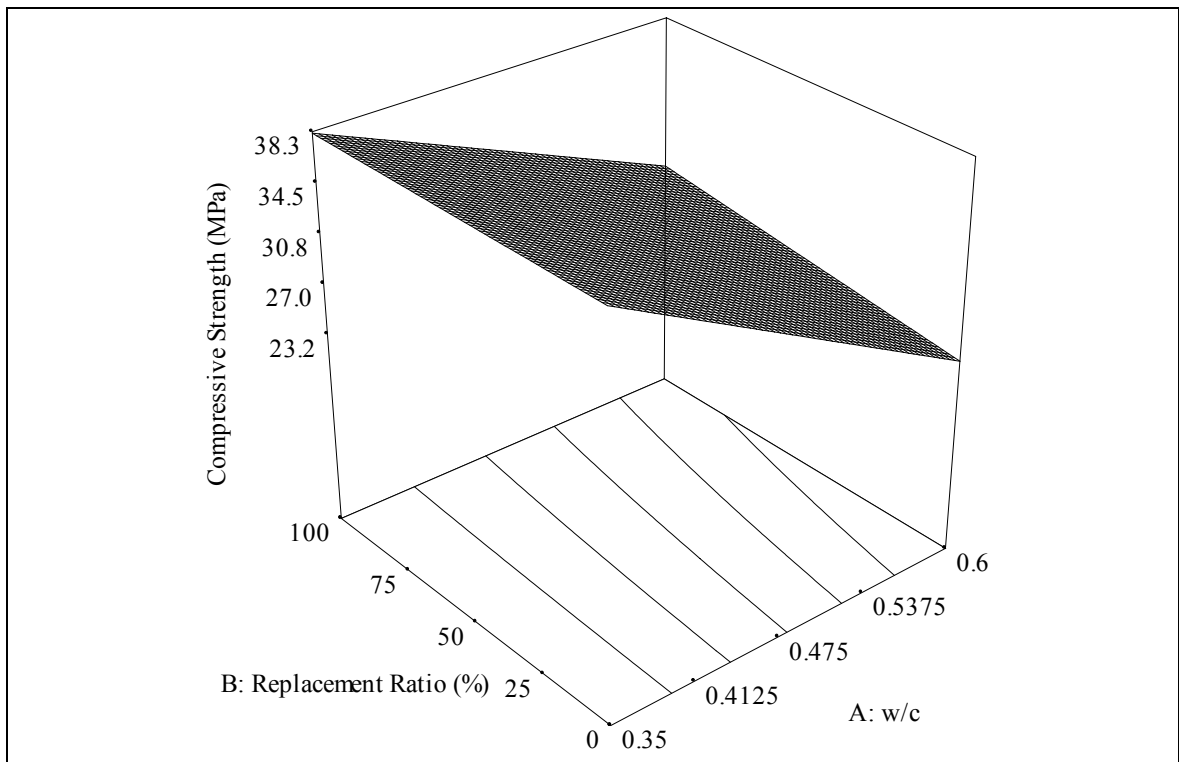


Figure 5.3. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)

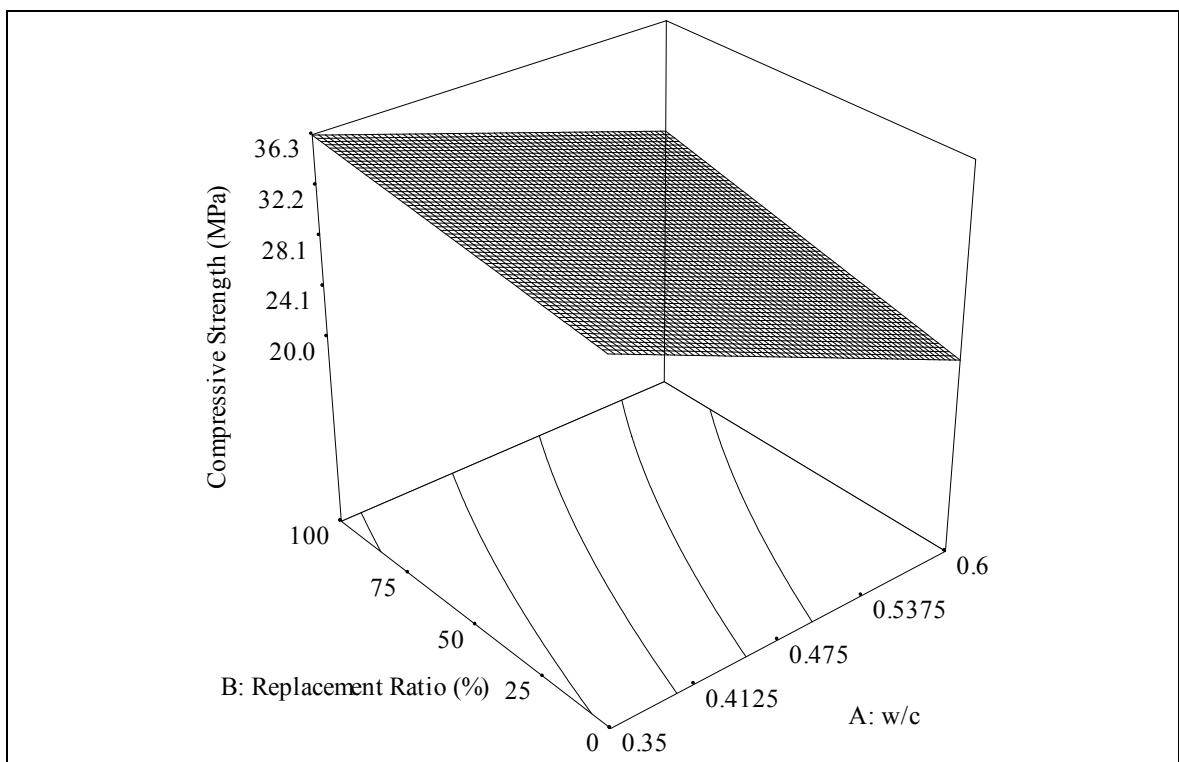


Figure 5.4. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)

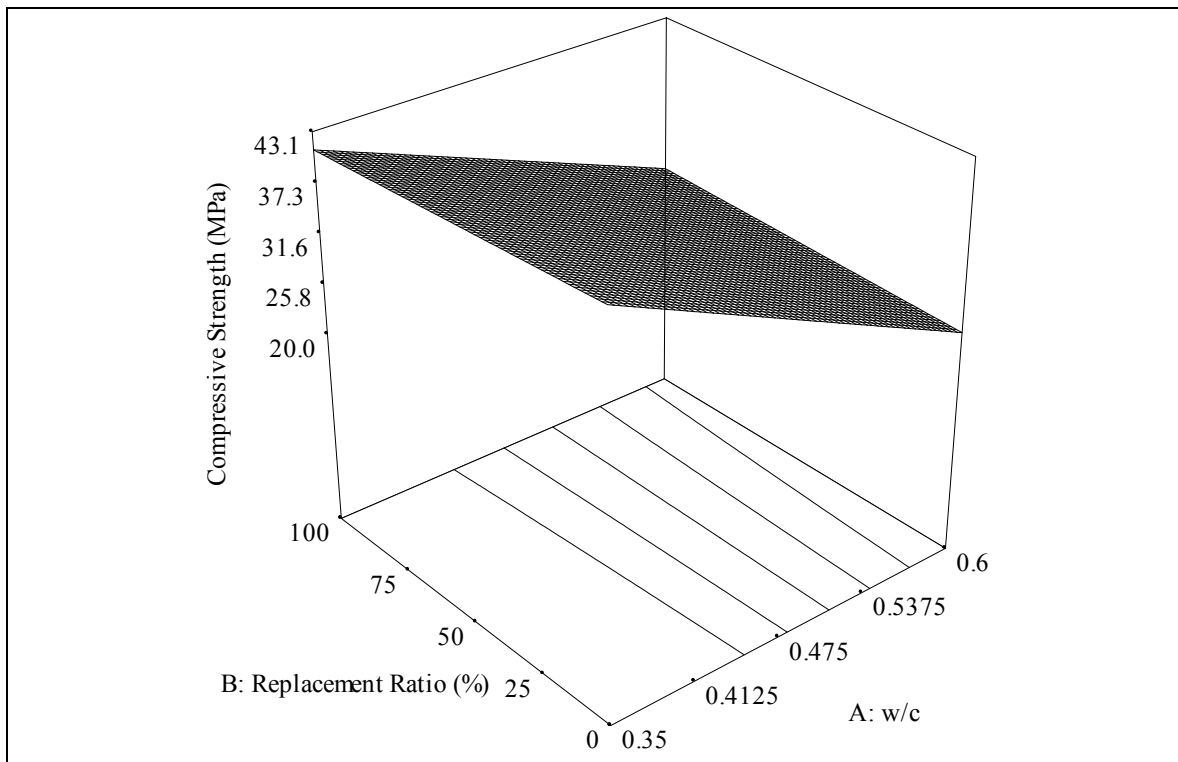


Figure 5.5. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)

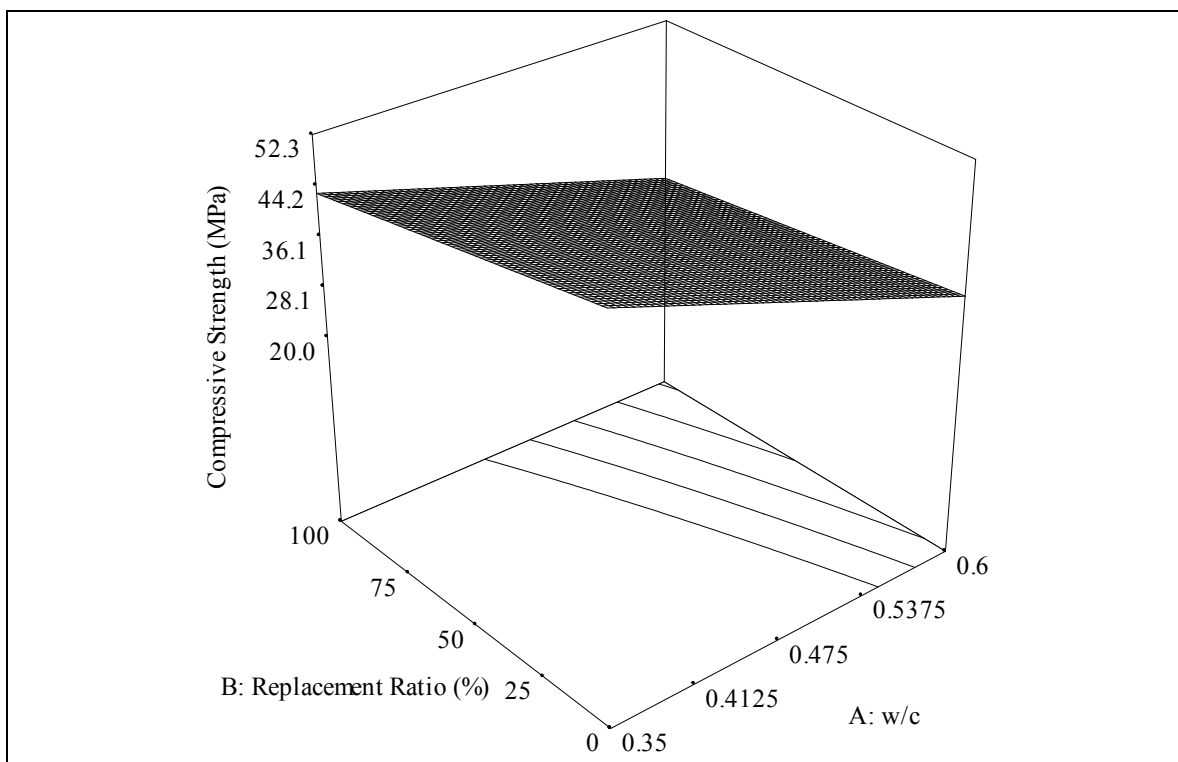


Figure 5.6. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)

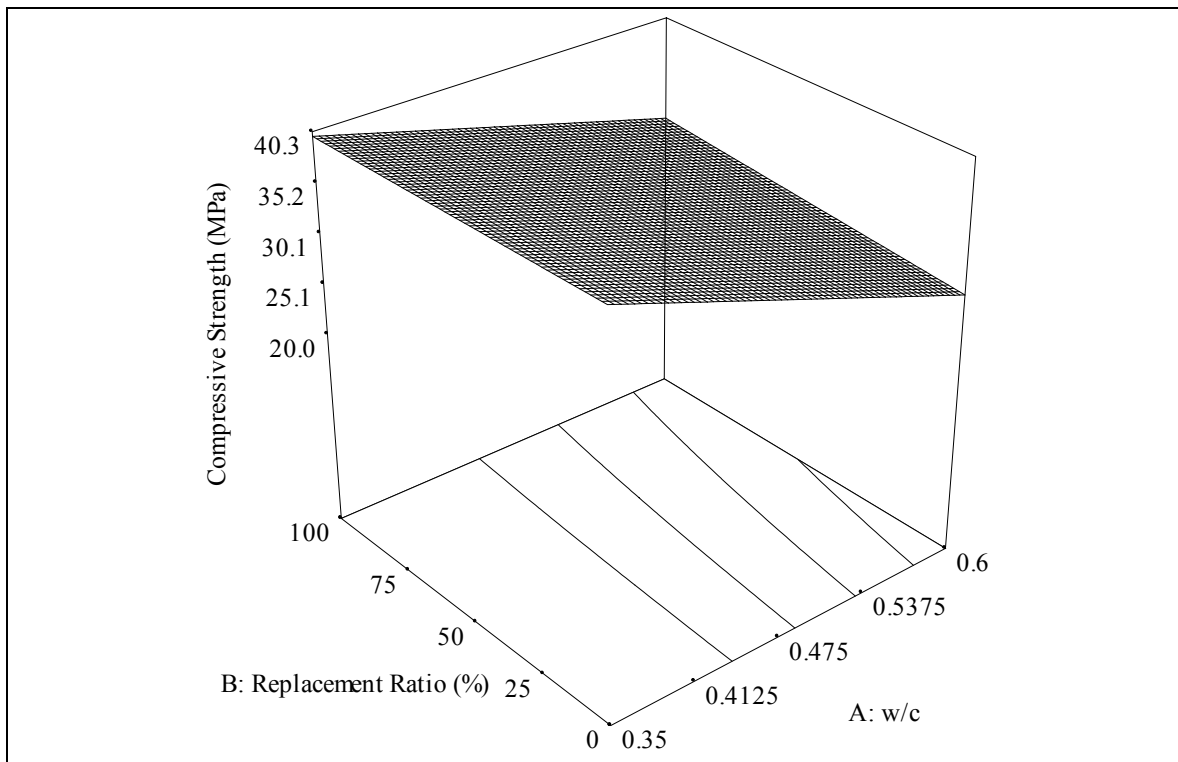


Figure 5.7. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)

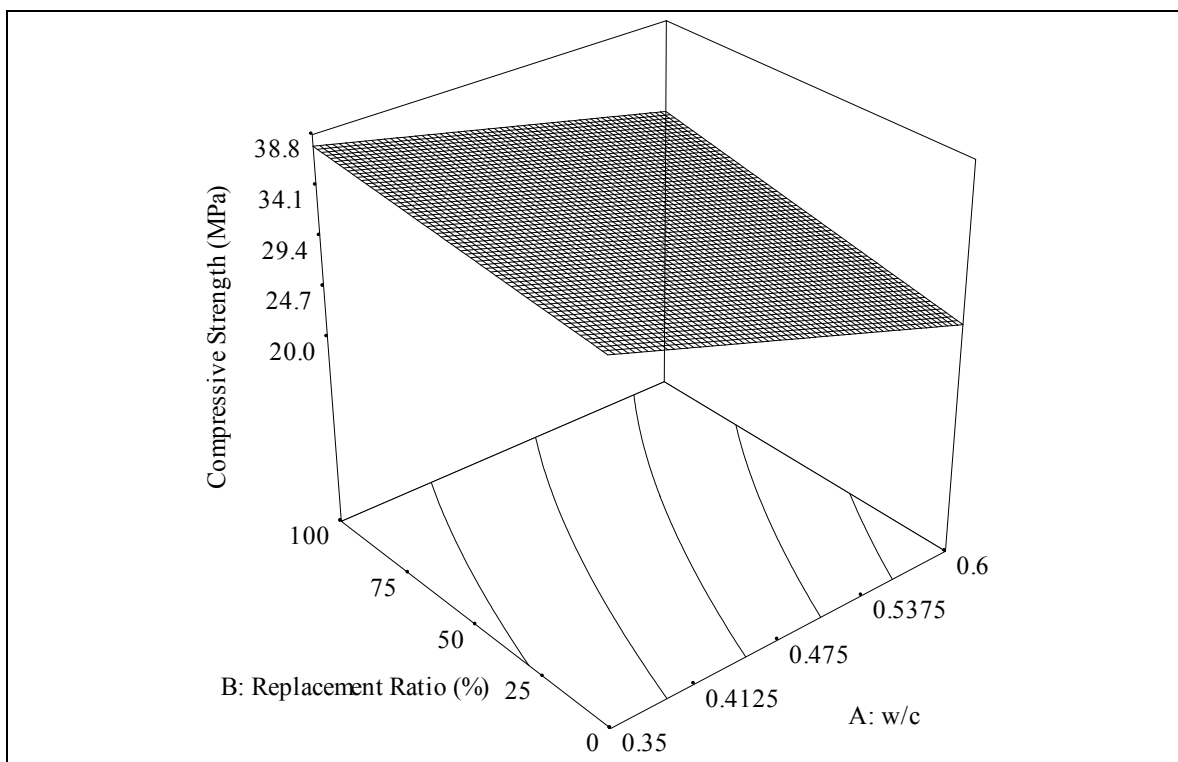


Figure 5.8. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)

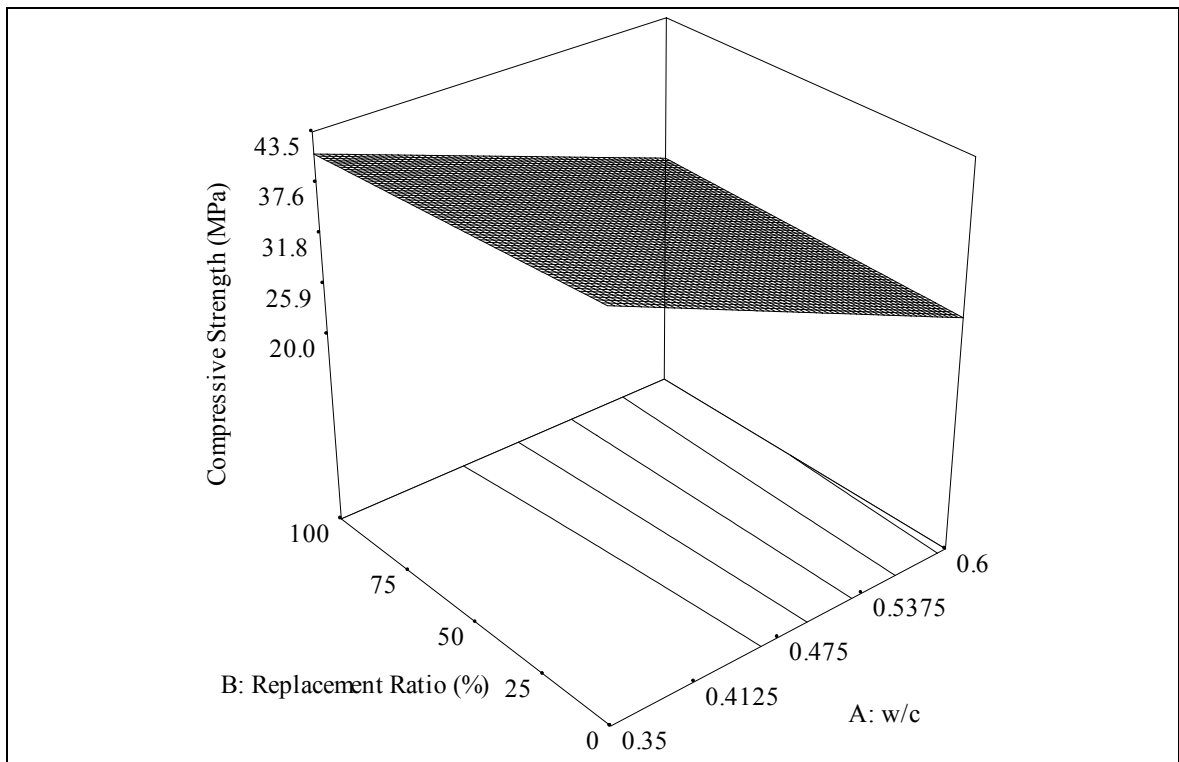


Figure 5.9. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)

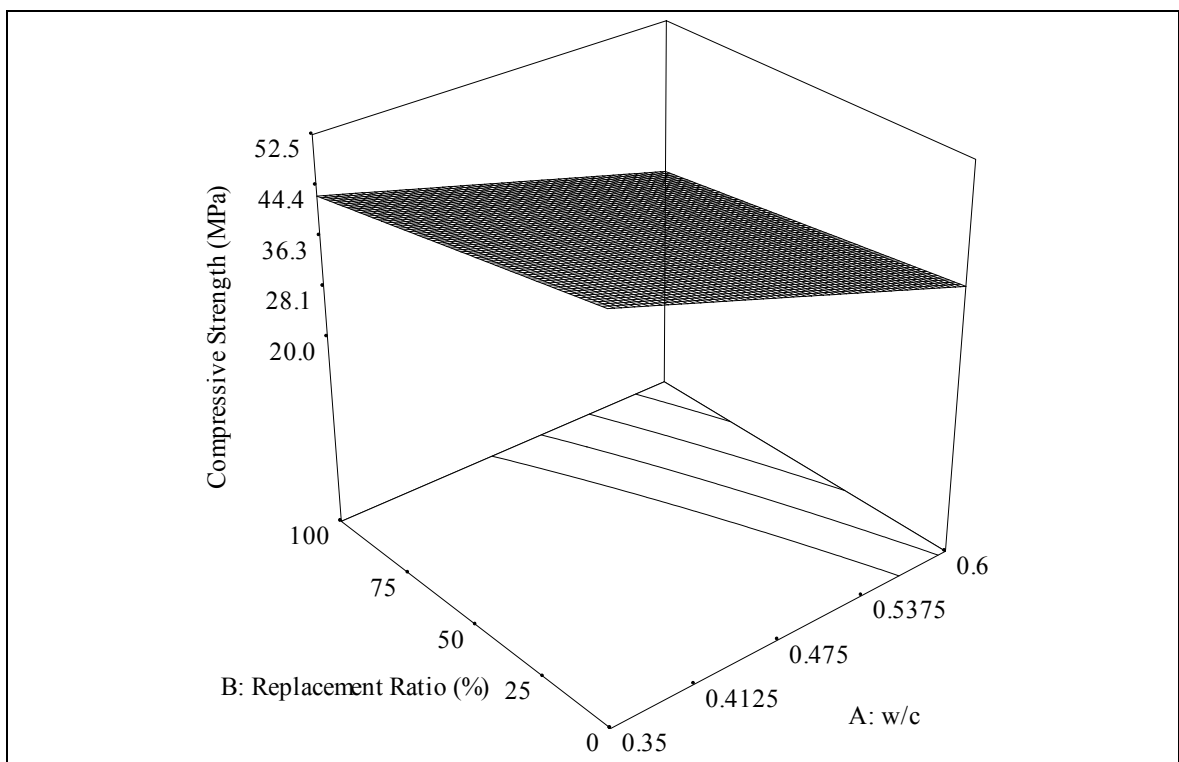


Figure 5.10. 3D surface view of compressive strength of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)

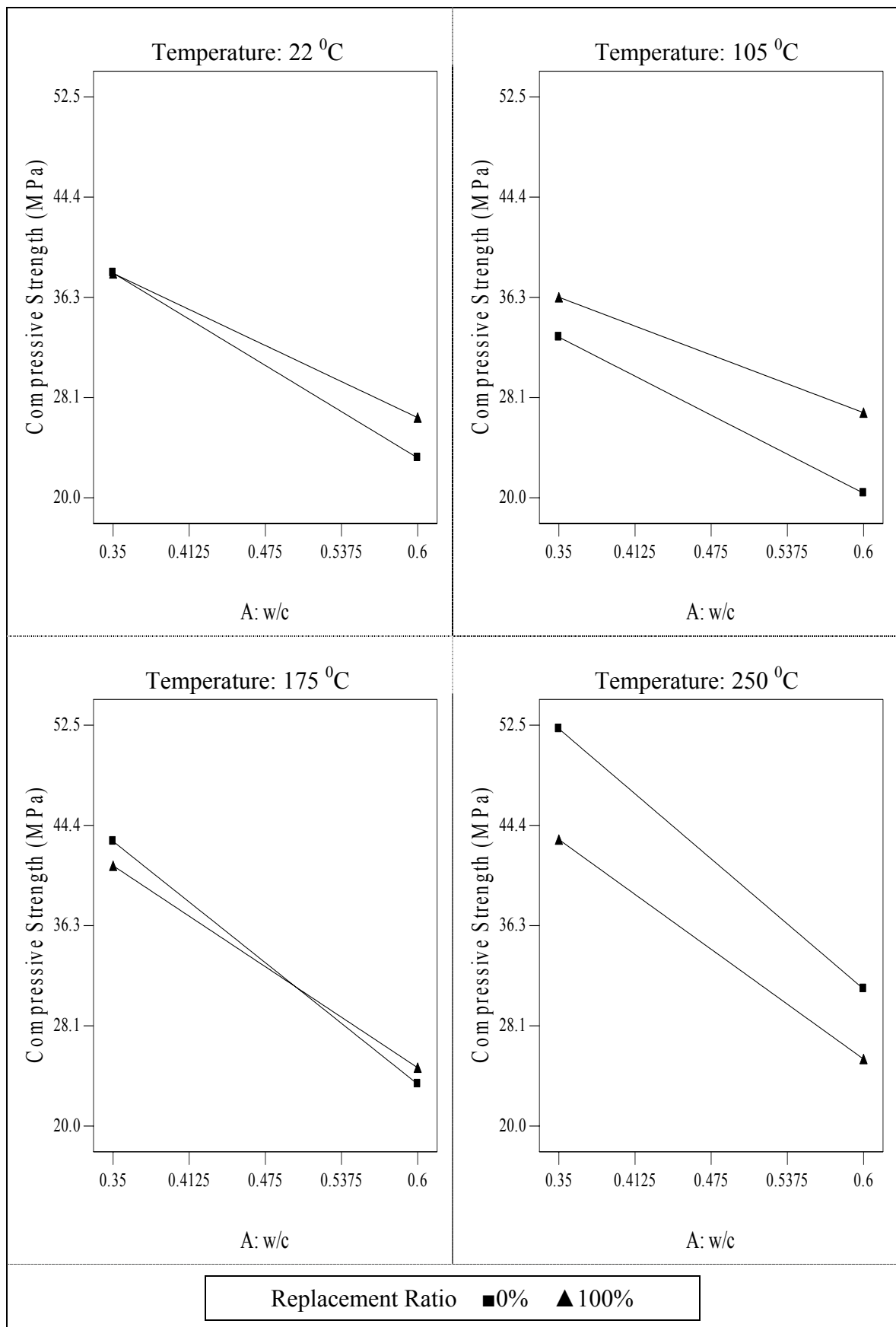


Figure 5.11. Interaction graph of w/c and replacement ratio at 28 days

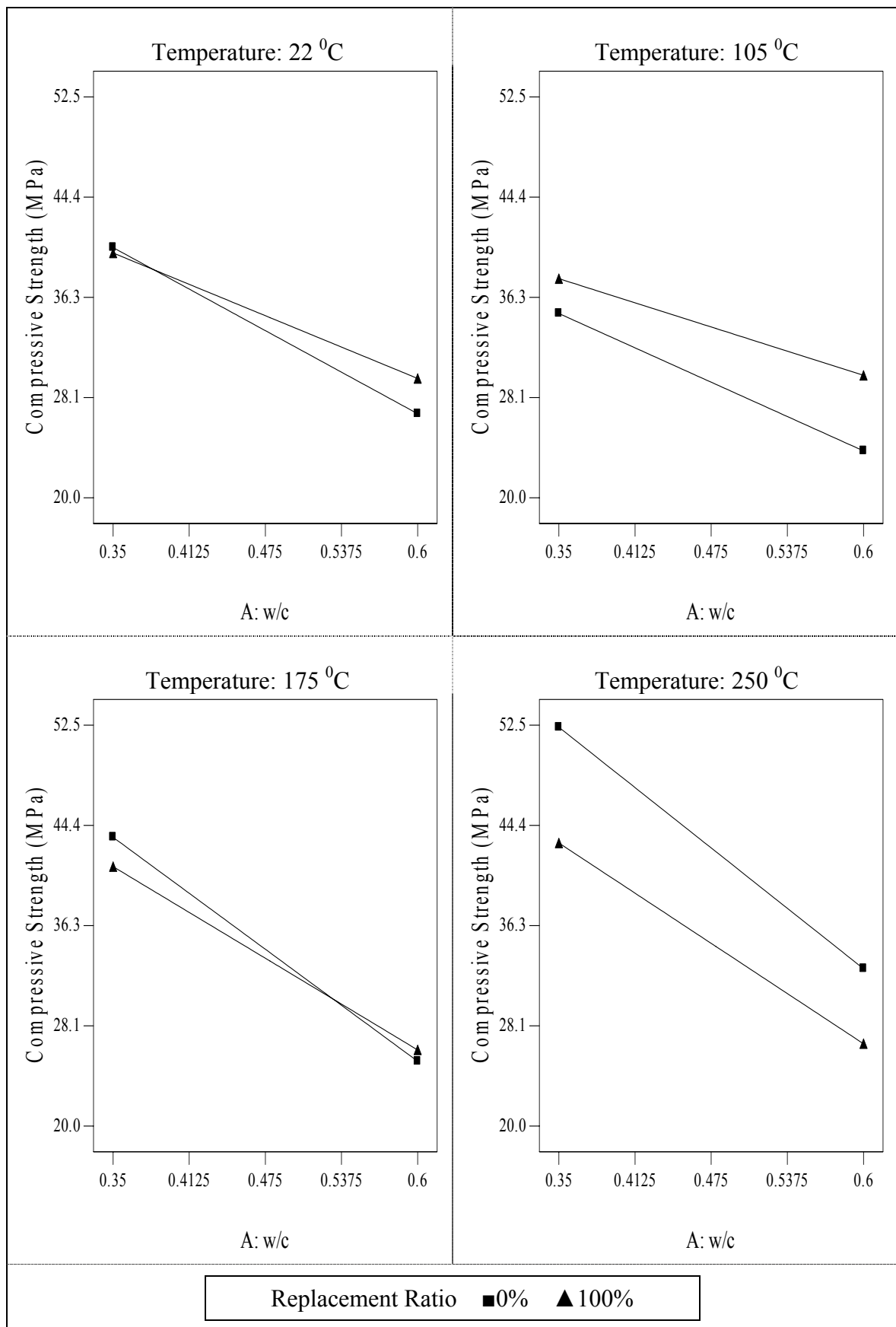


Figure 5.12. Interaction graph of w/c and replacement ratio at 90 days

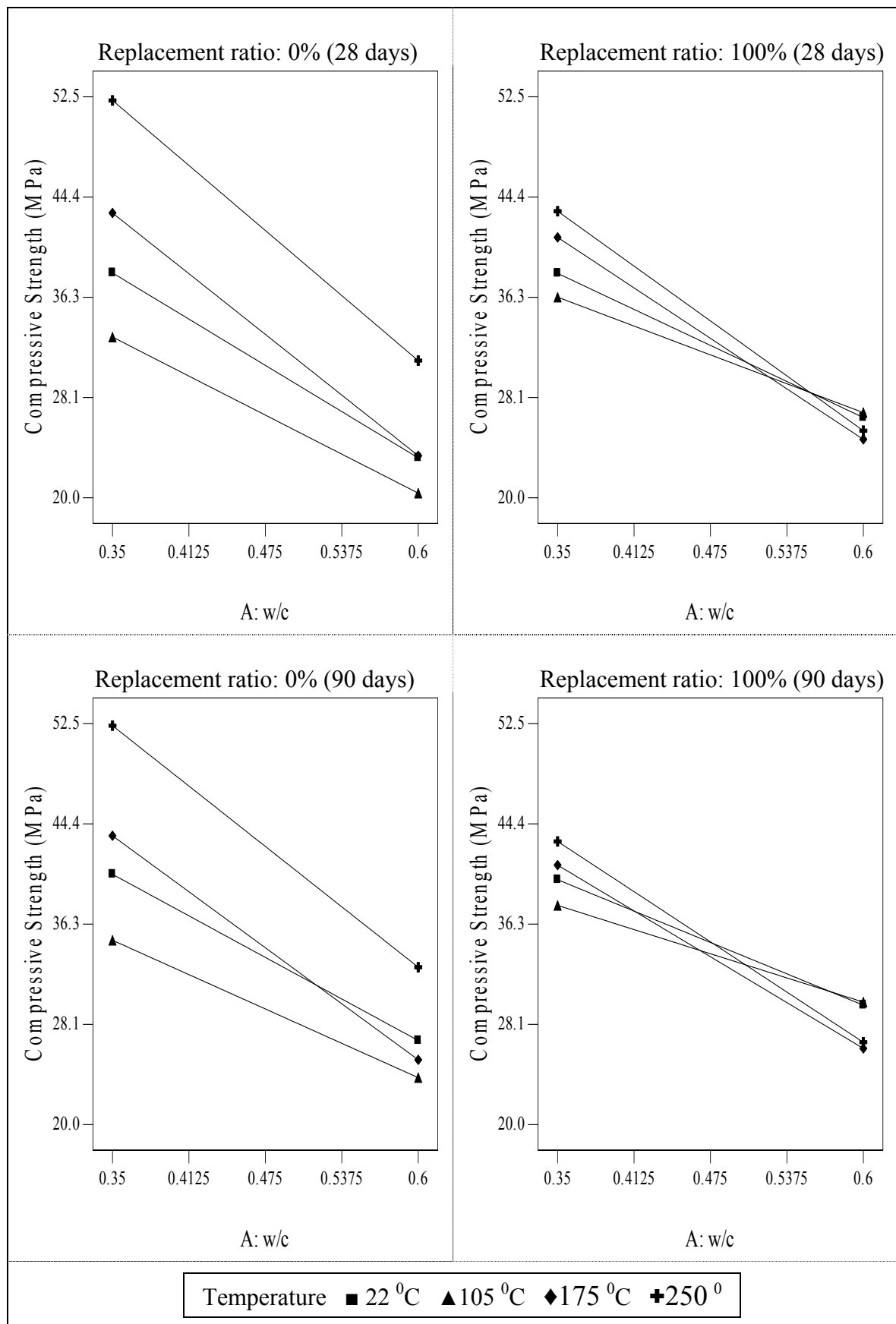


Figure 5.13. Interaction graph of w/c and temperature

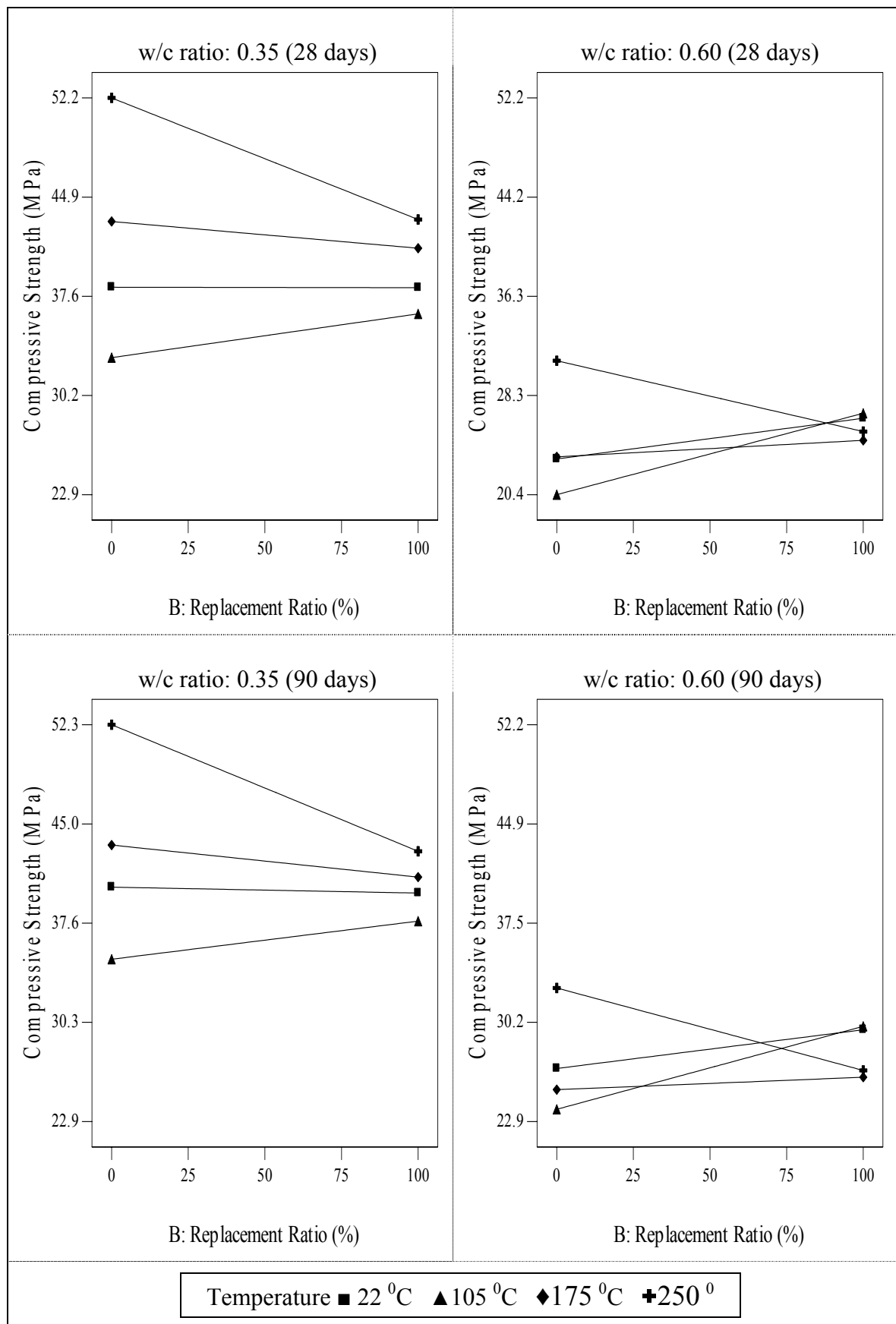


Figure 5.14. Interaction graph of replacement ratio and temperature

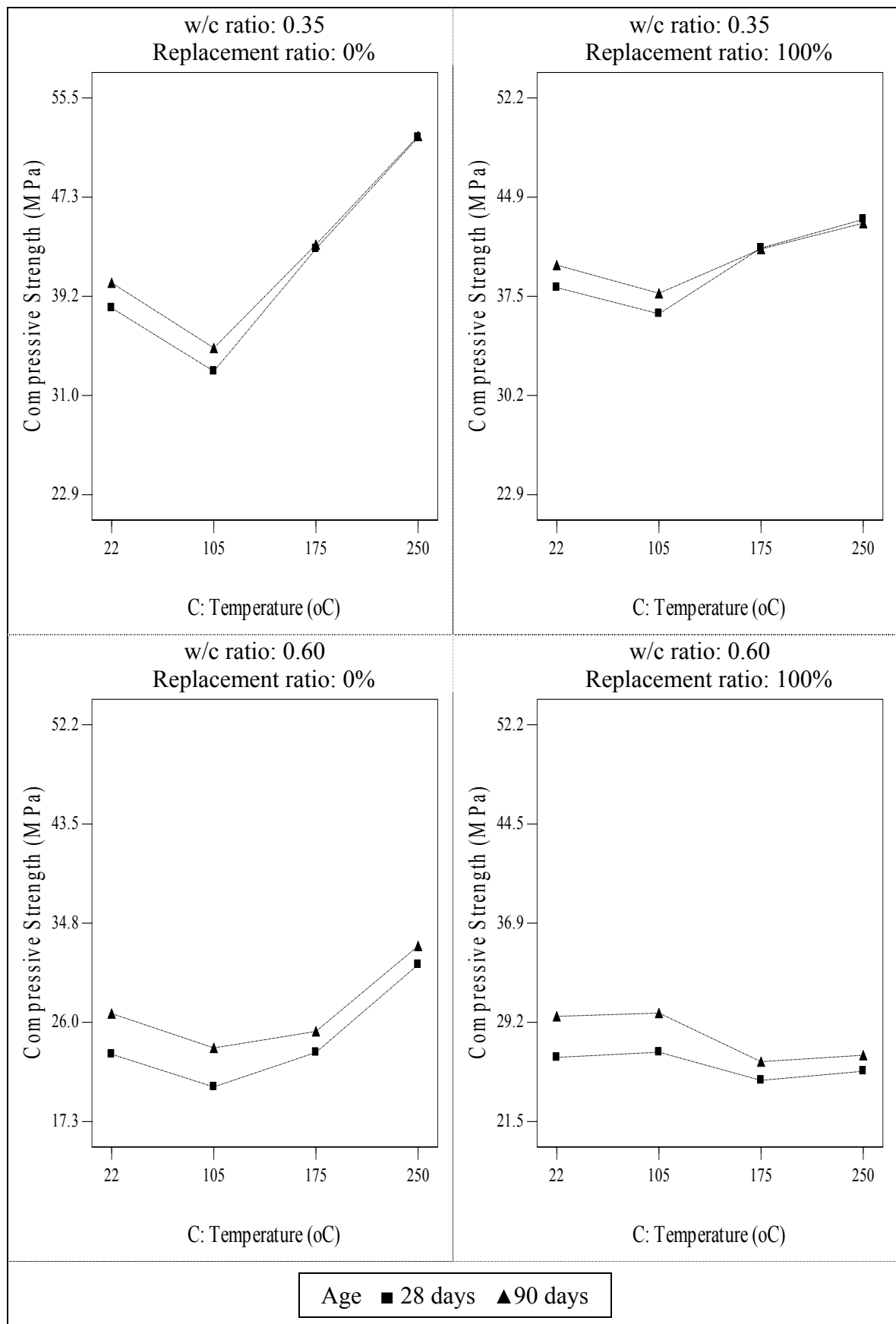


Figure 5.15. Interaction graph of temperature and age

5.3.2. Static Modulus of Elasticity

The static modulus of elasticity test results of second stage concretes, mixes 1-31, produced with various water/cement and replacement ratios and tested at 28 and 90 days at room temperature and after exposure to elevated temperatures are given in Table 5.13. The static modulus of elasticity was calculated from the stress strain curve. Two examples of compressive stress-strain curves are presented in Figures G.1-G.2 of Appendix G. 2FI model was used for analysis of modulus of elasticity values. The standard deviation and the R-squared values of the model were 1.41 and 0.95. The equation of 2FI model for the static modulus of elasticity in terms of coded factors was as follows:

$$\begin{aligned}
 \text{Modulus of Elasticity (GPa)} = & 17.46 - 3.63 * A - 0.27 * B + 0.20 * C[1] + 2.10 * C[2] \\
 & - 0.43 * C[3] + 0.29 * D + 0.024 * AB + 0.81 * AC[1] \\
 & + 2.00 * AC[2] - 1.20 * AC[3] + 0.020 * AD + 2.21 * BC[1] \\
 & + 0.22 * BC[2] - 0.29 * BC[3] - 0.28 * BD \\
 & + 0.28 * C[1]D - 0.055 * C[2]D + 0.59 * C[3]D
 \end{aligned} \quad (5.3)$$

The coded factors were used in the above equation. The values for coded factors were listed in Tables 5.7-5.10. The static modulus of elasticity values could be calculated with the above equation by using the coded values for each factor. The predicted and actual values of the static modulus of elasticity are seen in Table 5.14, and graphed in Figure 5.16; and predicted static modulus of elasticity values are demonstrated in Figures 5.17-5.24. The R-squared value of 0.95 shows that 2FI model is good enough for the prediction of the static modulus of elasticity. The results of the ANOVA for response surface 2FI model are presented in Table 5.15. As seen from ANOVA table the model terms A, C, AC, and BC are significant, because the Prob>F values are less than 0.05. This means that water/cement ratio (A) and the temperature (C) have important effects on the static modulus of elasticity of concretes produced in this study. For the static modulus of elasticity response, the relations between the water/cement and the replacement ratios are given in Figures 5.25-5.26; the interaction graphs of water/cement ratio and temperature are seen in Figure 5.27; the interaction graphs of replacement ratio and temperature are seen in Figure 5.28; and the interaction graphs of temperature and age are presented in Figure 5.29.

Table 5.13. Static modulus of elasticity values of second stage natural & recycled aggregate concretes

Run	w/c	RR (%)	T (°C)	Age (days)	E ₁ (Gpa)	E ₂ (Gpa)	E ₃ (Gpa)	¹ $\bar{E}$ (Gpa)	σ	CV
1	0.35	0	22	90	19.4	18.0	20.7	19.4	1.11	5.7%
2	0.35	0	105	28	22.9	22.1	17.9	21.0	2.21	10.6%
3	0.35	0	105	28	15.4	19.2	23.5	19.4	3.28	17.0%
4	0.35	0	175	28	19.2	26.1	22.5	22.6	2.84	12.5%
5	0.35	0	250	28	26.2	21.3	23.8	23.8	2.02	8.5%
6	0.35	50	22	28	17.0	18.2	19.3	18.2	0.95	5.2%
7	0.35	50	105	90	21.0	21.7	24.1	22.3	1.34	6.0%
8	0.35	100	22	90	23.9	22.6	25.2	23.9	1.04	4.3%
9	0.35	100	105	28	23.5	21.2	22.3	22.3	0.93	4.2%
10	0.35	100	175	90	21.4	19.1	23.1	21.2	1.66	7.8%
11	0.35	100	250	90	18.2	16.0	17.5	17.2	0.91	5.3%
12	0.4125	25	250	90	19.4	19.2	19.4	19.3	0.09	0.5%
13	0.475	0	175	90	18.8	17.4	17.8	18.0	0.62	3.4%
14	0.475	0	175	90	18.1	19.2	19.5	19.0	0.60	3.2%
15	0.475	25	22	28	17.8	19.6	15.8	17.7	1.55	8.8%
16	0.475	50	250	28	16.0	14.7	16.6	15.7	0.80	5.1%
17	0.475	100	105	90	18.9	16.2	18.3	17.8	1.16	6.5%
18	0.475	100	175	28	16.3	14.5	16.0	15.6	0.75	4.8%
19	0.5375	25	175	28	10.5	14.4	13.6	12.8	1.68	13.1%
20	0.5375	50	22	90	15.1	16.4	15.5	15.6	0.52	3.3%
21	0.5375	75	250	90	12.0	10.7	12.2	11.6	0.66	5.7%
22	0.60	0	22	28	14.5	11.4	10.7	12.2	1.66	13.6%
23	0.60	0	105	90	17.2	18.9	19.7	18.6	1.04	5.6%
24	0.60	0	105	90	17.7	18.5	21.3	19.2	1.58	8.2%
25	0.60	0	250	90	12.5	13.1	10.9	12.1	0.93	7.6%
26	0.60	50	105	28	16.7	19.0	14.6	16.8	1.79	10.7%
27	0.60	50	105	28	19.1	12.7	24.1	18.6	4.70	25.2%
28	0.60	100	22	28	15.7	17.0	16.3	16.3	0.54	3.3%
29	0.60	100	175	90	11.5	14.8	11.1	12.5	1.66	13.3%
30	0.60	100	175	90	12.1	12.7	15.0	13.3	1.26	9.5%
31	0.60	100	250	28	9.2	9.1	8.5	8.9	0.30	3.4%

¹ $\bar{E}$: Mean of E₁, E₂ and E₃, E: static modulus of elasticity

Table 5.14. Actual and predicted values of static modulus of elasticity

Run	w/c	RR (%)	T (°C)	Age (days)	Actual value (GPa)	Predicted value (GPa)	Residual (GPa)	Error
1	0.35	0	22	90	19.4	19.4	0.0	0.1%
2	0.35	0	105	28	21.0	20.8	0.2	0.9%
3	0.35	0	105	28	19.4	20.8	-1.4	7.3%
4	0.35	0	175	28	22.6	21.3	1.3	5.8%
5	0.35	0	250	28	23.8	23.5	0.2	1.0%
6	0.35	50	22	28	18.2	19.9	-1.8	9.7%
7	0.35	50	105	90	22.3	21.4	0.9	3.9%
8	0.35	100	22	90	23.9	22.7	1.2	5.1%
9	0.35	100	105	28	22.3	21.2	1.2	5.2%
10	0.35	100	175	90	21.2	21.9	-0.7	3.1%
11	0.35	100	250	90	17.2	17.6	-0.4	2.2%
12	0.4125	25	250	90	19.3	19.0	0.3	1.6%
13	0.475	0	175	90	18.0	18.8	-0.7	4.1%
14	0.475	0	175	90	19.0	18.8	0.2	1.1%
15	0.475	25	22	28	17.7	16.0	1.7	9.8%
16	0.475	50	250	28	15.7	16.1	-0.4	2.4%
17	0.475	100	105	90	17.8	19.5	-1.6	9.1%
18	0.475	100	175	28	15.6	15.9	-0.3	1.7%
19	0.5375	25	175	28	12.8	13.9	-1.0	8.1%
20	0.5375	50	22	90	15.6	16.8	-1.2	7.7%
21	0.5375	75	250	90	11.6	11.1	0.5	4.1%
22	0.60	0	22	28	12.2	12.0	0.2	1.6%
23	0.60	0	105	90	18.6	18.5	0.1	0.4%
24	0.60	0	105	90	19.2	18.5	0.7	3.5%
25	0.60	0	250	90	12.1	12.5	-0.4	3.1%
26	0.60	50	105	28	16.8	17.7	-0.9	5.4%
27	0.60	50	105	28	18.6	17.7	1.0	5.2%
28	0.60	100	22	28	16.3	16.5	-0.2	1.0%
29	0.60	100	175	90	12.5	12.3	0.2	1.4%
30	0.60	100	175	90	13.3	12.3	1.0	7.5%
31	0.60	100	250	28	8.9	8.7	0.2	1.7%

The effect of water/cement ratio was very much for the static modulus of elasticity as reported by analysis of variance. Also it is seen from Equation 5.3 that the coefficient of water/cement ratio factor, which is 3.63, was the highest coefficient. This highest coefficient and analysis of variance showed that the key factor for the static modulus of elasticity was the water/cement ratio. From 3D surface view figures it is plainly seen that static modulus of elasticity decreases as water/cement ratio increases at all ages, exposed temperatures and replacement ratios, as shown in Figures 5.17-5.24. Twenty eight days test results showed that at zero replacement ratio, when water/cement ratio decreased from 0.60 to 0.35 the increase in static modulus of elasticity was 48 percent at 22 °C, 19 percent at 105 °C, 84 percent at 175 °C, and 82 percent at 250 °C; whereas the increase was 41 percent at 22 °C, 18 percent at 105 °C, 69 percent at 175 °C, and 84 percent at 250 °C when 90 days test results were compared. When the replacement ratio was 100 percent, 28 days test results showed that the static modulus of elasticity increased 34 percent at 22 °C, 18 percent at 105 °C, 87 percent at 175 °C, and 120 percent at 250 °C as the water/cement ratio decreased from 0.60 to 0.35; and the increase was 32 percent at 22 °C, 18 percent at 105 °C, 78 percent at 175 °C, and 145 percent at 250 °C when the testing age was 90 days.

As seen from the analysis of variance table as listed in Table 5.15 the replacement ratio (B) together with temperature (C), that is the interaction of BC, have an important role on static modulus of elasticity while the sole function of replacement ratio factor was not a significant factor. As shown in Figures 5.25-5.26, the effect of replacement ratio on static modulus of elasticity is seen from the interaction graphs of replacement ratio at 28 and 90 days, and at every water/cement ratio ranging from 0.35 to 0.60 and at all exposed temperatures. The positive effect of replacement ratio factor on static modulus of elasticity was observed when concretes were tested at room temperature. At w/c ratio of 0.35, when the replacement ratio increased from zero to 100 percent, the increase in static modulus of elasticity was 25 and 17 percent, respectively, at 28 and 90 days and at 22 °C, two percent increase and three percent decrease was calculated respectively in static modulus of elasticity after exposure to 105 °C at 28 and 90 days; the decrease in static modulus of elasticity after exposure to 175 °C was 3 and 8 percent at 28 and 90 days, respectively; and 22 and 31 percent decrease at 28 and 90 days was respectively calculated in static modulus of elasticity after exposure to 250 °C. At w/c ratio of 0.60, when the replacement ratio increased from zero to 100 percent, the increase in static modulus of elasticity at 22 °C was

37 and 24 percent at 28 and 90 days, respectively; three percent increase and four percent decrease at 28 and 90 days was calculated in static modulus of elasticity after exposure to 105 °C; the decrease in static modulus of elasticity after exposure to 175 °C was 5 and 13 percent at 28 and 90 days, respectively; and 48 and 74 percent decrease at 28 and 90 days was calculated respectively in static modulus of elasticity after exposure to 250 °C.

As described in analysis of variance report and shown in temperature interaction figures the effect of temperature factor was significant. After exposure to 105 °C the static modulus of elasticity increased (with respect to 22 °C) 17 and 12 percent at 28 and 90 days respectively when the w/c and the replacement ratios were 0.35 and zero percent. When the w/c and the replacement ratios were 0.35 and 100 percent the decrease was 4 and 8 percent at 28 and 90 days, respectively. On the other hand, 45 and 34 percent increase was calculated when the w/c and the replacement ratios were 0.60 and zero percent, and an increase was calculated as 9 and 4 percent when the w/c and the replacement ratios were 0.60 and 100 percent. After exposure to 175 °C the static modulus of elasticity increased (with respect to 22 °C) 20 and 21 percent at 28 and 90 days, respectively, when the w/c and replacement ratios were 0.35 and zero percent. When the w/c and replacement ratios were 0.35 and 100 percent the decrease was 7 and 4 percent at 28 and 90 days, respectively. On the other hand 4 percent increase and one percent decrease was calculated when the w/c and replacement ratios were 0.60 and zero percent, and a decrease was calculated in amount of 49 and 39 percent when the w/c and replacement ratios were 0.60 and 100 percent. After exposure to 250 °C the static modulus of elasticity increased 33 and 19 percent at 28 and 90 days respectively when the w/c and replacement ratios were 0.35 and zero percent, when the w/c and replacement ratios were 0.35 and 100 percent the decrease was 15 and 29 percent at 28 and 90 days. On the other hand 8 percent increase and 10 percent decrease was calculated when the w/c and replacement ratios were 0.60 and zero percent, and a decrease was calculated in amount of 89 and 138 percent when the w/c and replacement ratios were 0.60 and 100 percent.

The coefficient belonging to single D factor was 0.29 in Equation 5.3 indicating that this small coefficient effect on the static modulus of elasticity of natural and recycled aggregate concretes was not too much. Test results and interaction figures of age showed that generally age has a positive effect on the static modulus of elasticity. At 22 °C and

0.35 w/c ratio, the increase in static modulus of elasticity from 28 to 90 days was 9 and 2 percent at zero and 100 percent replacement ratio, respectively; it was 15 and 4 percent at zero and 100 percent replacement ratio and 0.60 w/c ratios. After exposure to 105 °C the increase in static modulus of elasticity was 5 percent at zero percent replacement ratios and at 0.35 w/c ratio and it was 6 percent at zero percent replacement ratio and 0.60 w/c ratio. After exposure to 175 °C the increase in static modulus of elasticity was 11 and 6 percent at 0.35 w/c ratio and at zero and 100 percent replacement ratio respectively, and it was 20 and 11 percent respectively at zero and 100 percent replacement and at 0.60 w/c ratios. After exposure to 250 °C there was a decrease in static modulus of elasticity by increased age from 28 to 90 days, the reductions in static modulus of elasticity was two percent at zero percent replacement ratio and at 0.35 w/c ratio, three percent at zero percent replacement ratio and 0.60 w/c ratio, 9 percent at 100 percent replacement ratio and 0.35 w/c ratio, and 22 percent at 100 percent replacement ratio and 0.60 w/c ratio.

Based on the experimental test results and comparisons of the above paragraphs, the following conclusions for the static modulus of elasticity could be drawn:

1. The static modulus of elasticity of the concretes produced with recycled aggregates was 17-37 percent higher than that of natural aggregate concrete, when the concrete specimens were tested at room temperature. After exposure to 105, 175, and 250 °C, the static modulus of elasticity for recycled aggregates was calculated to be in the range 97-103, 88-97 and 57-82 percent, respectively of the natural aggregate concrete. Test results were not same as reported in literature. The reduction in the modulus of elasticity for recycled aggregate concrete was reported in the literature [33, 56]. It should be noted that the amount of decrease in modulus of elasticity is related to the parameters such as the type of concrete used for making the recycled aggregate, water/cement ratio, and the moisture condition of the recycled aggregate.
2. The characteristics of stress strain curves at compression showed similar trends for concretes with natural and recycled aggregate. Despite the same trend and same compressive strength values, static modulus of elasticity of recycled aggregate concretes tested at room temperature gave higher values. This means that lower strains were measured for the recycled aggregate concretes under equal loads. One reason for this could be the texture of our recycled aggregates. Therefore, more

experimental data is required in this subject in applications of recycled aggregate concrete where the modulus of elasticity and the stress strain behaviour are the critical parameters before conclusive results could be drawn.

3. Generally, temperature increases up to 250 °C increased the static modulus of elasticity of natural aggregate concretes, the maximum decrease with temperature increases observed in amount of 10 percent (with respect to the tests performed at room temperature) when natural aggregate concretes produced with water/cement ratio of 0.60 and tested after 250 °C exposure. On the other hand, temperature increases has negative effects on the static modulus of elasticity of recycled aggregate concretes. Recycled aggregate concrete produced with 0.60 w/c ratio lost 58 percent of their static modulus of elasticity at 22 °C after being exposed to 250 °C. This result showed that recycled aggregate concretes were affected more than natural aggregate concretes under elevated temperatures, because the variation of material properties of the ingredient phases and interfaces in recycled aggregate concretes were higher than the natural aggregate concretes.
4. Concretes produced with either natural or recycled aggregates in the temperature range of 22 to 250 °C showed both loss and gain in the static modulus of elasticity. Loss of the elasticity was attributed to the thermal swelling of the physically bound water, which caused disjoint pressures, especially in recycled aggregate concretes. Gain of the elasticity, observed especially in natural aggregate concretes, was attributed to the greater Van der Waals forces as a result of the cement gel layers moving closer to each other during heating.
5. ANOVA results of compressive strength and static modulus of elasticity seemed similar, they were both affected from the same factors.
6. Finally, despite the negative effects of temperature changes on recycled aggregate concrete, concretes that will be subjected to temperatures as high as 250 °C could be produced carefully with recycled aggregates by lowering the water/cement ratio and by performing adequate tests if the static modulus of elasticity is in concern. Because, it is clear from the results that the effect of replacement ratio factor on static modulus of elasticity was lower than the water/cement ratio factor. The raise of static modulus of elasticity value of natural and recycled aggregate concretes were 19-84 percent and 18-145 percent, respectively when the w/c ratio was decreased from 0.60 to 0.35.

Table 5.15. Results of ANOVA for static modulus of elasticity response surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	183.41	1	183.41	92.59	<0.0001
B	0.98	1	0.98	0.50	0.4947
C	59.08	3	19.69	9.94	0.0014
D	2.85	1	2.85	1.44	0.2537
AB	0.01	1	0.01	0.00	0.9555
AC	38.51	3	12.84	6.48	0.0074
AD	0.01	1	0.01	0.00	0.9578
BC	37.93	3	12.64	6.38	0.0078
BD	1.28	1	1.28	0.65	0.4371
CD	6.80	3	2.27	1.14	0.3709

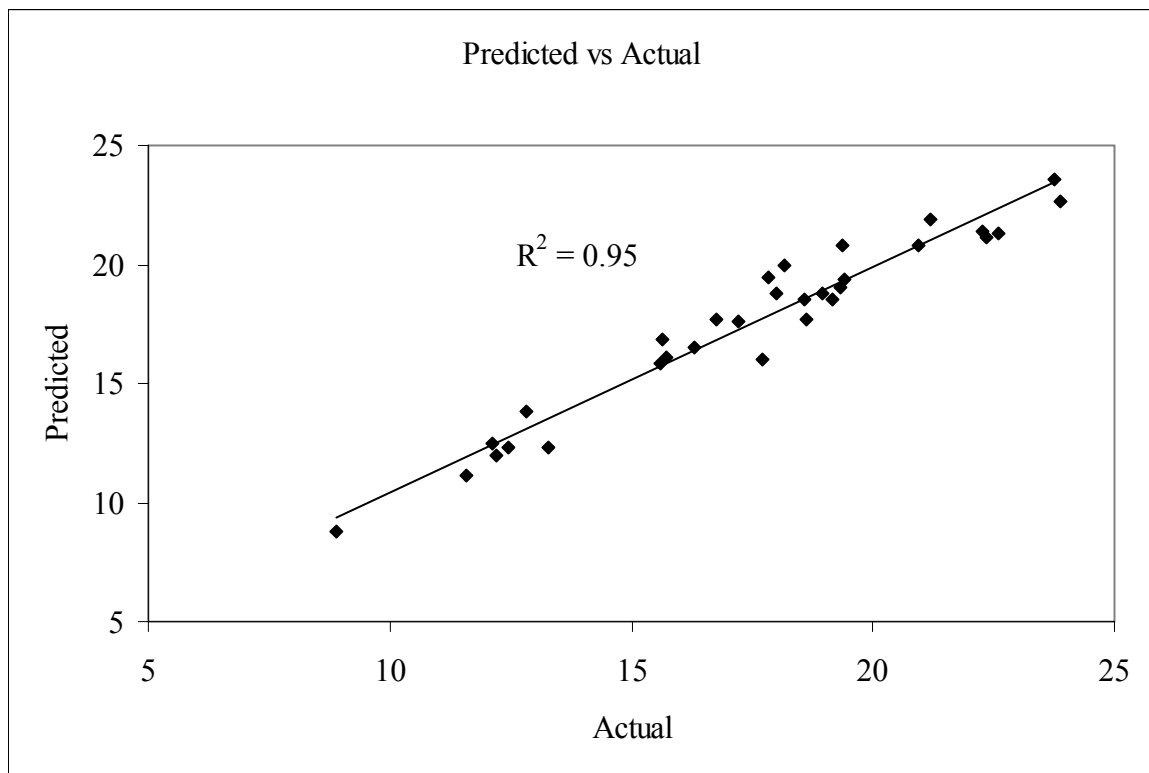


Figure 5.16. Predicted versus actual values of static modulus of elasticity (GPa)

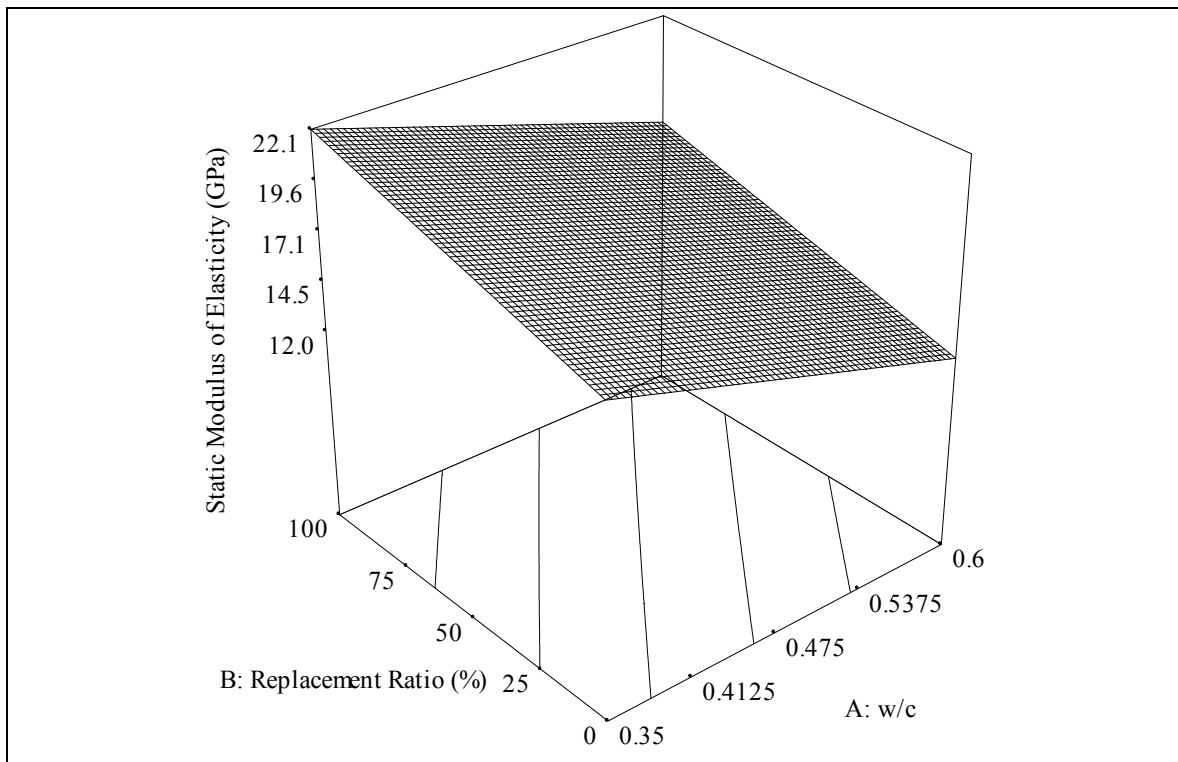


Figure 5.17. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)

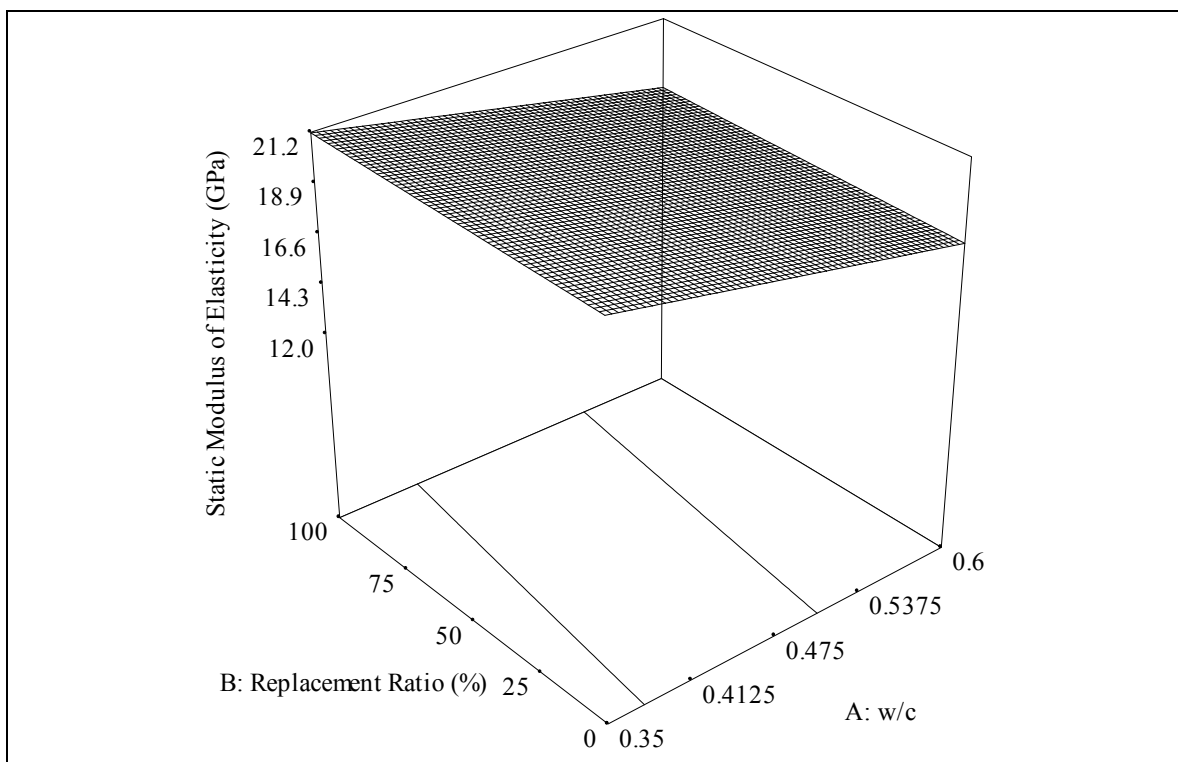


Figure 5.18. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)

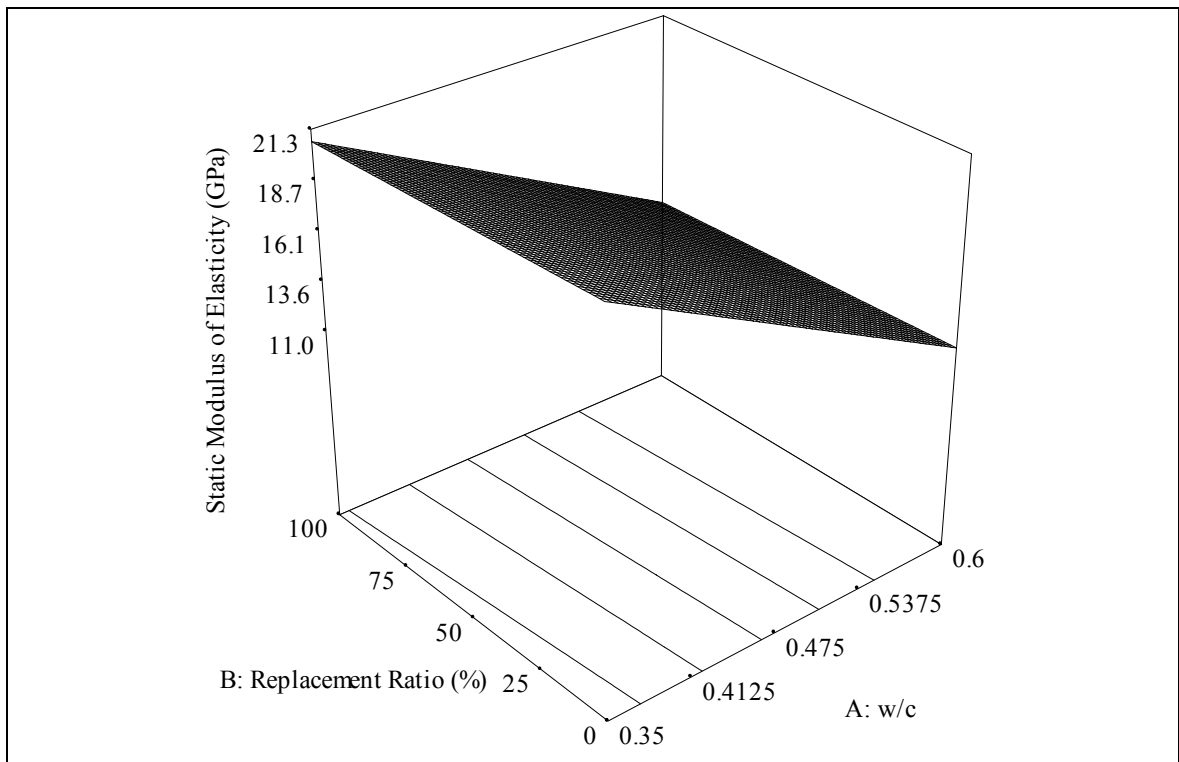


Figure 5.19. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)

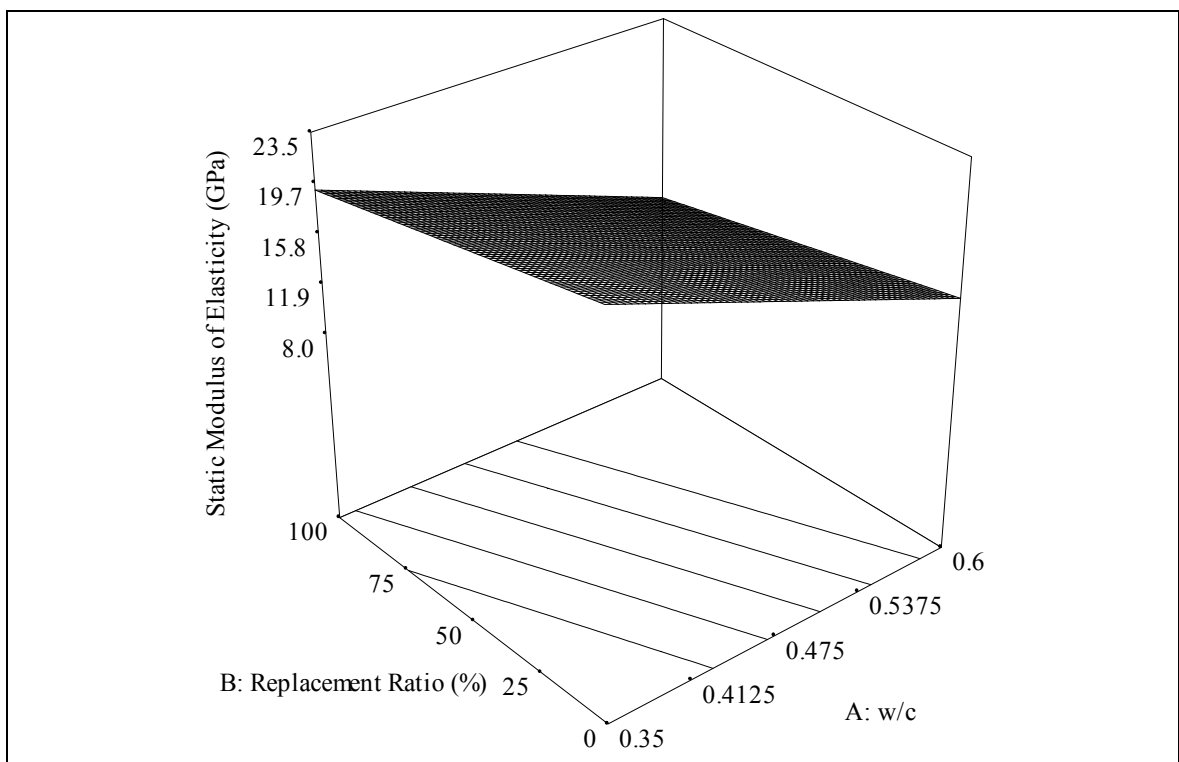


Figure 5.20. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)

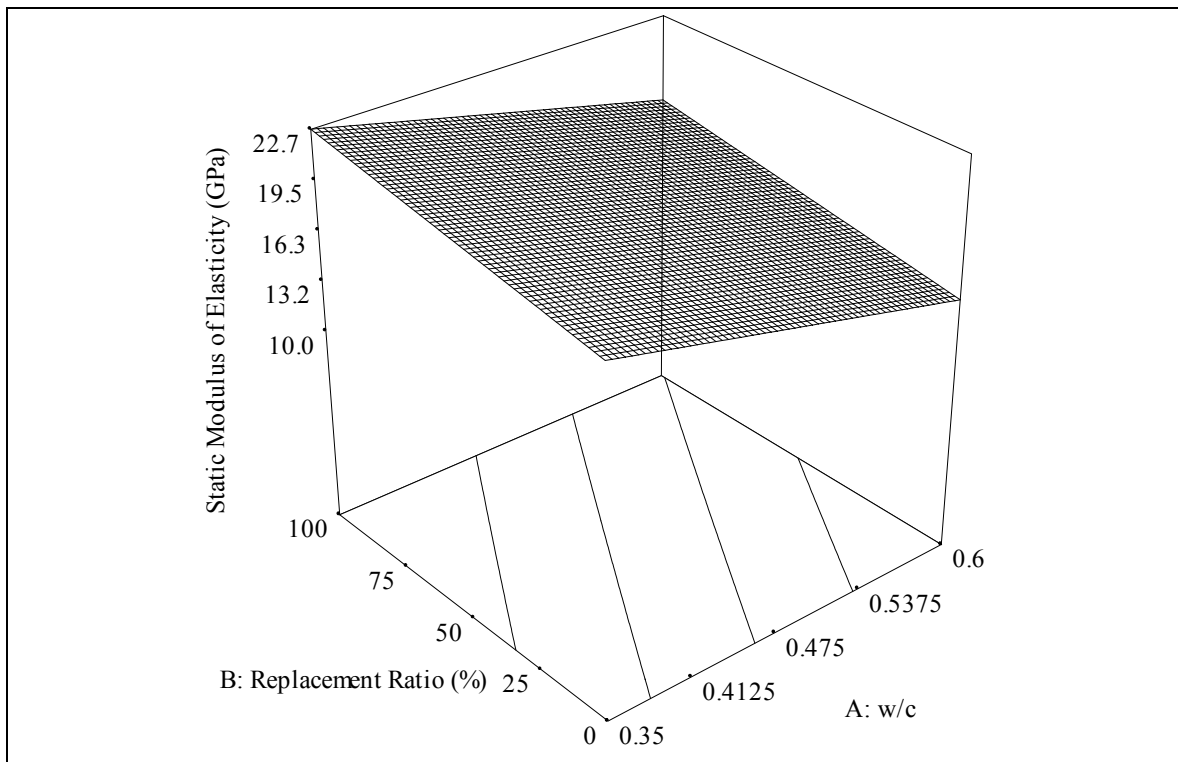


Figure 5.21. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)

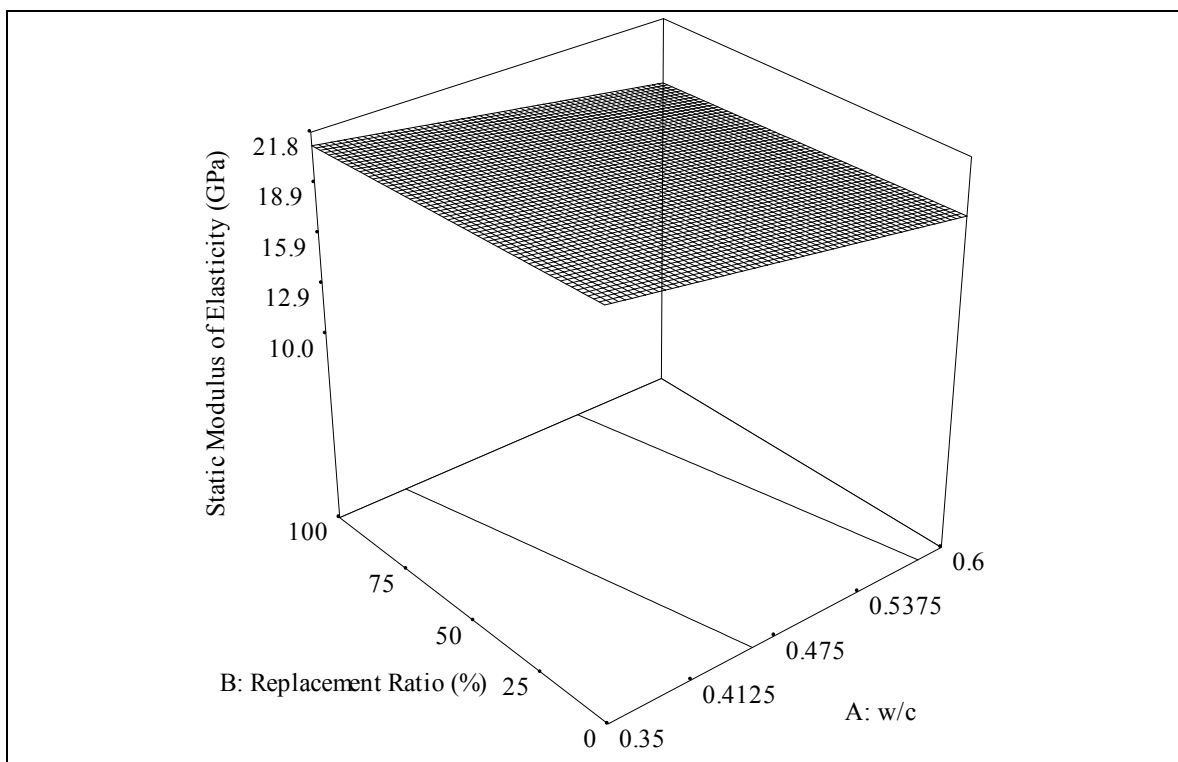


Figure 5.22. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)

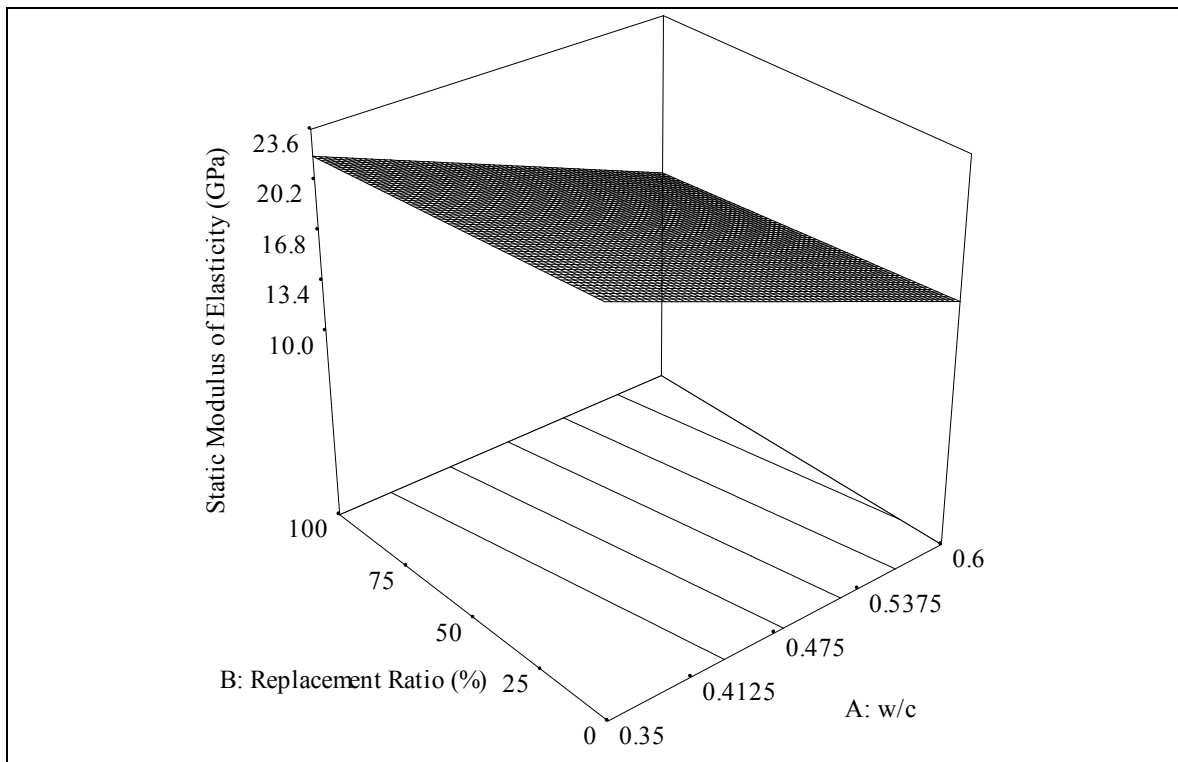


Figure 5.23. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)

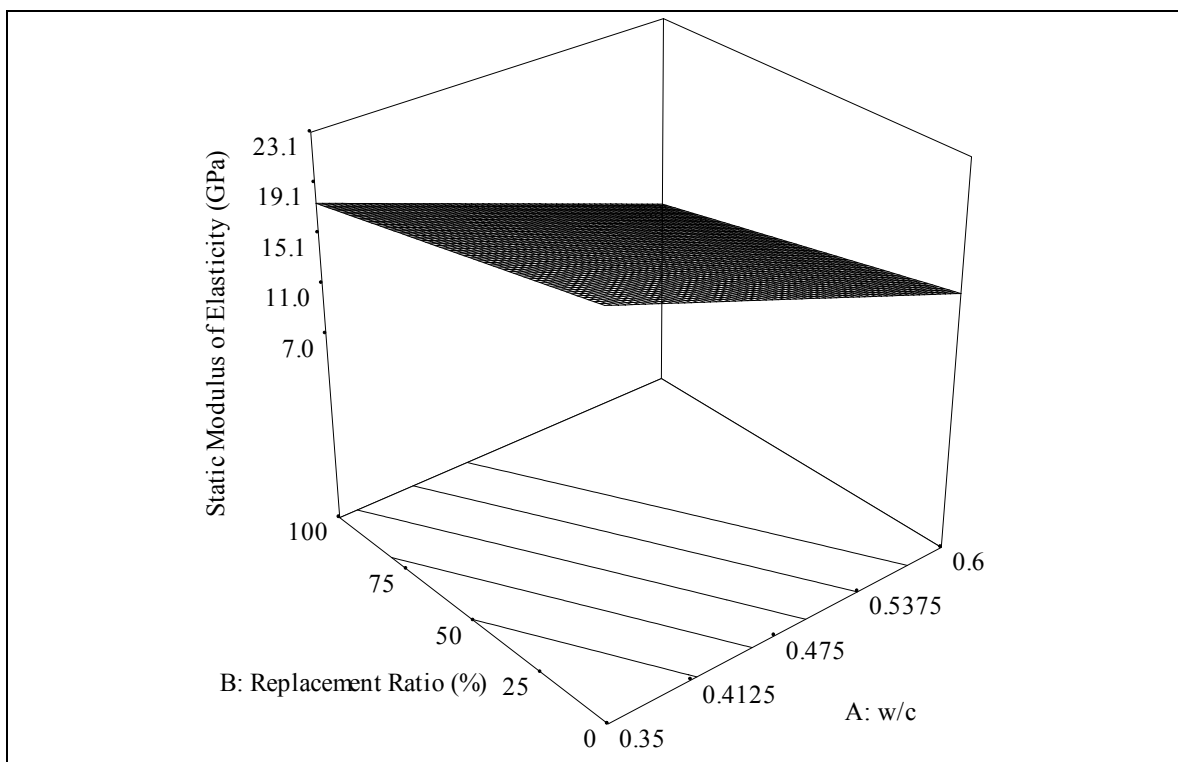


Figure 5.24. 3D surface view of static modulus of elasticity of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)

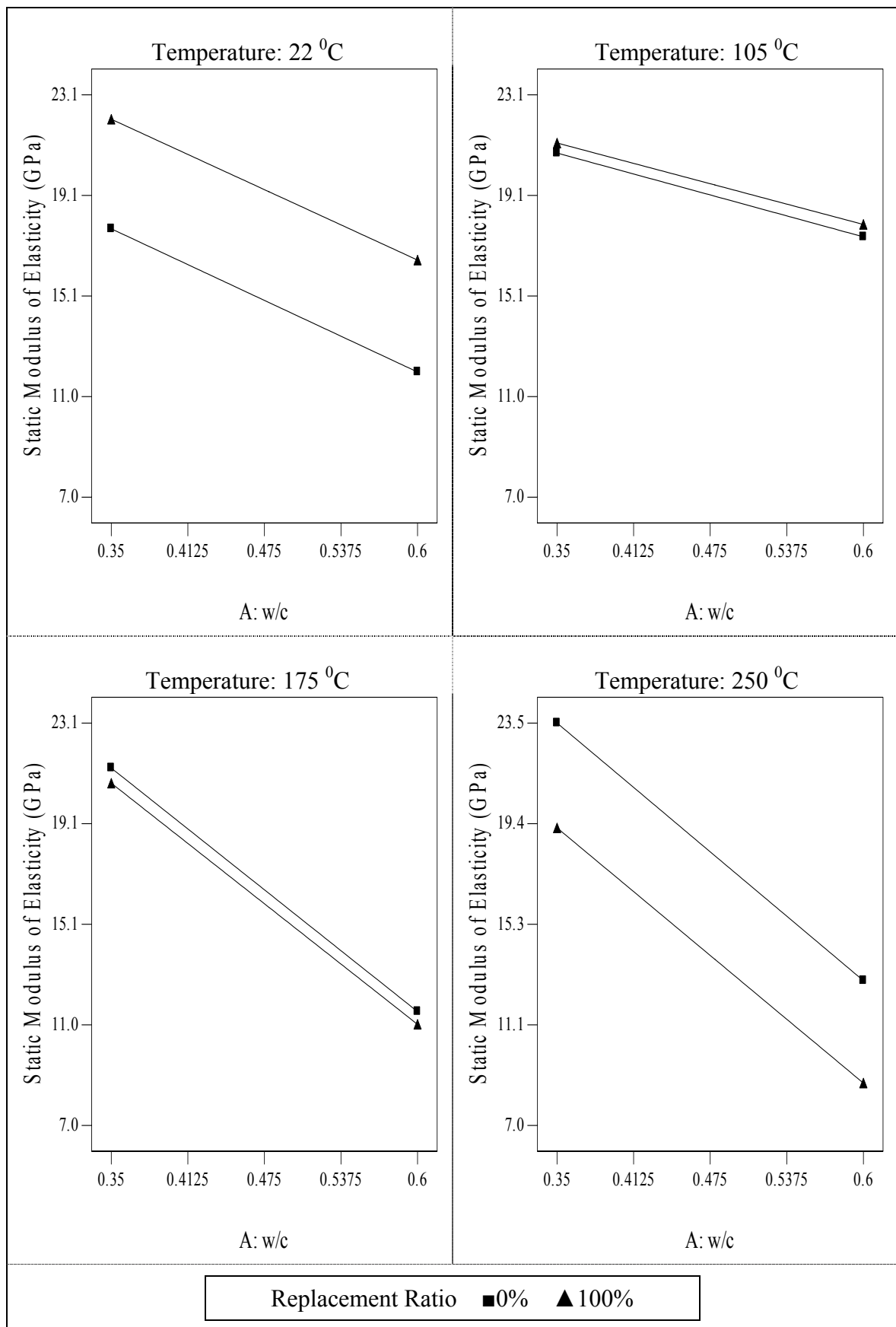


Figure 5.25. Interaction graph of w/c and replacement ratio at 28 days

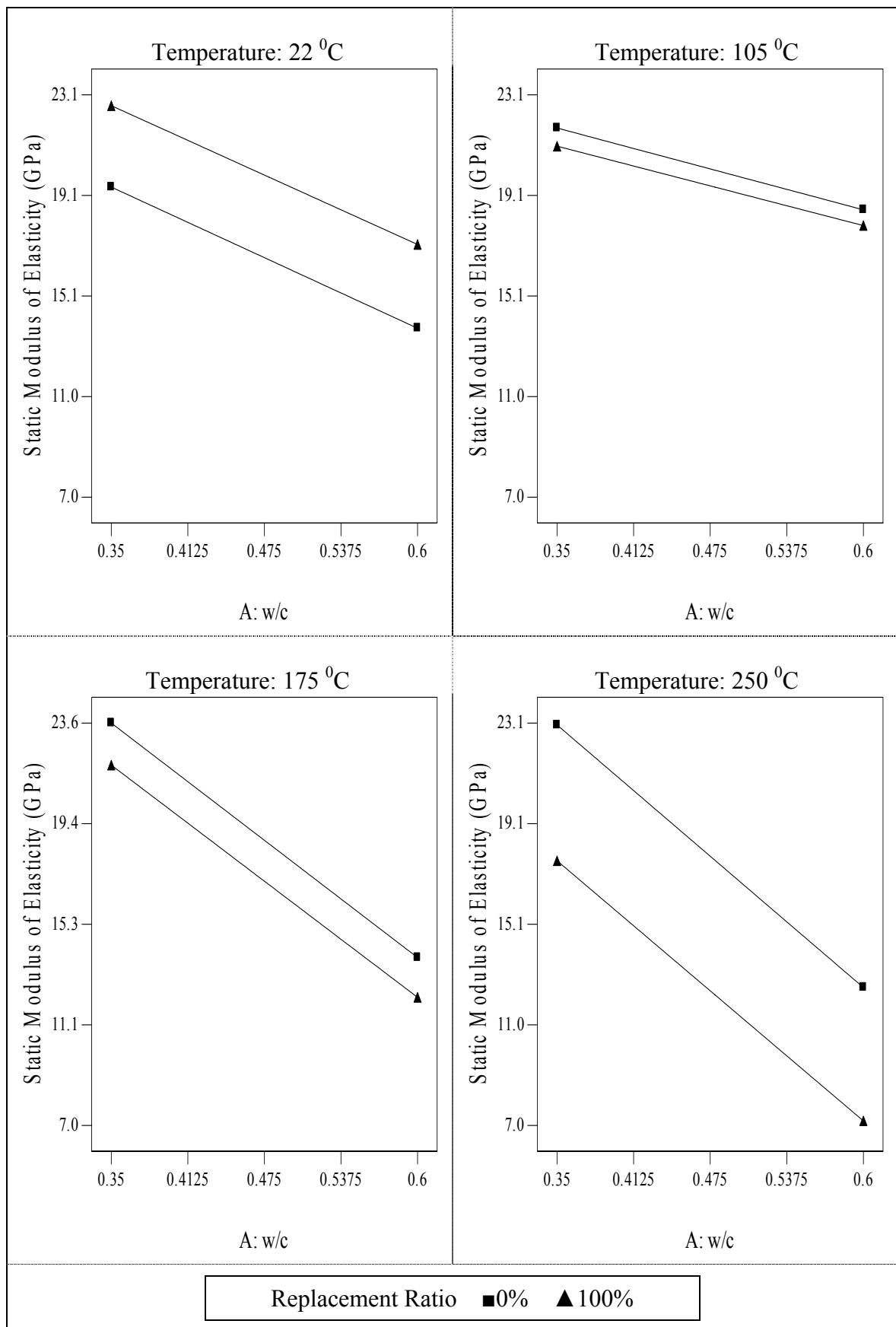


Figure 5.26. Interaction graph of w/c and replacement ratio at 90 days

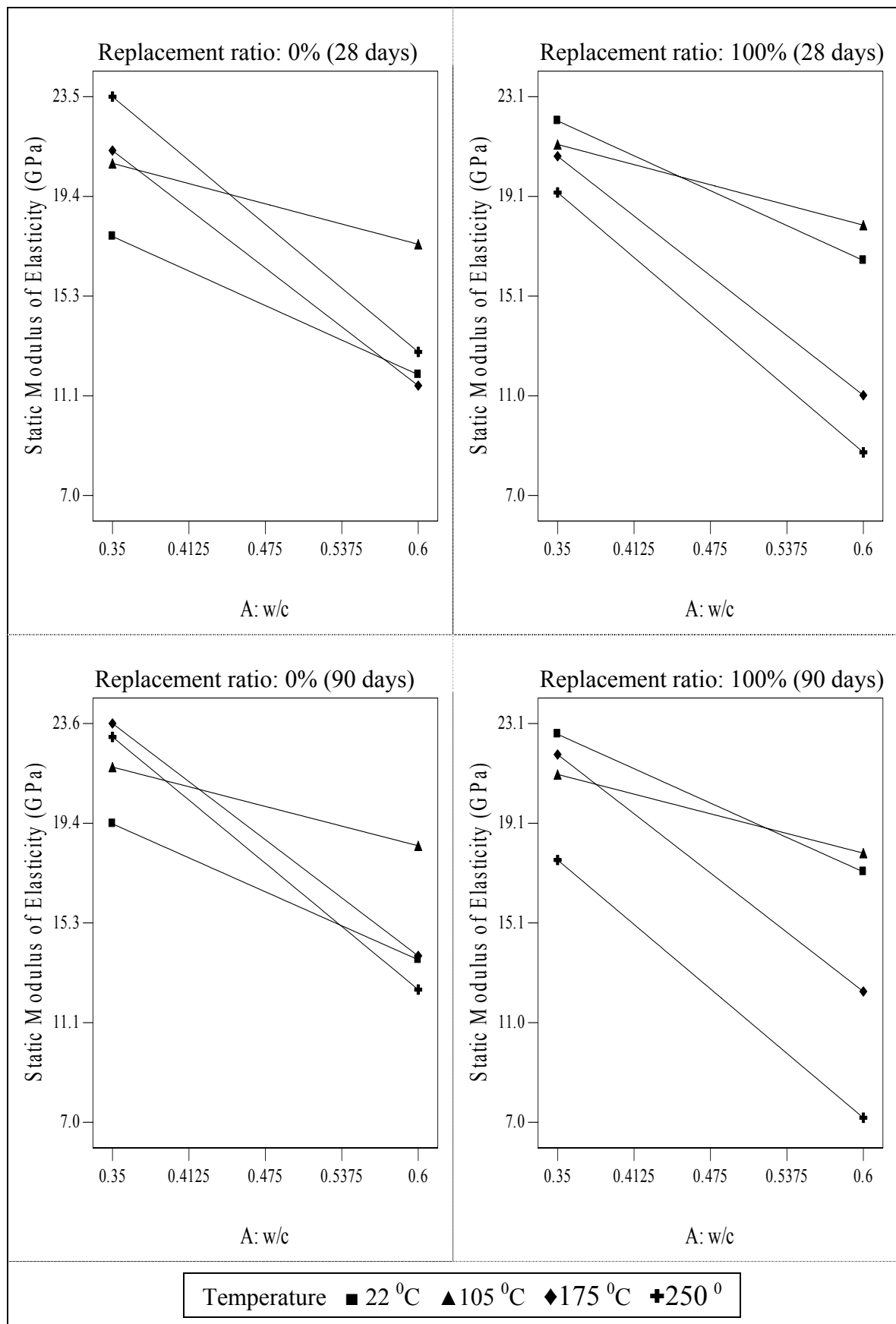


Figure 5.27. Interaction graph of w/c and temperature

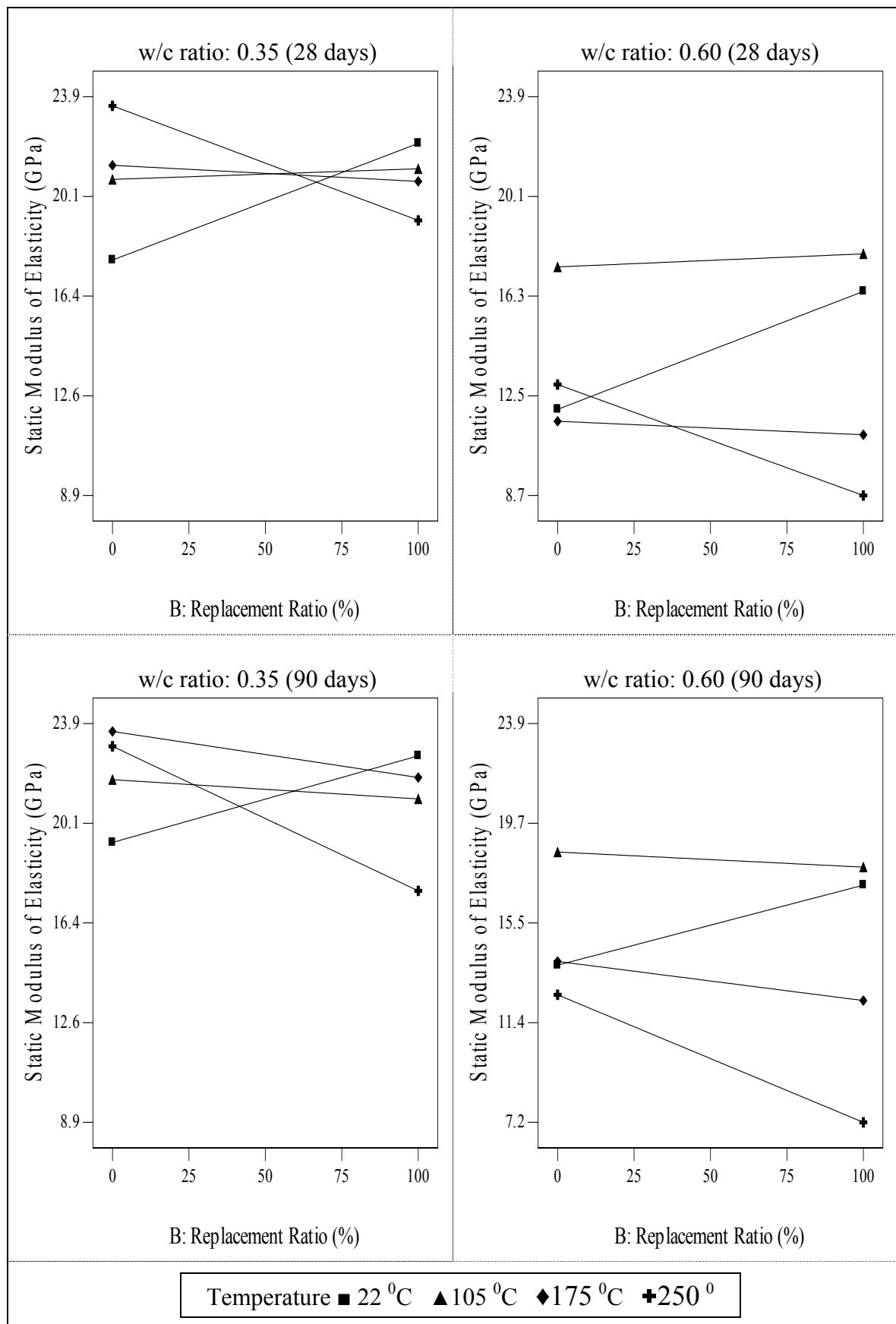


Figure 5.28. Interaction graph of replacement ratio and temperature

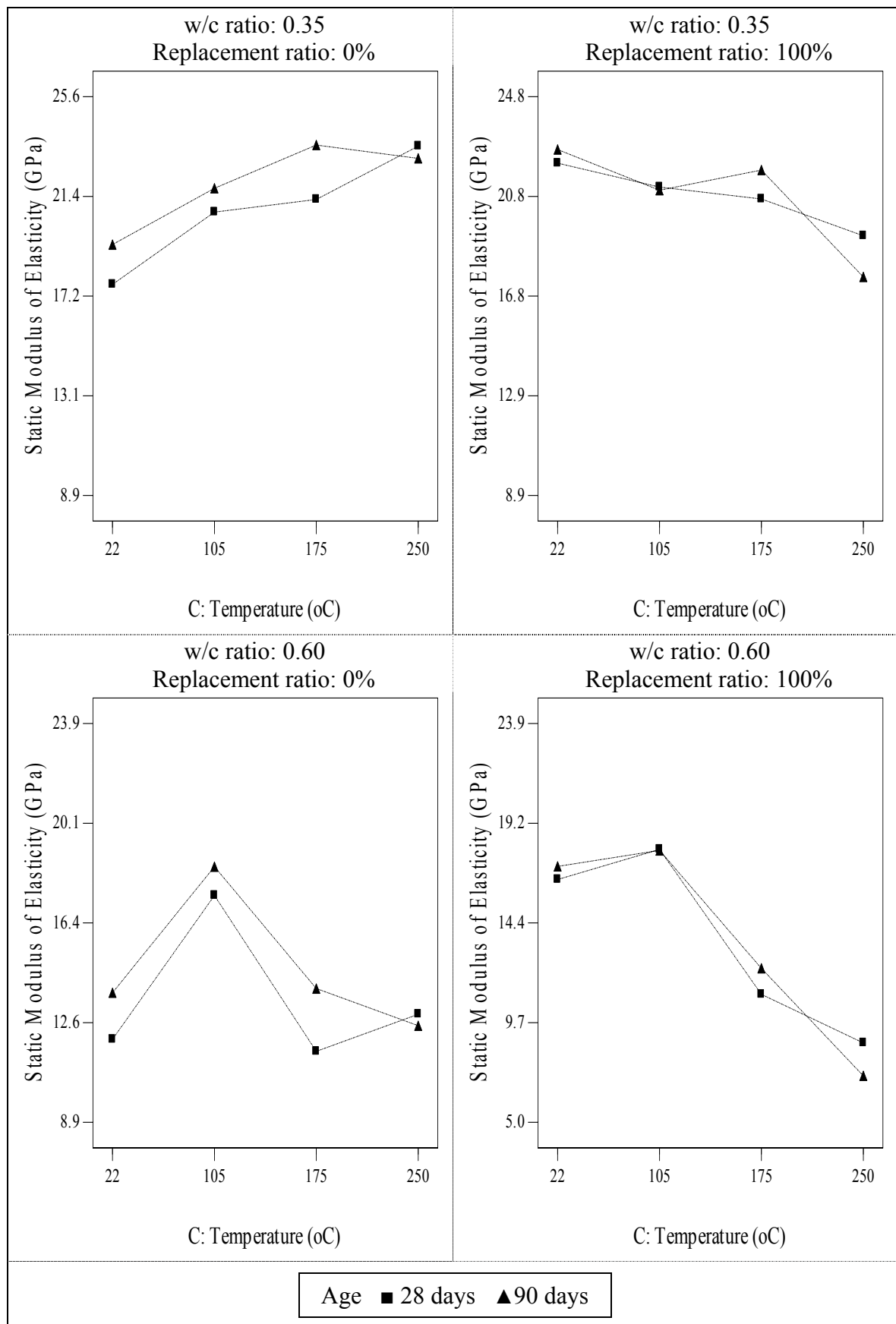


Figure 5.29. Interaction graph of temperature and age

5.3.3. Splitting Tensile Strength

The splitting tensile strength test results of second stage concretes, mixes 1-31, produced with various water/cement and replacement ratios and tested at 28 and 90 days at room temperature and after exposure to elevated temperatures are given in Table 5.16. 2FI model was used for the analysis of splitting tensile strength results. The standard deviation and the R-squared values of the model were 0.27 and 0.95. The equation of 2FI model for the splitting tensile strength in terms of coded factors was as follows:

$$\begin{aligned}
 \text{Splitting Tensile Strength (MPa)} = & 3.14 - 0.55 * A - 0.42 * B - 0.016 * C[1] - 0.034 * C[2] \\
 & - 0.18 * C[3] + 0.098 * D + 0.10 * AB + 0.16 * AC[1] \\
 & + 0.021 * AC[2] - 0.17 * AC[3] + 0.026 * AD + 0.20 * BC[1] \\
 & + 0.33 * BC[2] - 0.12 * BC[3] + 0.12 * BD \\
 & - 0.021 * C[1]D - 0.082 * C[2]D + 0.28 * C[3]D
 \end{aligned} \quad (5.4)$$

The coded factors were used in the above equation. The values for coded factors were listed in Tables 5.7-5.10. The splitting tensile strength values could be calculated with the above equation by using the coded values for each factor. The predicted and actual values of the splitting tensile strength are seen in Table 5.17, and graphed in Figure 5.30; and predicted splitting tensile strength values are demonstrated in Figures 5.31-5.38. The R-squared value of 0.95 shows that 2FI model is good enough for the splitting tensile strength prediction. The results of the ANOVA for response surface 2FI model are presented in Table 5.18. As seen from ANOVA table the model terms A, B, and BC are significant, because the Prob>F values are less than 0.05. This means that the water/cement ratio (A) and the replacement ratio (B) have important effects on splitting tensile strength. For the splitting tensile strength response, the relations between water/cement and replacement ratios are given in Figures 5.39-5.40; the interaction graphs of water/cement ratio and temperature are seen in Figure 5.41; the interaction graphs of replacement ratio and temperature are seen in Figure 5.42; and the interaction graphs of temperature and age are presented in Figure 5.43.

Table 5.16. Splitting tensile strength values of second stage natural & recycled aggregate concretes

Run	w/c	RR (%)	T (°C)	Age (days)	f_{t1} (MPa)	f_{t2} (MPa)	f_{t3} (MPa)	$\bar{f}_t$ (MPa)	σ	CV
1	0.35	0	22	90	3.99	3.92	2.82	3.58	0.54	15.0%
2	0.35	0	105	28	4.13	3.85	3.98	3.99	0.11	2.9%
3	0.35	0	105	28	3.29	4.52	3.79	3.87	0.50	13.0%
4	0.35	0	175	28	2.94	4.20	4.58	3.91	0.70	17.9%
5	0.35	0	250	28	5.06	5.47	5.26	5.26	0.17	3.2%
6	0.35	50	22	28	4.05	2.66	4.05	3.59	0.66	18.3%
7	0.35	50	105	90	4.01	4.01	3.65	3.89	0.17	4.4%
8	0.35	100	22	90	3.34	2.58	4.13	3.35	0.63	18.9%
9	0.35	100	105	28	3.13	2.73	3.51	3.13	0.32	10.2%
10	0.35	100	175	90	3.87	3.14	3.13	3.38	0.34	10.2%
11	0.35	100	250	90	2.48	3.36	3.14	2.99	0.37	12.5%
12	0.4125	25	250	90	2.57	3.44	5.03	3.68	1.02	27.8%
13	0.475	0	175	90	4.13	3.89	3.38	3.80	0.31	8.2%
14	0.475	0	175	90	4.14	3.46	4.45	4.02	0.41	10.2%
15	0.475	25	22	28	3.04	3.32	3.37	3.24	0.15	4.6%
16	0.475	50	250	28	3.34	3.74	2.91	3.33	0.34	10.2%
17	0.475	100	105	90	3.24	2.92	3.13	3.09	0.13	4.3%
18	0.475	100	175	28	2.31	2.05	2.50	2.29	0.18	8.0%
19	0.5375	25	175	28	2.03	2.50	2.45	2.33	0.21	9.0%
20	0.5375	50	22	90	2.85	3.48	3.30	3.21	0.26	8.1%
21	0.5375	75	250	90	3.14	2.49	3.43	3.02	0.39	13.0%
22	0.60	0	22	28	3.28	2.59	2.73	2.87	0.29	10.3%
23	0.60	0	105	90	2.03	2.91	1.96	2.30	0.43	18.7%
24	0.60	0	105	90	2.33	2.28	2.74	2.45	0.21	8.4%
25	0.60	0	250	90	3.34	3.51	3.15	3.33	0.14	4.3%
26	0.60	50	105	28	2.69	2.76	2.12	2.52	0.29	11.5%
27	0.60	50	105	28	2.93	2.62	2.82	2.79	0.13	4.6%
28	0.60	100	22	28	2.21	2.13	2.36	2.23	0.09	4.2%
29	0.60	100	175	90	2.12	2.33	2.13	2.19	0.10	4.5%
30	0.60	100	175	90	2.56	2.25	1.89	2.23	0.28	12.4%
31	0.60	100	250	28	2.09	1.81	1.92	1.94	0.12	6.0%

f_{t1} , f_{t2} and f_{t3} : splitting tensile strength of specimens no 1, 2 and 3

$\bar{f}_t$: mean of f_{t1} , f_{t2} and f_{t3}

Table 5.17. Actual and predicted values of splitting tensile strength

Run	w/c	RR (%)	T (°C)	Age (days)	Actual value (MPa)	Predicted value (MPa)	Residual (MPa)	Error
1	0.35	0	22	90	3.58	3.77	-0.19	5.3%
2	0.35	0	105	28	3.99	3.95	0.04	1.0%
3	0.35	0	105	28	3.87	3.95	-0.08	2.1%
4	0.35	0	175	28	3.91	4.09	-0.18	4.6%
5	0.35	0	250	28	5.26	5.06	0.20	3.8%
6	0.35	50	22	28	3.59	3.46	0.13	3.6%
7	0.35	50	105	90	3.89	3.62	0.27	7.0%
8	0.35	100	22	90	3.35	3.35	-0.01	0.2%
9	0.35	100	105	28	3.13	3.33	-0.21	6.6%
10	0.35	100	175	90	3.38	3.50	-0.12	3.4%
11	0.35	100	250	90	2.99	3.02	-0.03	0.9%
12	0.4125	25	250	90	3.68	3.94	-0.26	7.0%
13	0.475	0	175	90	3.80	3.75	0.05	1.3%
14	0.475	0	175	90	4.02	3.75	0.27	6.6%
15	0.475	25	22	28	3.24	3.21	0.03	1.0%
16	0.475	50	250	28	3.33	3.44	-0.11	3.4%
17	0.475	100	105	90	3.09	3.15	-0.05	1.7%
18	0.475	100	175	28	2.29	1.92	0.36	15.9%
19	0.5375	25	175	28	2.33	2.51	-0.18	7.9%
20	0.5375	50	22	90	3.21	3.02	0.19	6.0%
21	0.5375	75	250	90	3.02	2.70	0.32	10.4%
22	0.60	0	22	28	2.87	2.86	0.00	0.1%
23	0.60	0	105	90	2.30	2.48	-0.18	8.0%
24	0.60	0	105	90	2.45	2.48	-0.04	1.4%
25	0.60	0	250	90	3.33	3.36	-0.03	0.8%
26	0.60	50	105	28	2.52	2.53	-0.01	0.4%
27	0.60	50	105	28	2.79	2.53	0.26	9.1%
28	0.60	100	22	28	2.23	2.40	-0.16	7.3%
29	0.60	100	175	90	2.19	2.31	-0.12	5.5%
30	0.60	100	175	90	2.23	2.31	-0.08	3.7%
31	0.60	100	250	28	1.94	2.03	-0.09	4.5%

As seen from Equation 5.4 the biggest coefficient, 0.55, was associated with “A” (water/cement ratio), and this highest coefficient and analysis of variance showed that the major factor for splitting tensile strength was water the cement ratio. It is obviously seen from 3D surface view figures that splitting tensile strength decreased as water/cement ratio increased at all ages, exposed temperatures and replacement ratios as shown in Figures 5.31-5.38. Twenty eight days test results showed that at zero replacement ratio, when water/cement ratio decreased from 0.60 to 0.35 the increase in splitting tensile strength was 36 percent at 22 °C, 50 percent at 105 °C, 71 percent at 175 °C, and 37 percent at 250 °C; and the increase was 33 percent at 22 °C, 49 percent at 105 °C, 54 percent at 175 °C, and 38 percent at 250 °C when 90 days test results were compared. When the replacement ratio was 100 percent, 28 days test results showed that the splitting tensile strength increased 25 percent at 22 °C, 37 percent at 105 °C, 99 percent at 175 °C, and 46 percent at 250 °C as water/cement ratio decreased from 0.60 to 0.35; and the increase was 18 percent at 22 °C, 28 percent at 105 °C, 51 percent at 175 °C, and 39 percent at 250 °C when the testing age was 90 days.

In Equation 5.4 the coefficient of “B” (replacement ratio) was 0.42 which was the second biggest coefficient in the equation. This higher coefficient and the analysis of variance showed that the replacement ratio was also a significant factor for splitting tensile strength. As shown in Figures 5.39-5.40, the interaction graphs of replacement ratio at 28 and 90 days, at every water/cement ratio ranging from 0.35 to 0.60, and at all exposed temperatures showed also the considerable effect of the replacement ratio. The negative effect of replacement ratio factor on splitting tensile strength was generally observed. At w/c ratio of 0.35, when the replacement ratio increased from zero to 100 percent, the loss in splitting tensile strength at 22 °C was 30 and 13 percent at 28 and 90 days, respectively; 19 and 5 percent decrease at 28 and 90 days respectively was evaluated in splitting tensile strength after being exposed to 105 °C. The decrease in splitting tensile strength after being exposed to 175 °C was 60 and 30 percent at 28 and 90 days, respectively and 70 and 54 percent decrease at 28 and 90 days respectively was calculated in splitting tensile strength after being exposed to 250 °C. At w/c ratio of 0.60, when the replacement ratio increased from zero to 100 percent, the reduction in splitting tensile strength at 22 °C and 105 °C was 19 percent and 8 percent, respectively at 28 days; the drop in splitting tensile strength after exposure to 175 °C was 87 and 28 percent at 28 and 90 days, respectively; and 82 and 56

percent decrease at 28 and 90 days was respectively computed in splitting tensile strength after exposure to 250 °C.

The effect of temperature factor was less than water/cement and replacement ratio factors. After exposed to 105 °C the splitting tensile strength change with respect to that tested at room temperature was negligible at 28 and 90 days when the w/c and replacement ratios were 0.35 and zero percent, respectively. When the w/c and replacement ratios were 0.35 and 100 percent the strength gain was 10 and 5 percent at 28 and 90 days, respectively. Nine and 15 percent reduction was determined when the w/c and replacement ratios were 0.60 and zero percent, respectively; and there was almost no change in the splitting tensile strength when the w/c and replacement ratios were 0.60 and 100 percent. After exposure to 175 °C the splitting tensile strength increased 5 and 20 percent at 28 and 90 days respectively when the w/c and replacement ratios were 0.35 and zero percent. When the w/c and replacement ratios were 0.35 and 100 percent the decrease was 18 percent at 28 days and the increase was 4 percent at 90 days. Twenty percent decrease and 4 percent increase was calculated at 28 and 90 days, respectively, when the w/c and replacement ratios were 0.60 and zero percent, and a decrease was calculated in amount of 88 and 23 percent respectively when the w/c and replacement ratios were 0.60 and 100 percent. After exposure to 250 °C the gain in splitting tensile strength was 30 and 23 percent at 28 and 90 days respectively when the w/c and replacement ratios were 0.35 and zero percent, and when the w/c and replacement ratios were 0.35 and 100 percent the decrease was one and 11 percent at 28 and 90 days, respectively. Twenty nine and 18 percent increase was evaluated when the w/c and replacement ratios were 0.60 and zero percent, and a decrease was calculated in amount of 18 and 31 percent when the w/c and replacement ratios were 0.60 and 100 percent.

In Equation 5.4 the coefficient of “D” (age) was 0.098. This small coefficient affected the splitting tensile strength of recycled aggregate concrete a little. Test results and interaction figures of age showed that age has a positive effect generally on splitting tensile strength of recycled aggregate concretes. At 22 °C and 0.35 w/c ratio, the increase in splitting tensile strength from 28 to 90 days was 11 percent at 100 percent replacement ratio, and it was 18 percent at 100 percent replacement ratio and 0.60 w/c ratio. After exposure to 105 °C the increase in splitting tensile strength was 6 percent at 100 percent

replacement ratio and 0.35 w/c ratio, and it was 13 percent at 100 percent replacement ratio and 0.60 w/c ratio. After exposure to 175 °C the increase in splitting tensile strength was 11 and 36 percent at 0.35 w/c ratio and at zero and 100 percent replacement ratio respectively and it was 23 and 80 percent at zero and 100 percent replacement and at 0.60 w/c ratios. After exposure to 250 °C there was a little change in splitting tensile strength at 100 percent replacement ratio and at 0.35 w/c ratio, and the increase was 6 percent at 100 percent replacement ratio and 0.60 w/c ratio.

From the experimental test results and comparisons of the above paragraphs regarding the splitting tensile strength it could be concluded that:

1. Concretes produced with natural and recycled aggregates were much more affected by the w/c ratio than replacement ratio, temperature increases, and age factors. Increasing the w/c ratio from 0.35 to 0.60 caused a strength decrease of 33-71 percent in natural aggregate concretes, and 18-99 percent in recycled aggregate concretes.
2. Incorporation of recycled aggregates generally lowered the splitting tensile strength of concretes by 5-87 percent. Splitting tensile strength of natural aggregate concretes varied in the range of 2.39 to 5.06 MPa, whereas the concretes produced with fully recycled aggregates had a split tensile strength of 1.28-3.53 MPa. In literature it is also reported that there was a reduction in splitting tensile strength for 100 percent replacement ratio compared to the natural aggregate concrete. This result showed that bond between natural aggregate and new mortar matrix was better than the bond between recycled aggregate and new matrix and original virgin aggregate and adhered mortar.
3. Splitting tensile strength of natural and recycled aggregate concretes exposed to elevated temperatures showed that the strength was sensitive to temperature changes. Most significant reduction, 88 percent, was observed in recycled aggregate concretes produced with w/c ratio of 0.60 and tested after exposure to 175 °C. The loss in splitting tensile strength is considerably sharp. This is because the tensile strength is more sensitive to cracks either on macro or micro scales which are caused by thermal incompatibility [92, 100].

4. It is clear from the results that the effect of replacement ratio factor on splitting tensile strength is higher. If the splitting tensile strength is in concern, concretes that will be subjected to elevated temperatures could be produced with recycled aggregates by lowering the w/c and replacement ratio. Lowering the water/cement and replacement ratio would compensate the negative effect of recycled aggregates.

Table 5.18. Results of ANOVA for splitting tensile strength response surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	5.1134	1	5.1134	69.8068	<0.0001
B	2.7465	1	2.7465	37.4940	<0.0001
C	0.5104	3	0.1701	2.3227	0.1268
D	0.2500	1	0.2500	3.4130	0.0895
AB	0.1233	1	0.1233	1.6831	0.2189
AC	0.1960	3	0.0653	0.8919	0.4733
AD	0.0095	1	0.0095	0.1303	0.7244
BC	1.4833	3	0.4944	6.7501	0.0064
BD	0.2207	1	0.2207	3.0128	0.1082
CD	0.7568	3	0.2523	3.4437	0.0518

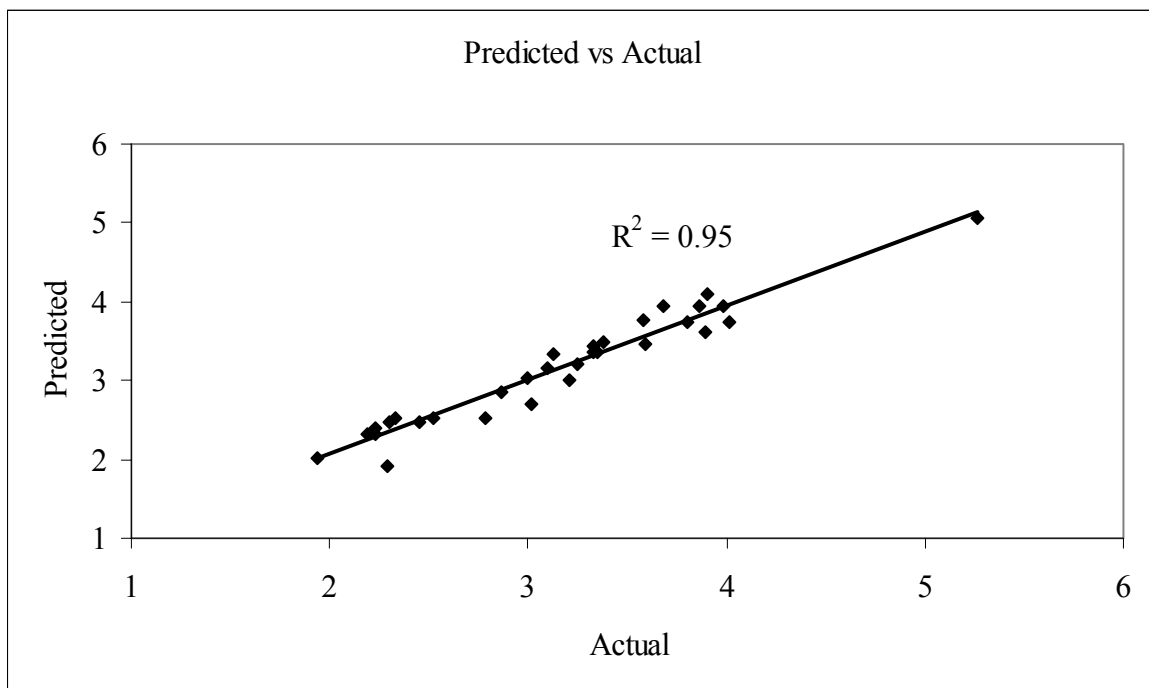


Figure 5.30. Predicted versus actual values of splitting tensile strength (MPa)

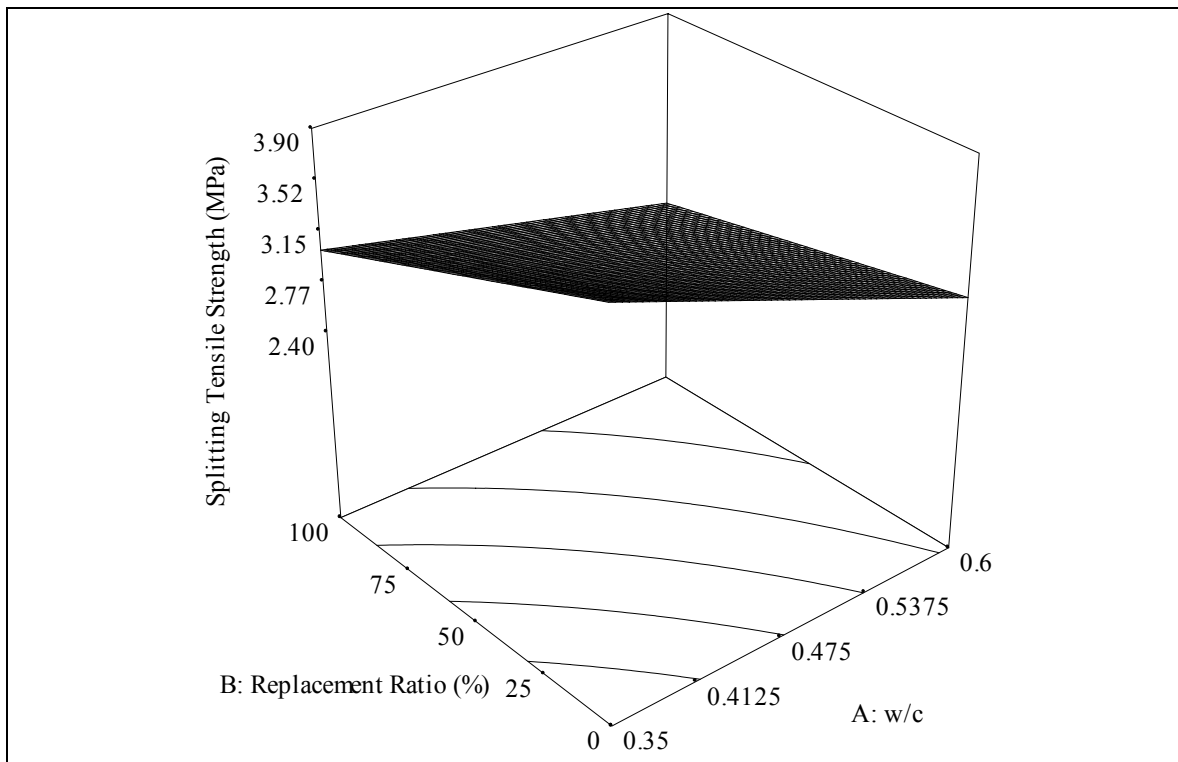


Figure 5.31. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)

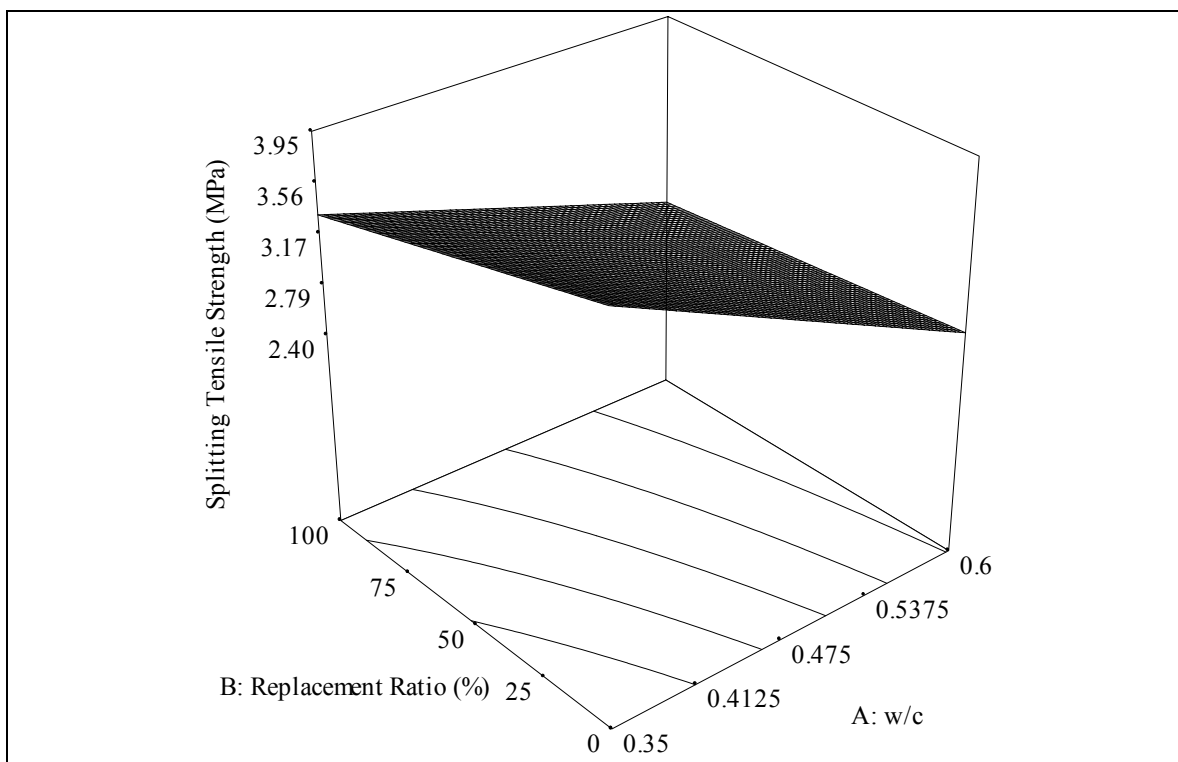


Figure 5.32. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)

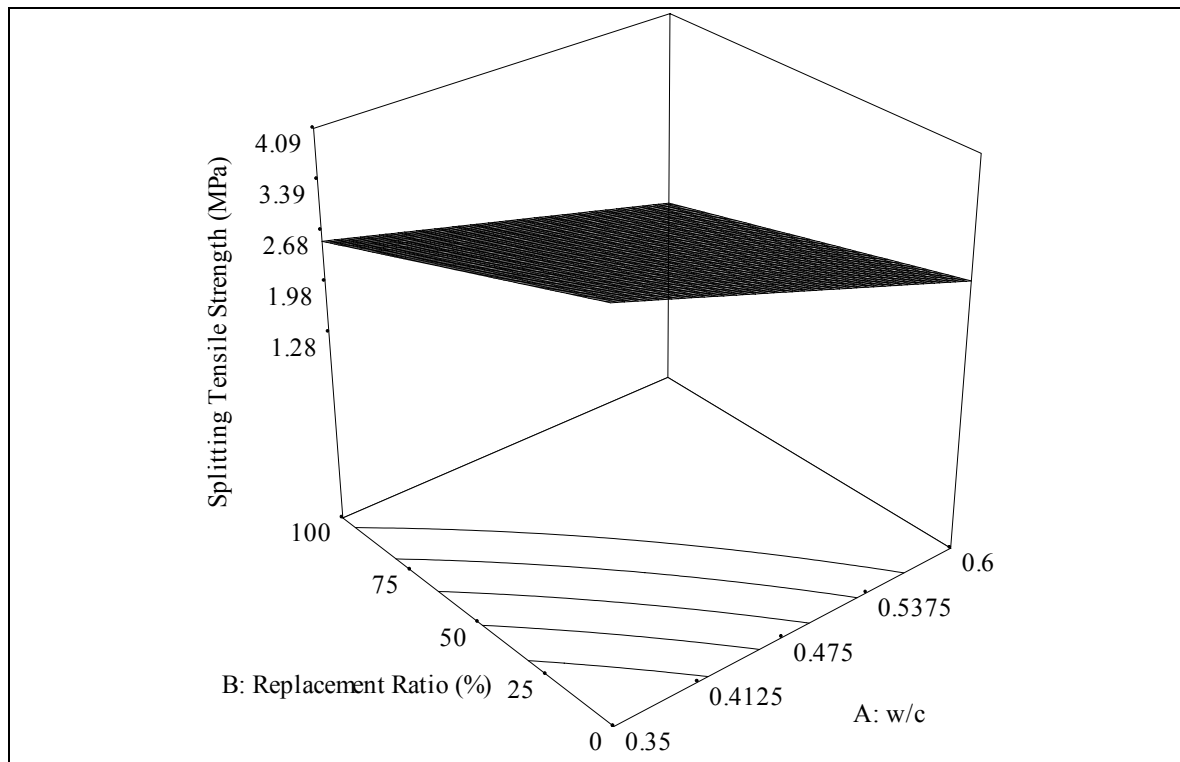


Figure 5.33. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)

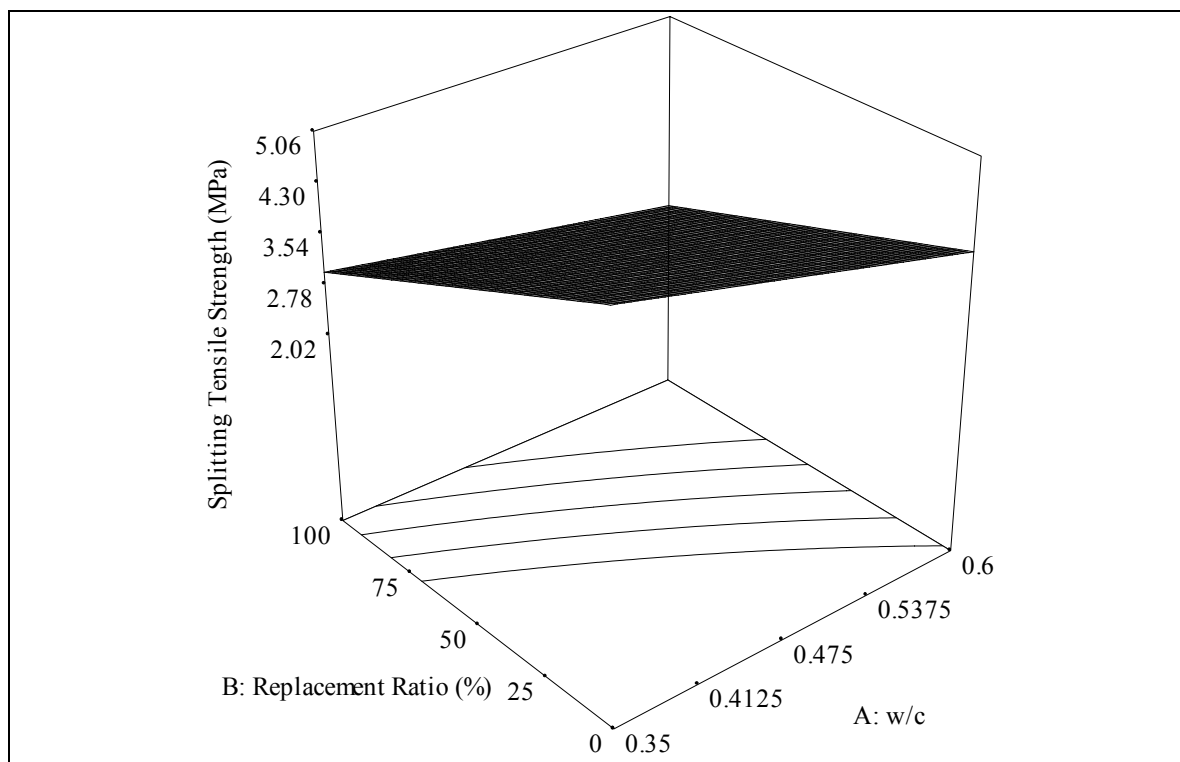


Figure 5.34. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)

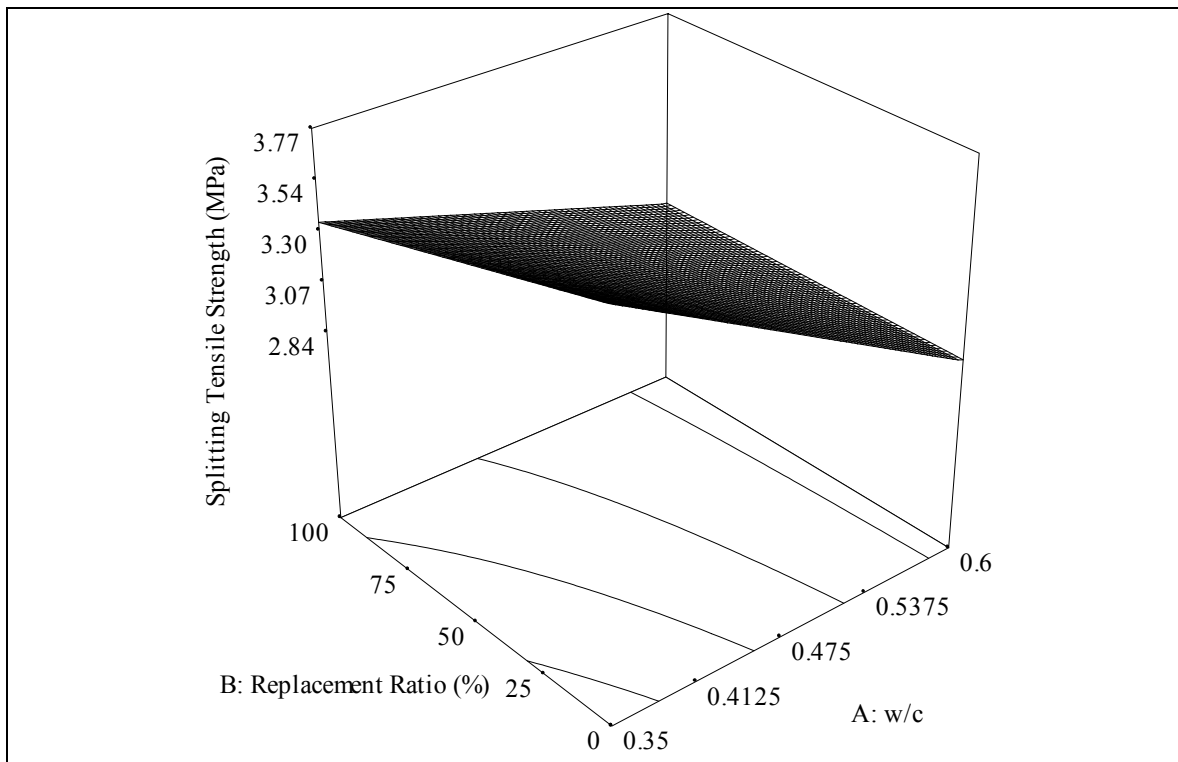


Figure 5.35. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)

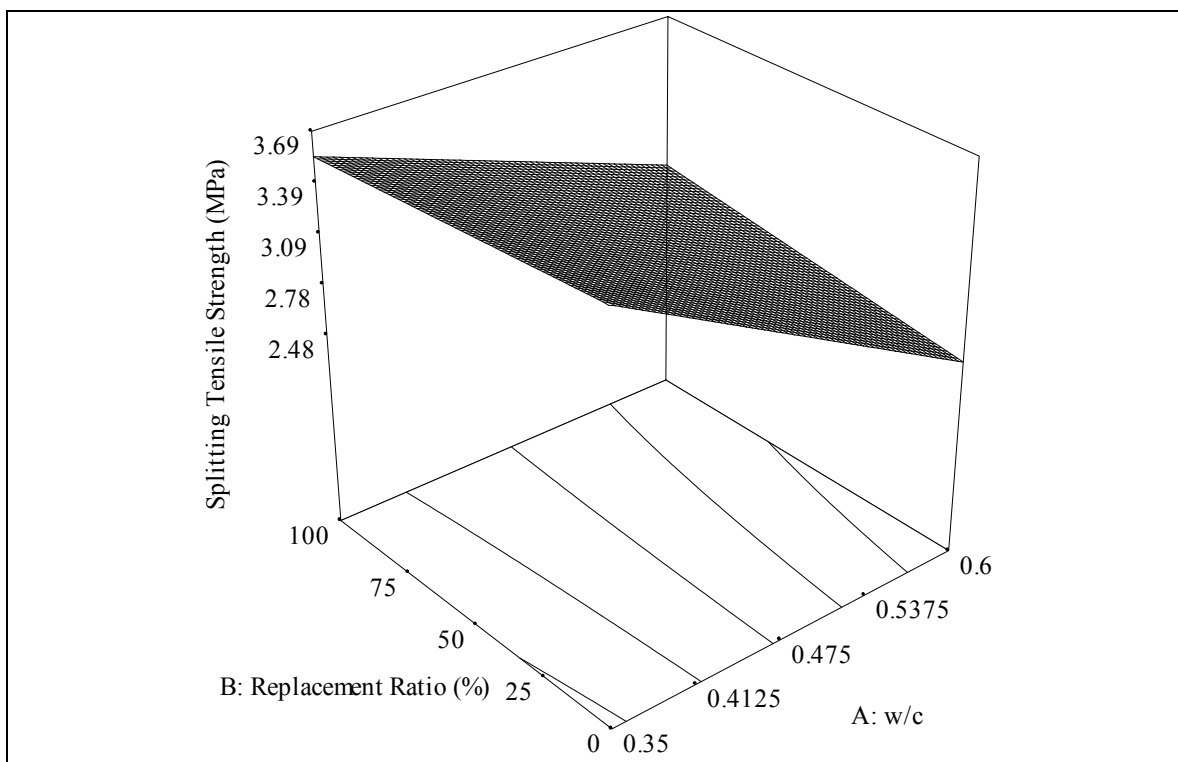


Figure 5.36. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)

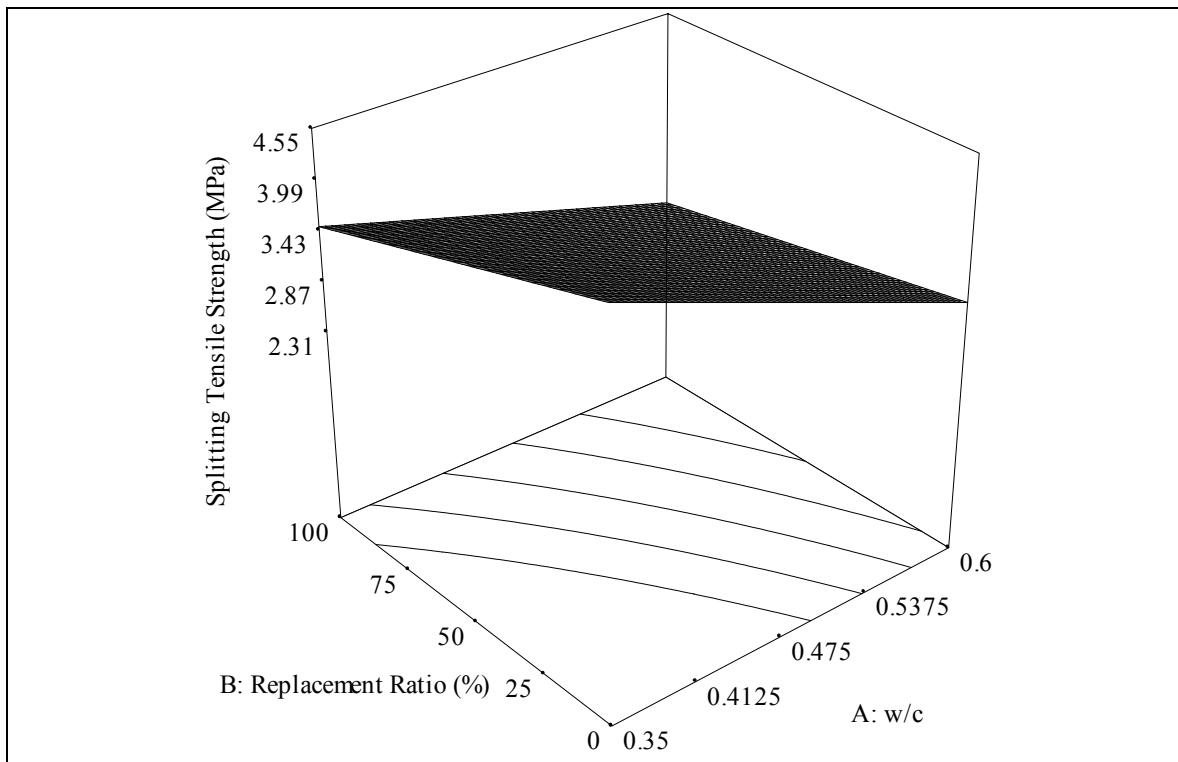


Figure 5.37. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)

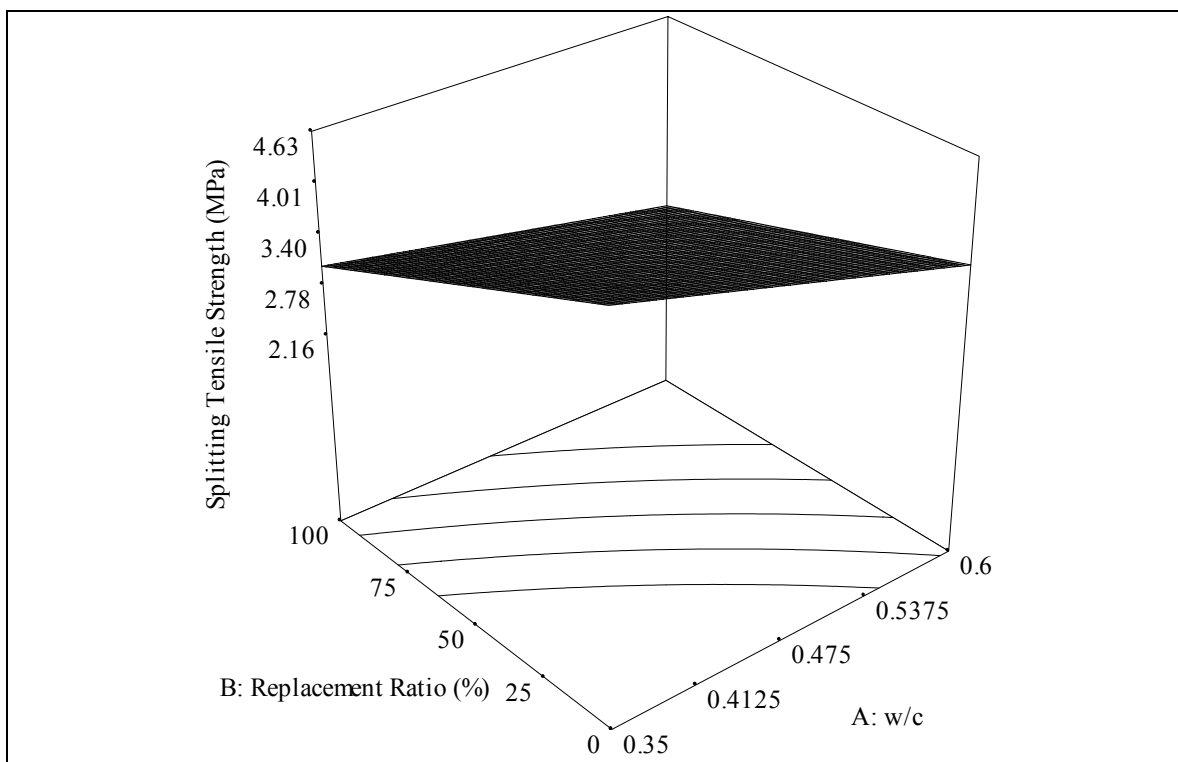


Figure 5.38. 3D surface view of splitting tensile strength of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)

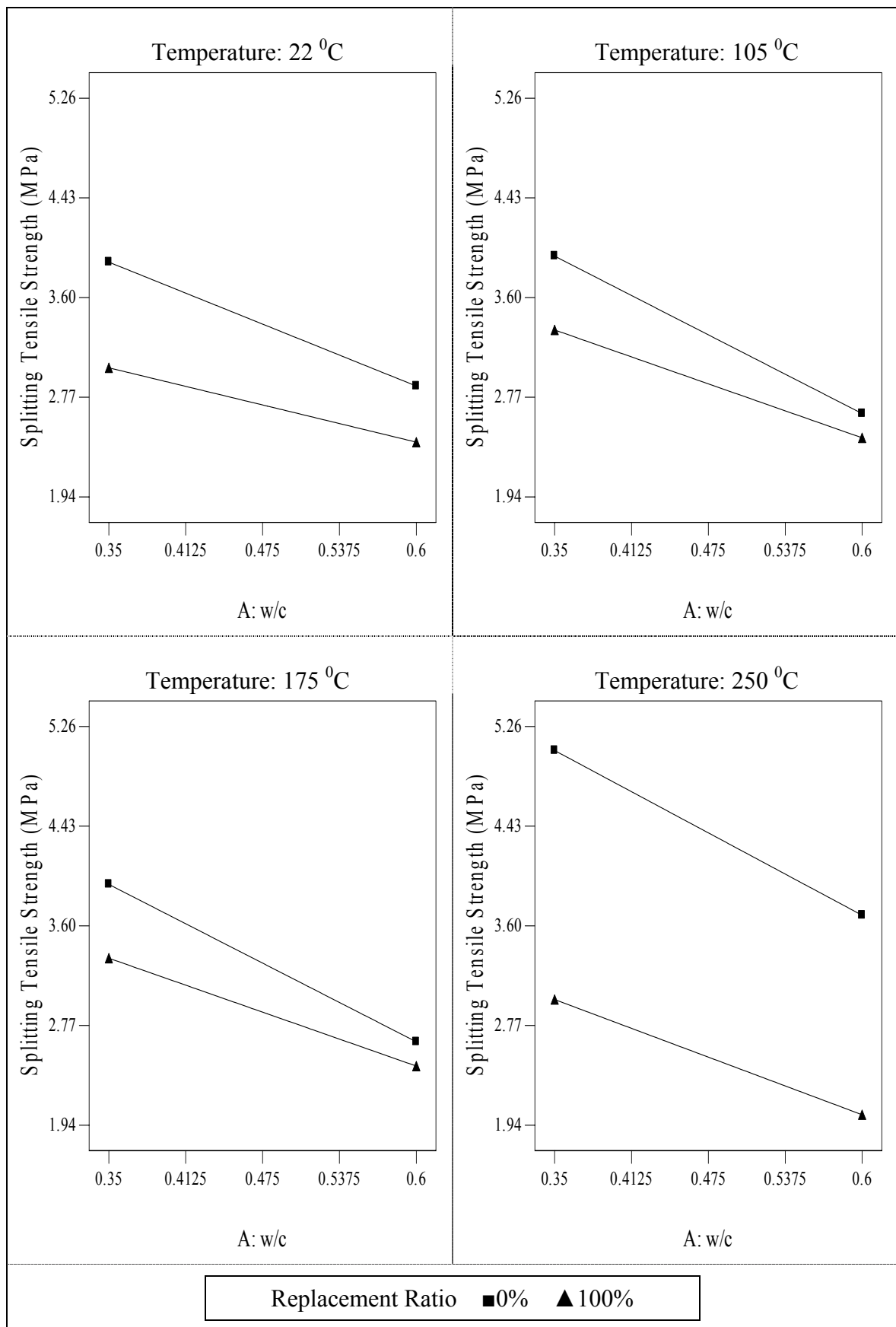


Figure 5.39. Interaction graph of w/c and replacement ratio at 28 days

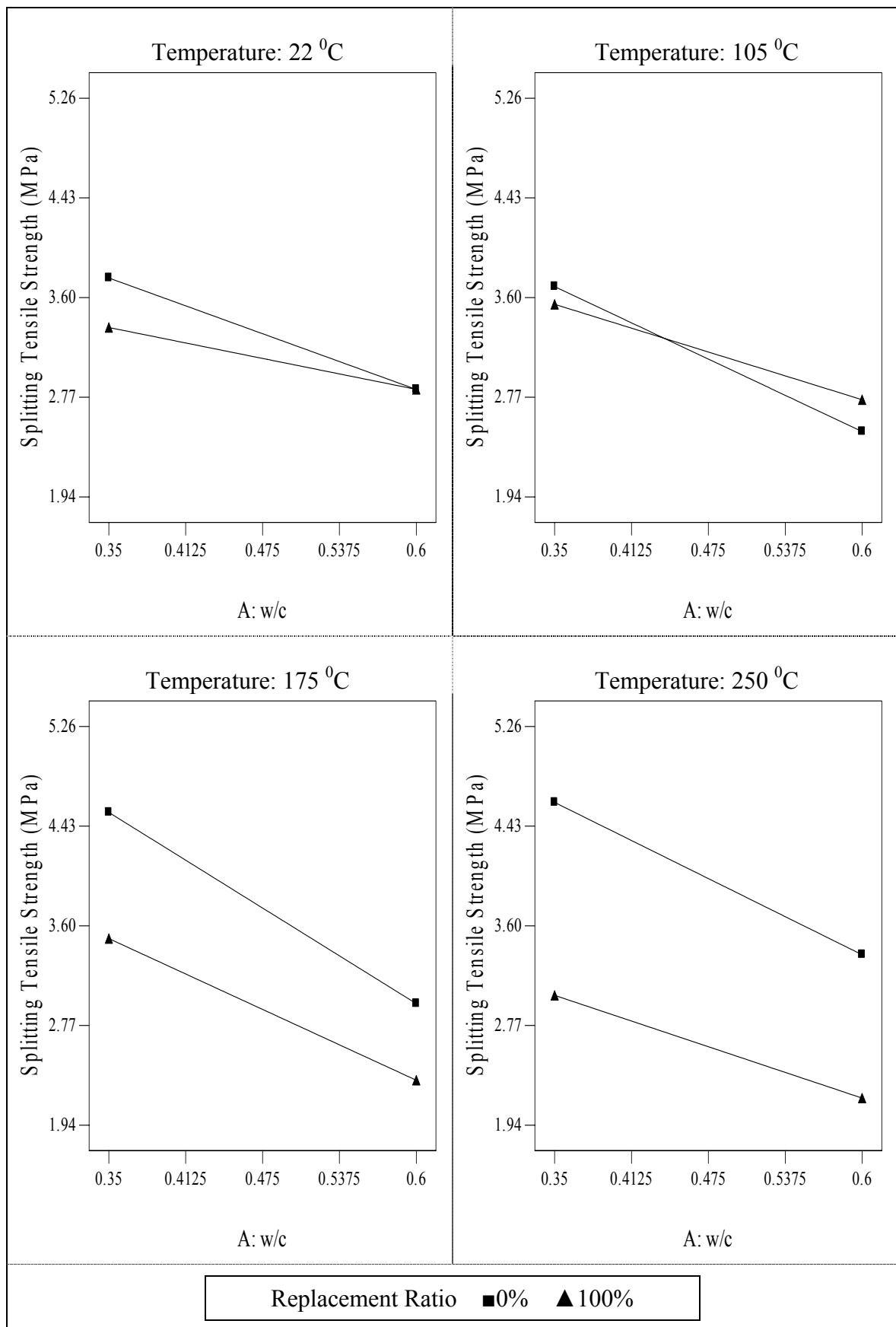


Figure 5.40. Interaction graph of w/c and replacement ratio at 90 days

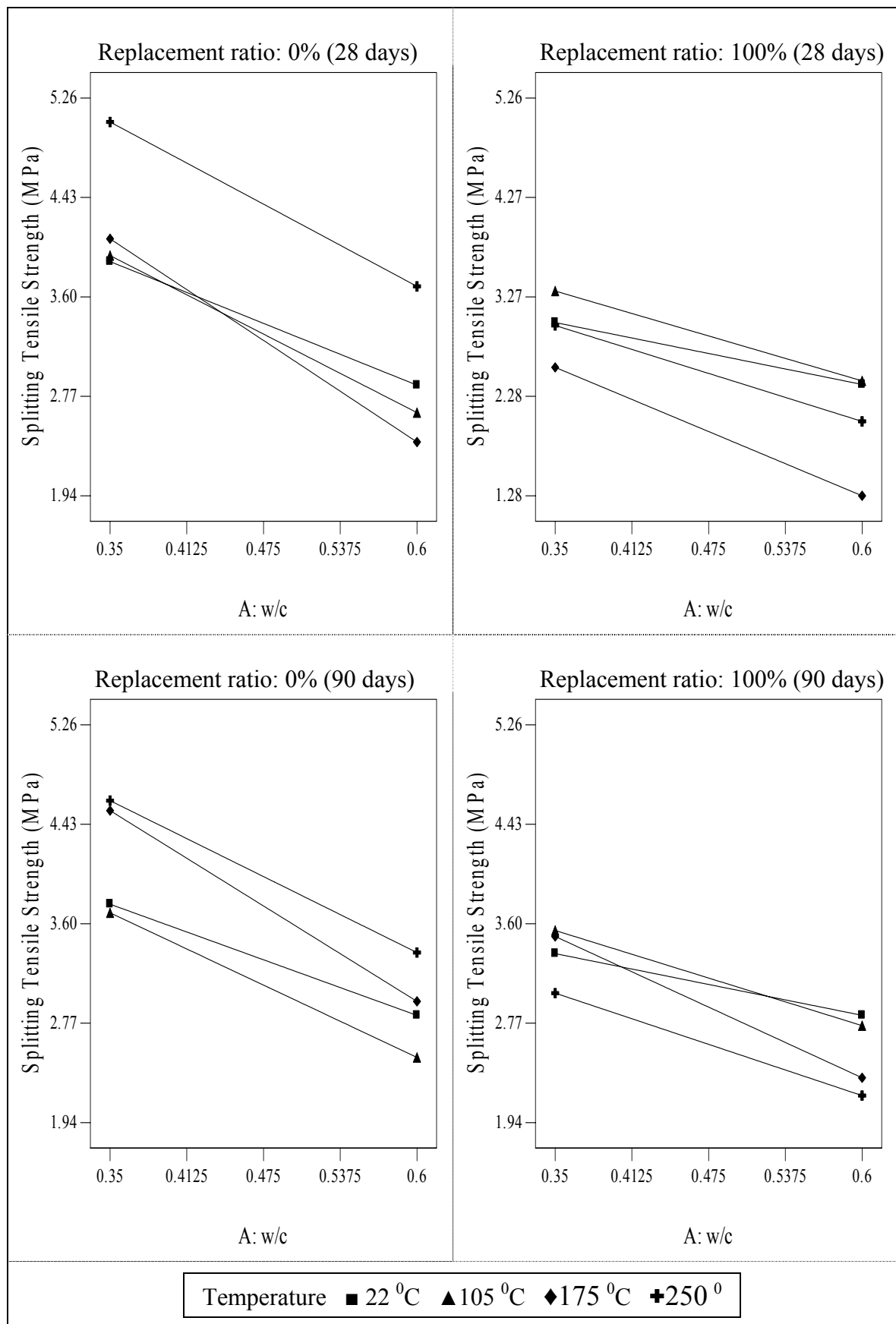


Figure 5.41. Interaction graph of w/c and temperature

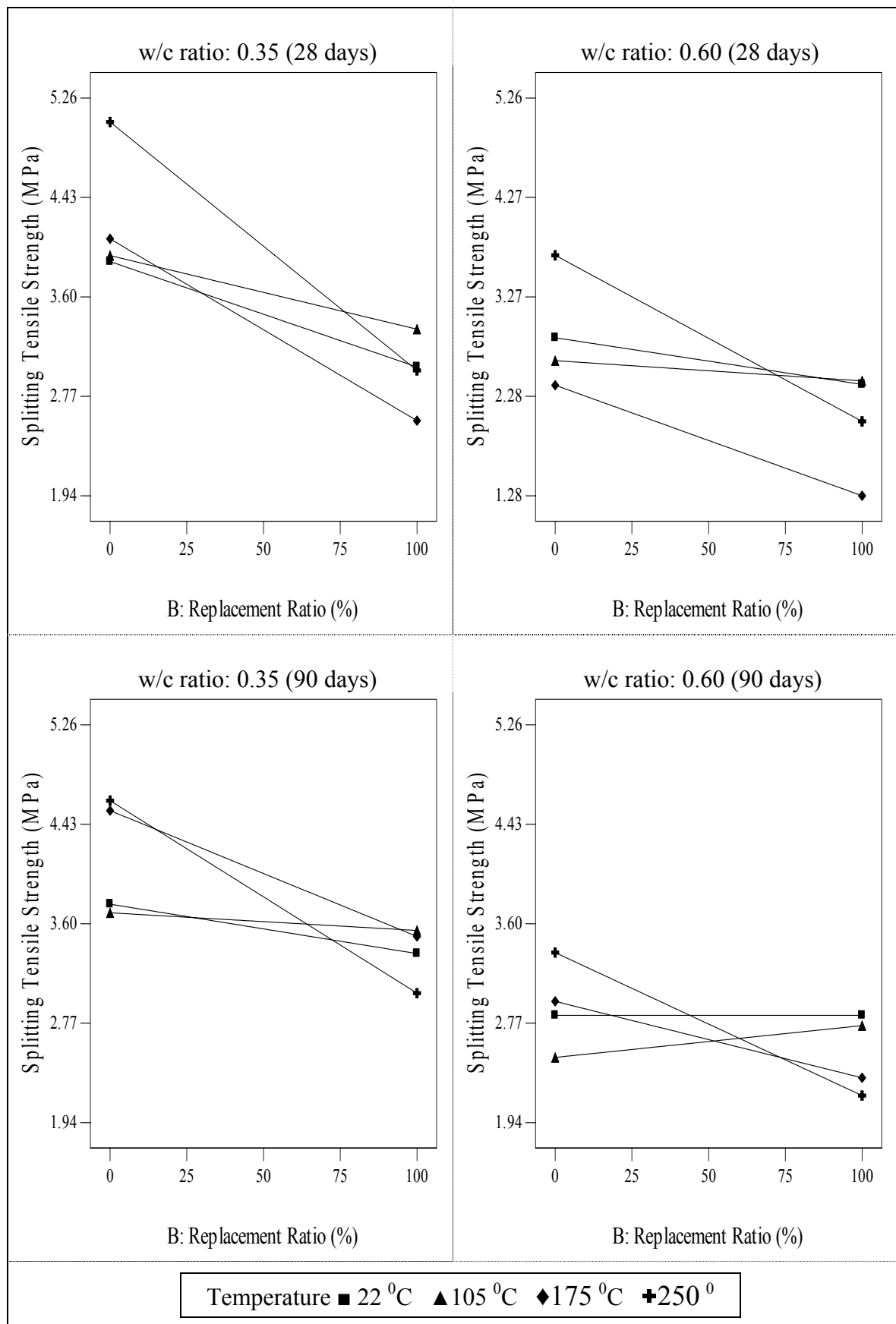


Figure 5.42. Interaction graph of replacement ratio and temperature

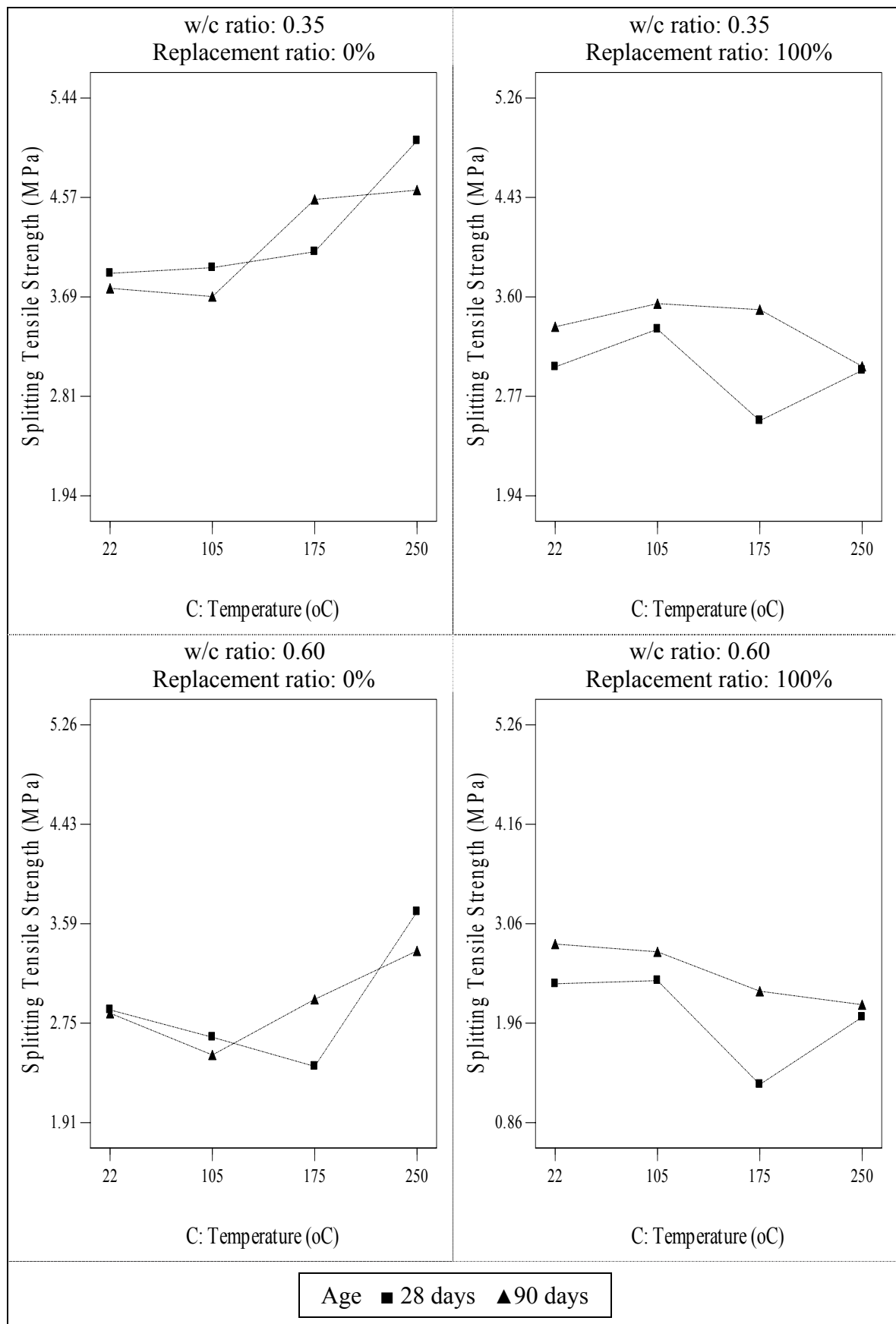


Figure 5.43. Interaction graph of temperature and age

5.4. Durability Properties of Natural and Recycled Aggregate Concretes

5.4.1. Water Absorption

Water absorption test results at 28 days of the first stage concrete specimens made with natural aggregates (mixture I) and recycled aggregates (mixture II-IV) are given in Table 5.19. As seen from that table recycled aggregate concretes absorbed more water than natural aggregate concrete, which was due to high absorption capacity of the adhered mortar in recycled aggregate concrete. On the other hand, there was not any direct relation observed between the source concrete quality and water absorption. Other than source concrete quality, adhered mortar content had an effective role in water absorption.

Table 5.19. Water absorption of first stage concretes at 28 days

Mixture No	Water absorption 1 st sample	Water absorption 2 nd sample	Water absorption 3 rd sample	Water absorption average
I	4.26%	4.13%	4.58%	4.32%
II	5.36%	5.31%	5.35%	5.34%
III	5.64%	6.10%	5.56%	5.77%
IV	4.86%	4.76%	5.60%	5.08%

The water absorption (absorption after immersion), dry bulk density, saturated bulk density, apparent density, and volume of permeable pore space (voids) test results of second stage concretes, mixes 1-31, produced with various water/cement and replacement ratios and tested at 28 and 90 days at room temperature and after exposure to elevated temperatures are given in Table 5.20. 2FI model was used for analysis of water absorption results. The standard deviation and the R-squared values of the model were 0.43 and 0.99. The equation of 2FI model for the water absorption in terms of coded factors is shown in Equation 5.5.

Table 5.20. Voids in hardened concrete, water absorption, and density values of second stage natural & recycled aggregate concretes

Run	w/c	RR (%)	T (°C)	Age (days)	WA ¹ (%)	DBD ² (g/cm ³)	SBD ³ (g/cm ³)	AD ⁴ (g/cm ³)	Voids ⁵ (%)
1	0.35	0	22	90	2.94	2.38	2.45	2.56	6.99
2	0.35	0	105	28	3.69	2.36	2.44	2.58	8.7
3	0.35	0	105	28	3.51	2.35	2.44	2.57	8.26
4	0.35	0	175	28	4.31	2.36	2.46	2.62	10.15
5	0.35	0	250	28	5.36	2.32	2.45	2.65	12.43
6	0.35	50	22	28	4.56	2.29	2.4	2.56	10.45
7	0.35	50	105	90	4.3	2.3	2.4	2.55	9.9
8	0.35	100	22	90	5.49	2.22	2.34	2.53	12.2
9	0.35	100	105	28	7.22	2.16	2.32	2.56	15.6
10	0.35	100	175	90	7.23	2.18	2.33	2.58	15.73
11	0.35	100	250	90	9.02	2.13	2.32	2.63	19.17
12	0.4125	25	250	90	6.75	2.25	2.4	2.65	15.17
13	0.475	0	175	90	6.38	2.25	2.4	2.63	14.37
14	0.475	0	175	90	6.32	2.26	2.4	2.64	14.27
15	0.475	25	22	28	6.19	2.22	2.35	2.57	13.72
16	0.475	50	250	28	9.64	2.12	2.32	2.66	20.44
17	0.475	100	105	90	7.78	2.14	2.3	2.56	16.63
18	0.475	100	175	28	9.5	2.1	2.29	2.62	19.91
19	0.5375	25	175	28	8.61	2.15	2.33	2.63	18.48
20	0.5375	50	22	90	7.08	2.18	2.34	2.58	15.44
21	0.5375	75	250	90	10.1	2.07	2.27	2.61	20.87
22	0.6	0	22	28	6.84	2.21	2.36	2.61	15.13
23	0.6	0	105	90	8.86	2.13	2.32	2.63	18.88
24	0.6	0	105	90	8.82	2.12	2.31	2.61	18.69
25	0.6	0	250	90	8.91	2.16	2.35	2.67	19.22
26	0.6	50	105	28	9.02	2.11	2.3	2.61	19.07
27	0.6	50	105	28	9	2.1	2.29	2.59	18.88
28	0.6	100	22	28	10.49	2.03	2.24	2.58	21.29
29	0.6	100	175	90	10.36	2.03	2.24	2.57	21.04
30	0.6	100	175	90	10.52	2.03	2.24	2.58	21.32
31	0.6	100	250	28	11.22	2.03	2.26	2.63	22.78

¹WA: water absorption (absorption after immersion)

²DBD: dry bulk density

³SBD: saturated bulk density

⁴AD: apparent density

⁵Voids: volume of permeable pore space

$$\begin{aligned}
& 7.55 + 1.81 * A + 1.44 * B - 1.02 * C[1] - 0.50 * C[2] \\
& + 0.31 * C[3] - 0.23 * D - 0.37 * AB + 0.22 * AC[1] \\
\text{Water} & \\
\text{Absorption (\%)} = & + 0.29 * AC[2] + 0.088 * AC[3] + 0.19 * AD + 0.071 * BC[1] \quad (5.5) \\
& - 0.35 * BC[2] - 0.028 * BC[3] - 0.29 * BD \\
& - 0.025 * C[1]D + 0.057 * C[2]D - 0.044 * C[3]D
\end{aligned}$$

The coded factors were used in the above equation. The values for coded factors were listed in Tables 5.7-5.10. The water absorption values could be calculated with the above equation by using the coded values for each factor. The predicted and actual values of the water absorption are seen in Table 5.21, and graphed in Figure 5.44; and predicted water absorption values are shown in Figures 5.45-5.52. The R-squared value of 0.99 shows that 2FI model is quite good for the water absorption prediction. The results of the ANOVA for response surface 2FI model are presented in Table 5.22. As seen from ANOVA table the model terms A, B, C, D, AB, AC, and BD are significant, because the Prob>F values are less than 0.05. This means that the water/cement ratio (A), the replacement ratio (B), temperature (C), and age (D) have important effects on water absorption. For the water absorption response, the relations between water/cement and replacement ratios are given in Figures 5.53-5.54; the interaction graphs of water/cement ratio and temperature are seen in Figure 5.55; the interaction graphs of replacement ratio and temperature are illustrated in Figure 5.56; and the interaction graphs of temperature and age are presented in Figure 5.57.

As seen from Equation 5.5 the biggest coefficient was 1.81 and associated with “A” (water/cement ratio). This highest coefficient and analysis of variance showed that the major factor for water absorption was the water/cement ratio. In Equation 5.5 the coefficient of “B” (replacement ratio) was 1.44 which was the second biggest coefficient in the equation. This higher coefficient and analysis of variance showed that the replacement ratio was also a significant factor for water absorption. As reported in the analysis of variance the temperature and age factors were significant, too. It is seen from 3D surface figures and from interaction figures that water absorption increased when water/cement ratio increased, the increment was also observed with the increase in recycled aggregate replacement and exposed temperatures. According to the test results water absorption was higher at early ages. The actual water absorption values ranged from 2.94 to 11.22 percent.

Table 5.21. Actual and predicted values of water absorption

Run	w/c	RR (%)	T (°C)	Age (days)	Actual value (%)	Predicted value (%)	Residual (%)	Error
1	0.35	0	22	90	2.94	2.47	0.47	16.0%
2	0.35	0	105	28	3.69	3.56	0.13	3.5%
3	0.35	0	105	28	3.51	3.56	-0.05	1.4%
4	0.35	0	175	28	4.31	4.36	-0.05	1.2%
5	0.35	0	250	28	5.36	5.57	-0.21	3.9%
6	0.35	50	22	28	4.56	4.95	-0.39	8.6%
7	0.35	50	105	90	4.30	4.58	-0.28	6.5%
8	0.35	100	22	90	5.49	5.65	-0.16	2.9%
9	0.35	100	105	28	7.22	7.06	0.16	2.2%
10	0.35	100	175	90	7.23	6.99	0.24	3.3%
11	0.35	100	250	90	9.02	8.98	0.04	0.4%
12	0.4125	25	250	90	6.75	7.03	-0.28	4.1%
13	0.475	0	175	90	6.38	6.47	-0.09	1.4%
14	0.475	0	175	90	6.32	6.47	-0.15	2.4%
15	0.475	25	22	28	6.19	5.89	0.30	4.8%
16	0.475	50	250	28	9.64	8.99	0.65	6.7%
17	0.475	100	105	90	7.78	7.68	0.10	1.3%
18	0.475	100	175	28	9.50	9.83	-0.33	3.5%
19	0.5375	25	175	28	8.61	8.23	0.38	4.4%
20	0.5375	50	22	90	7.08	7.40	-0.32	4.5%
21	0.5375	75	250	90	10.10	9.90	0.20	2.0%
22	0.60	0	22	28	6.84	7.19	-0.35	5.1%
23	0.60	0	105	90	8.86	8.75	0.11	1.2%
24	0.60	0	105	90	8.82	8.75	0.07	0.8%
25	0.60	0	250	90	8.91	8.87	0.04	0.4%
26	0.60	50	105	28	9.02	9.13	-0.11	1.2%
27	0.60	50	105	28	9.00	9.13	-0.13	1.4%
28	0.60	100	22	28	10.49	10.06	0.43	4.1%
29	0.60	100	175	90	10.36	10.44	-0.08	0.8%
30	0.60	100	175	90	10.52	10.44	0.08	0.8%
31	0.60	100	250	28	11.22	11.67	-0.45	4.0%

When the water/cement ratio of natural aggregate concretes increased from 0.35 to 0.60, 28 days test results showed that the predicted water absorption values increased from 2.78 to 7.19 percent at 22 °C, from 3.56 to 8.11 percent at 105 °C, from 4.36 to 8.50 percent at 175 °C, and from 5.57 to 8.33 percent at 250 °C. When the water/cement ratio of natural aggregate concretes increased from 0.35 to 0.60, 90 days test results showed that the water absorption values increased from 2.47 to 7.65 percent at 22 °C, from 3.41 to 8.75 percent at 105 °C, from 4.01 to 8.93 percent at 175 °C, and from 5.33 to 8.87 percent at 250 °C. When the water/cement ratio of concretes, which were produced with fully coarse recycled aggregates, increased from 0.35 to 0.60, 28 days test results showed that the water absorption values increased from 7.12 to 10.06 percent at 22 °C, 7.06 to 10.14 percent at 105 °C, from 8.50 to 11.17 percent at 175 °C, and from 10.38 to 11.67 percent at 250 °C. When the water/cement ratio of concretes, which were produced with fully coarse recycled aggregates, increased from 0.35 to 0.60, 90 days test results showed that the water absorption values increased from 5.65 to 9.36 percent at 22 °C, from 5.75 to 9.61 percent at 105 °C, from 6.99 to 10.44 percent at 175 °C, and from 8.98 to 11.05 percent at 250 °C.

Based on the experimental test results and analyses and comparisons of the above paragraphs, the following conclusions for water absorption could be drawn:

1. Higher water absorption values were recorded at early ages, elevated temperatures, and bigger replacement ratios.
2. The measured water absorption of 28 days test results was higher than that of 90 days test results, because the hydration of concrete by time resulted in denser cement matrix which decreased the pore size.
3. The presence of adhered mortar on recycled aggregates was the main reason for recycled aggregate concretes having higher water absorption values than natural aggregate concretes. Besides, incorporation of recycled aggregate into concrete may have caused the formation of relatively poor interfacial zone, especially when higher replacement ratios were employed.
4. Elevated temperatures led to an increase in water absorption. This increase could be explained by changing of the concrete microstructure most probably because thermal mismatch allowed crack formation due to elevated temperatures. Concretes produced

with lower w/c ratios suffered more from higher absorbed water with elevated temperatures much more than those produced with higher w/c ratios.

5. Water absorption values increased in the range of 12-210 percent when w/c ratio increased from 0.35 to 0.60. Elevating the temperature from room temperature to 250 °C caused also an increase in water absorption values in an amount of 16-116 percent.

Table 5.22. Results of ANOVA for water absorption response surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	61.50	1	61.50	328.65	< 0.0001
B	34.87	1	34.87	186.36	< 0.0001
C	19.79	3	6.60	35.25	< 0.0001
D	1.14	1	1.14	6.10	0.0295
AB	1.56	1	1.56	8.36	0.0135
AC	2.16	3	0.72	3.85	0.0385
AD	0.53	1	0.53	2.85	0.1171
BC	1.07	3	0.36	1.91	0.1825
BD	1.35	1	1.35	7.20	0.0199
CD	0.04	3	0.01	0.08	0.9711

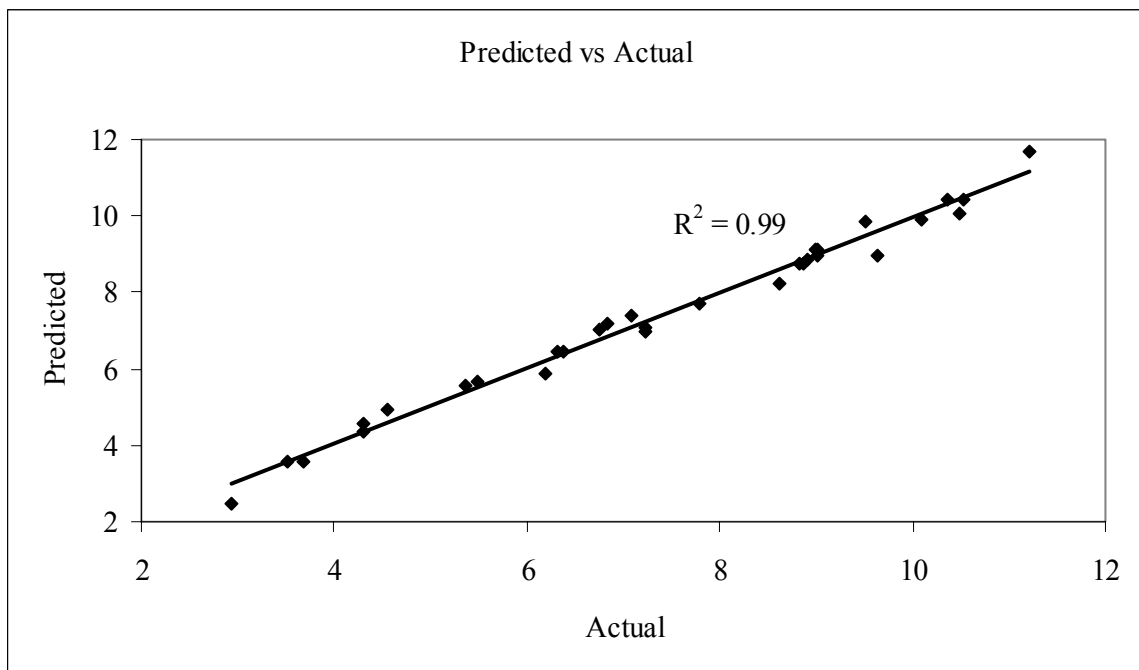


Figure 5.44. Predicted versus actual values of water absorption (%)

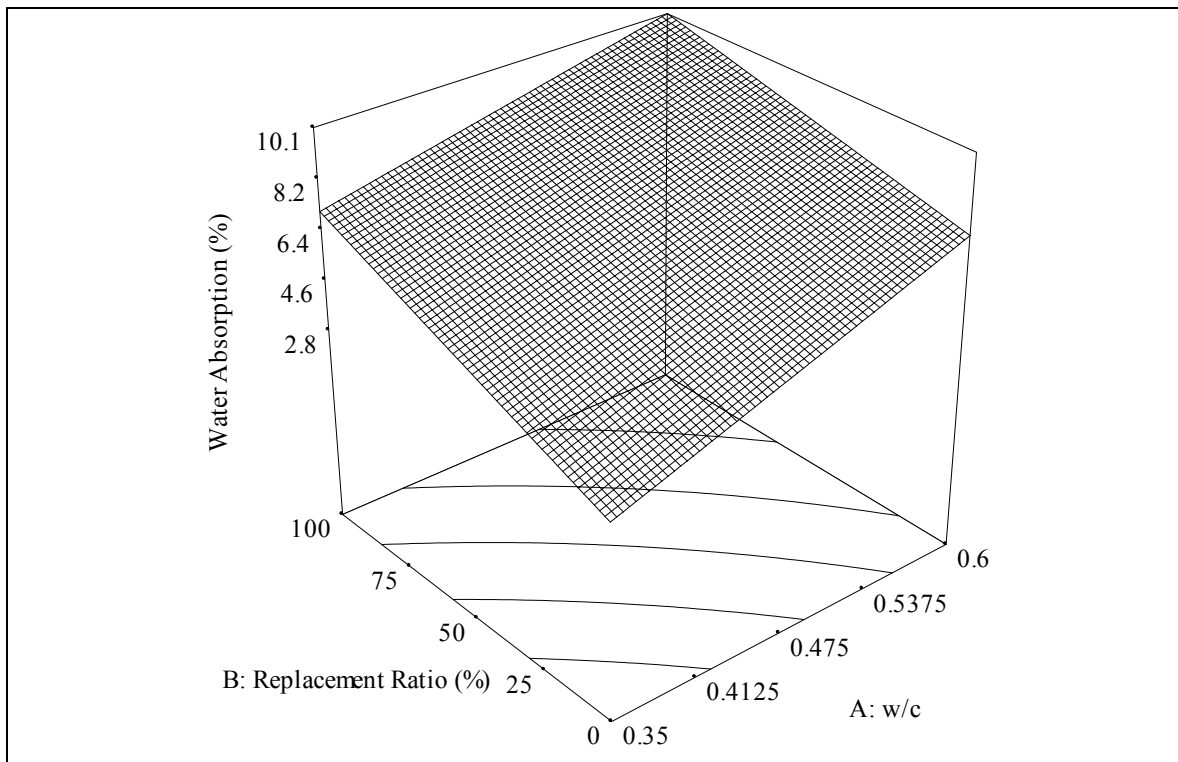


Figure 5.45. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)

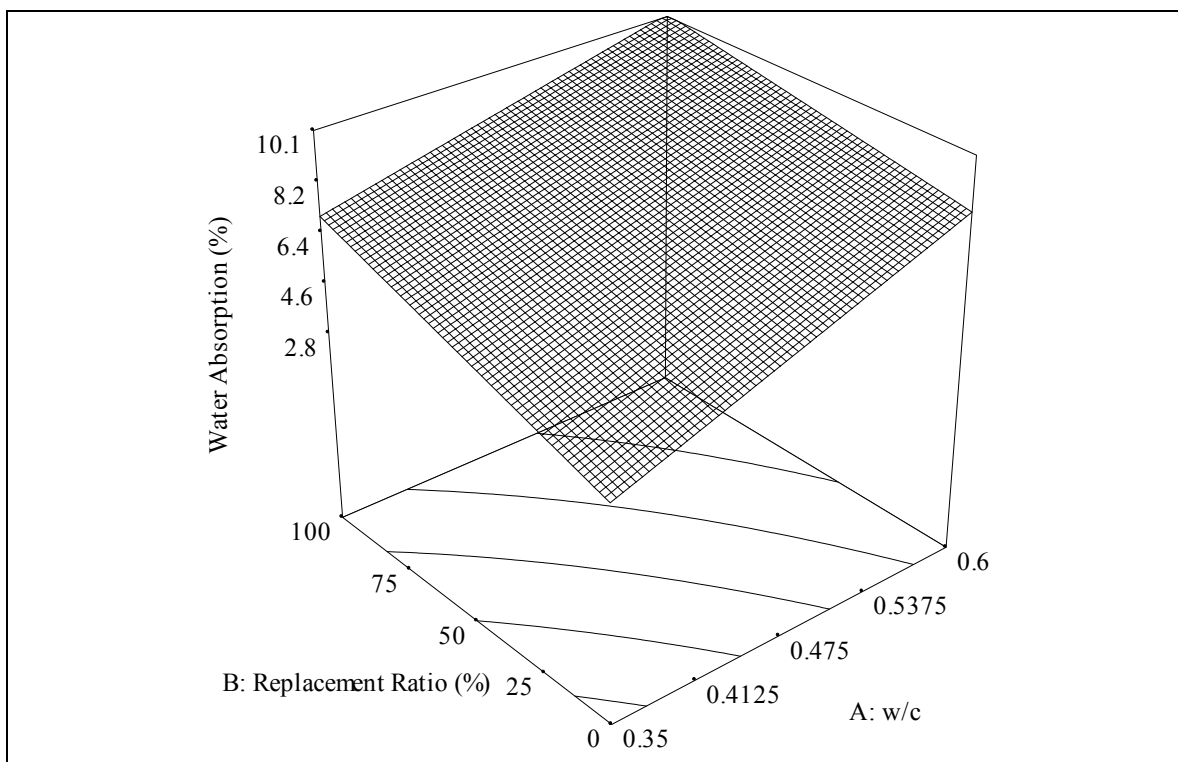


Figure 5.46. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)

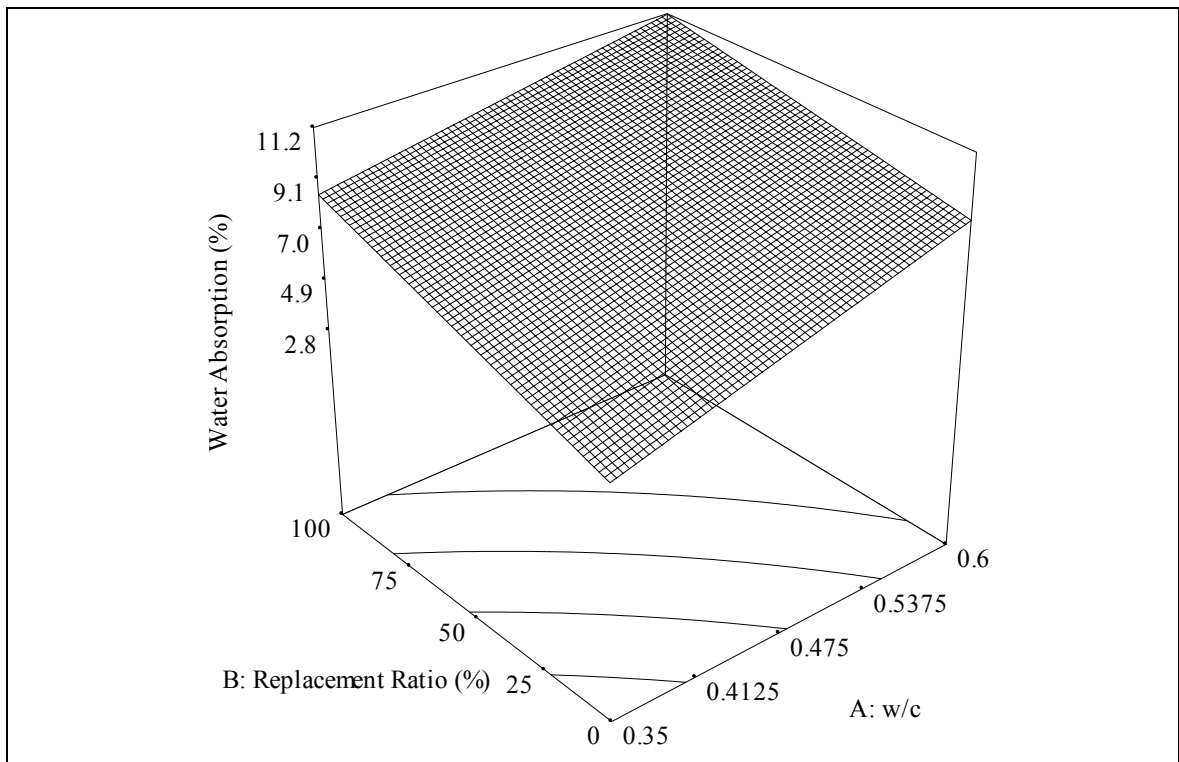


Figure 5.47. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)

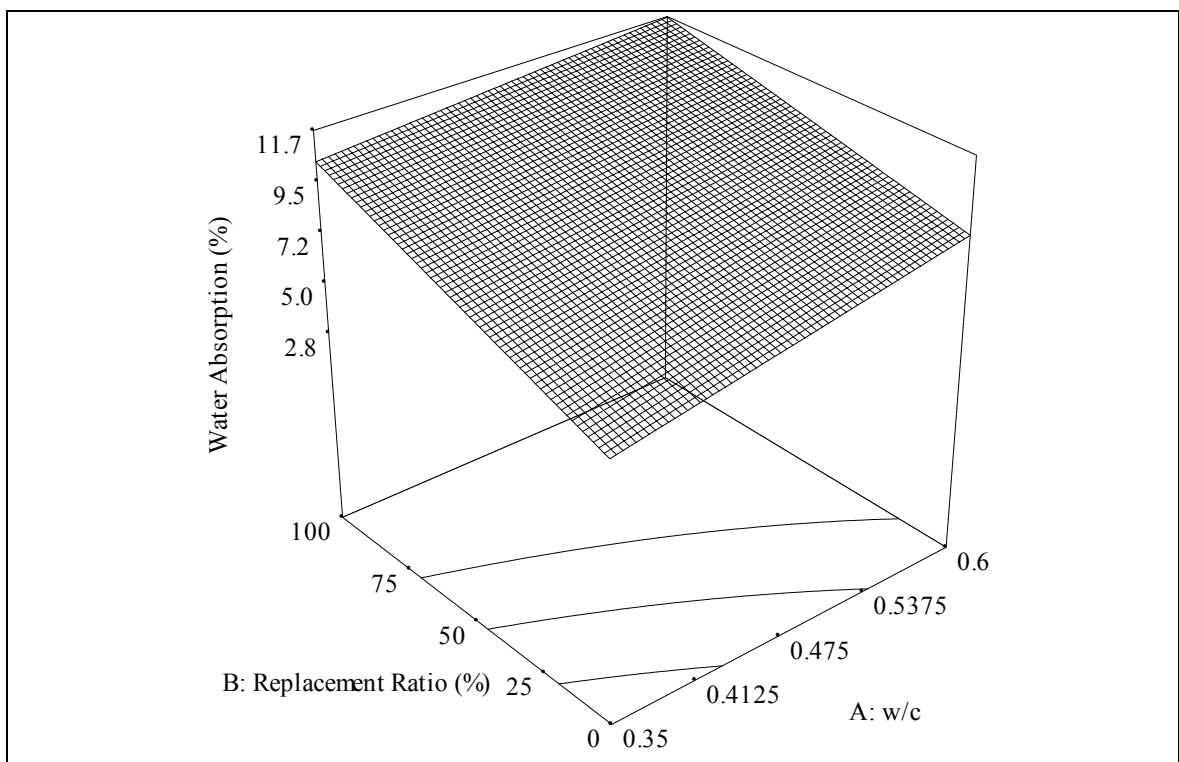


Figure 5.48. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)

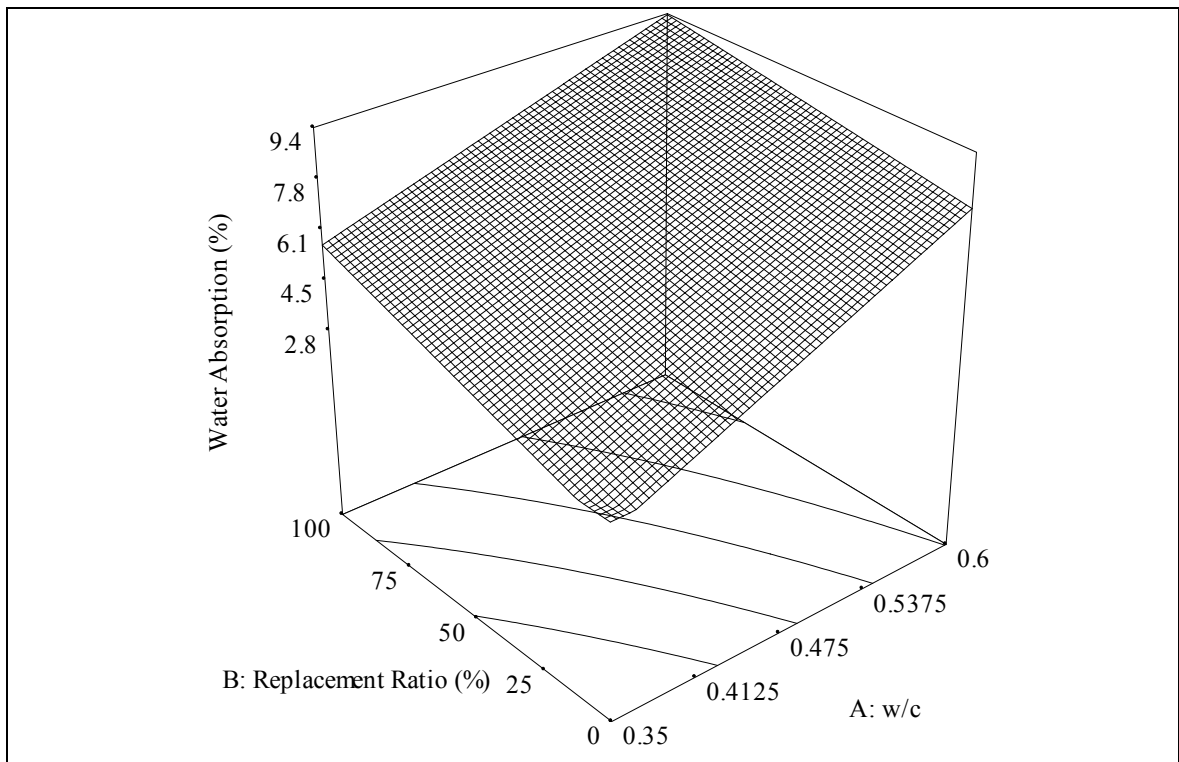


Figure 5.49. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)

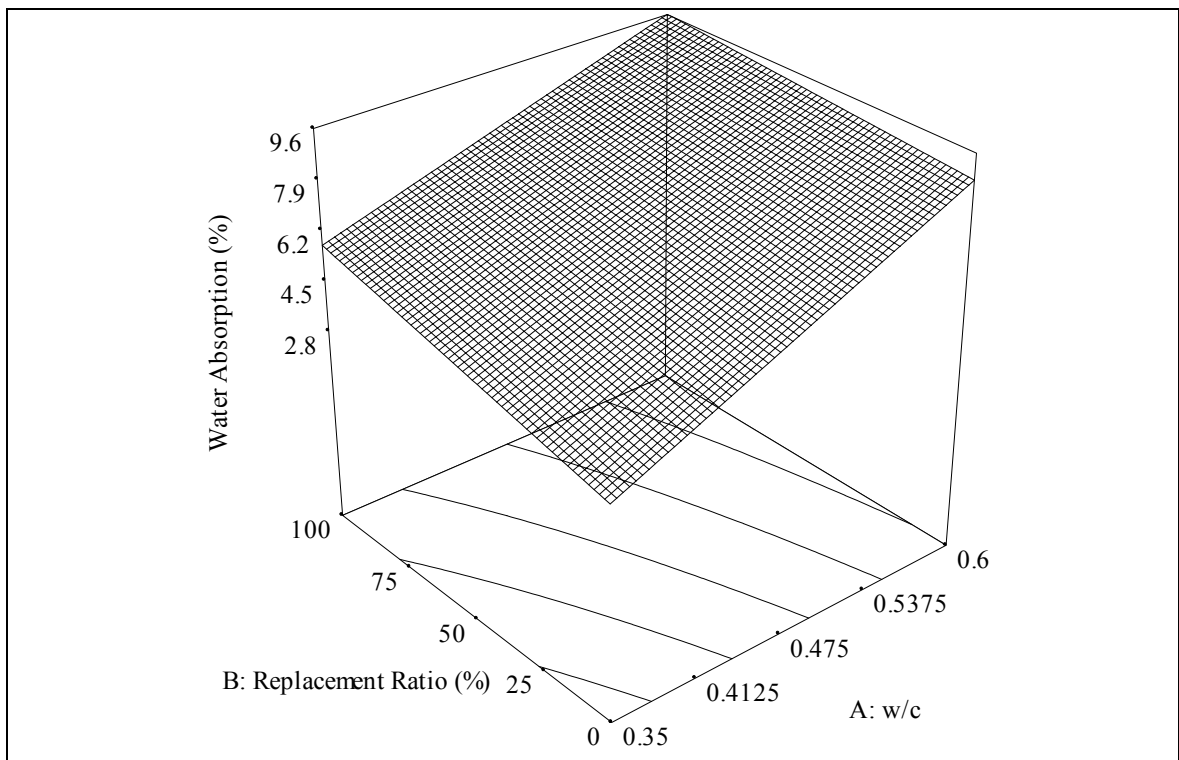


Figure 5.50. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)

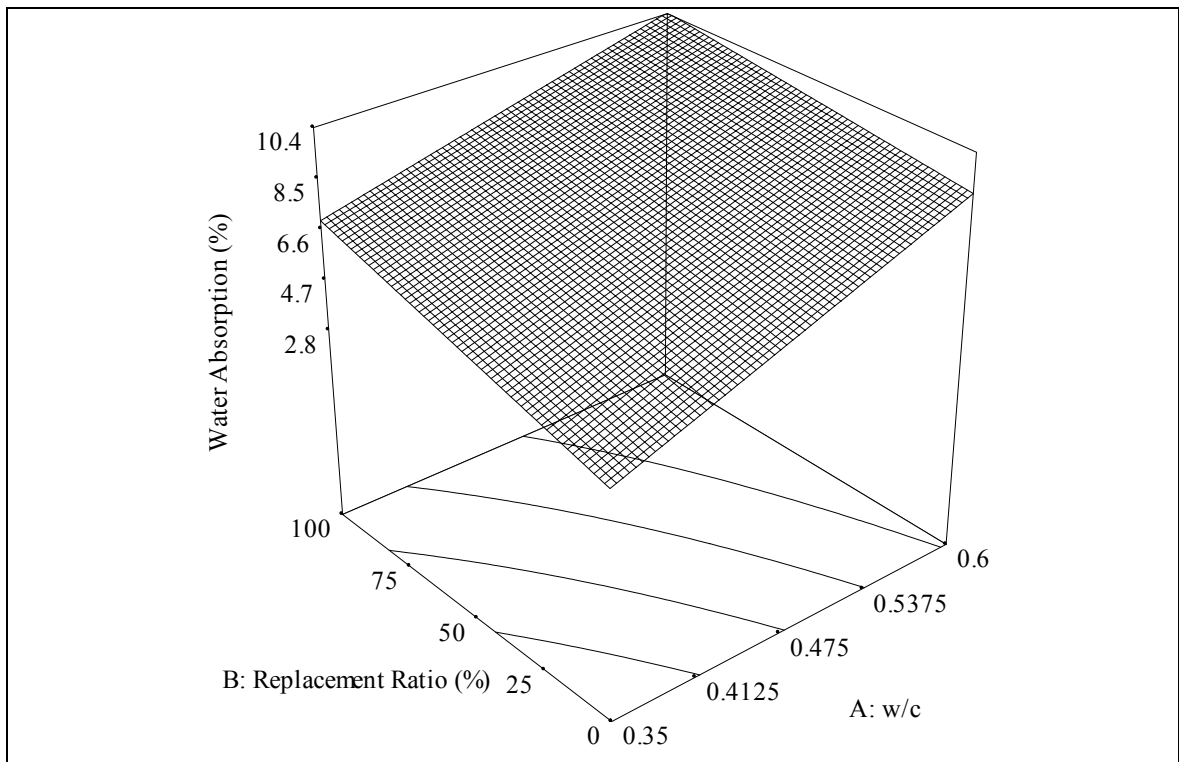


Figure 5.51. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)

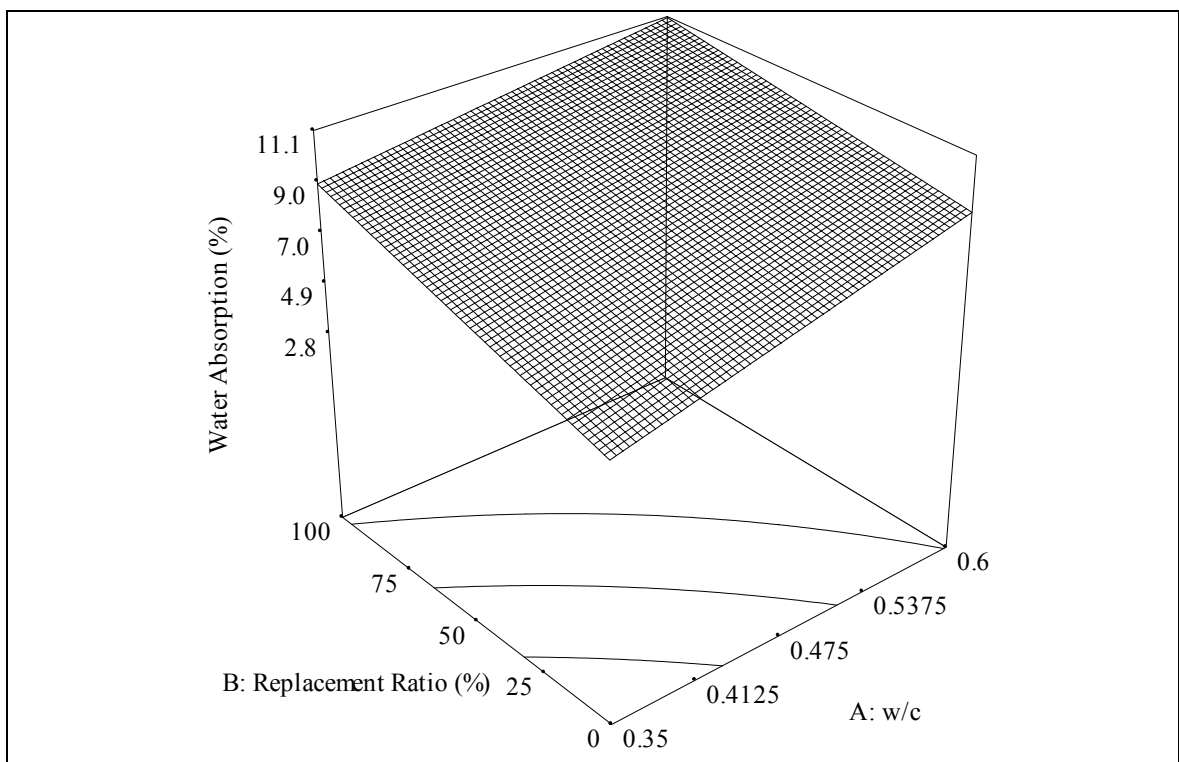


Figure 5.52. 3D surface view of water absorption of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)

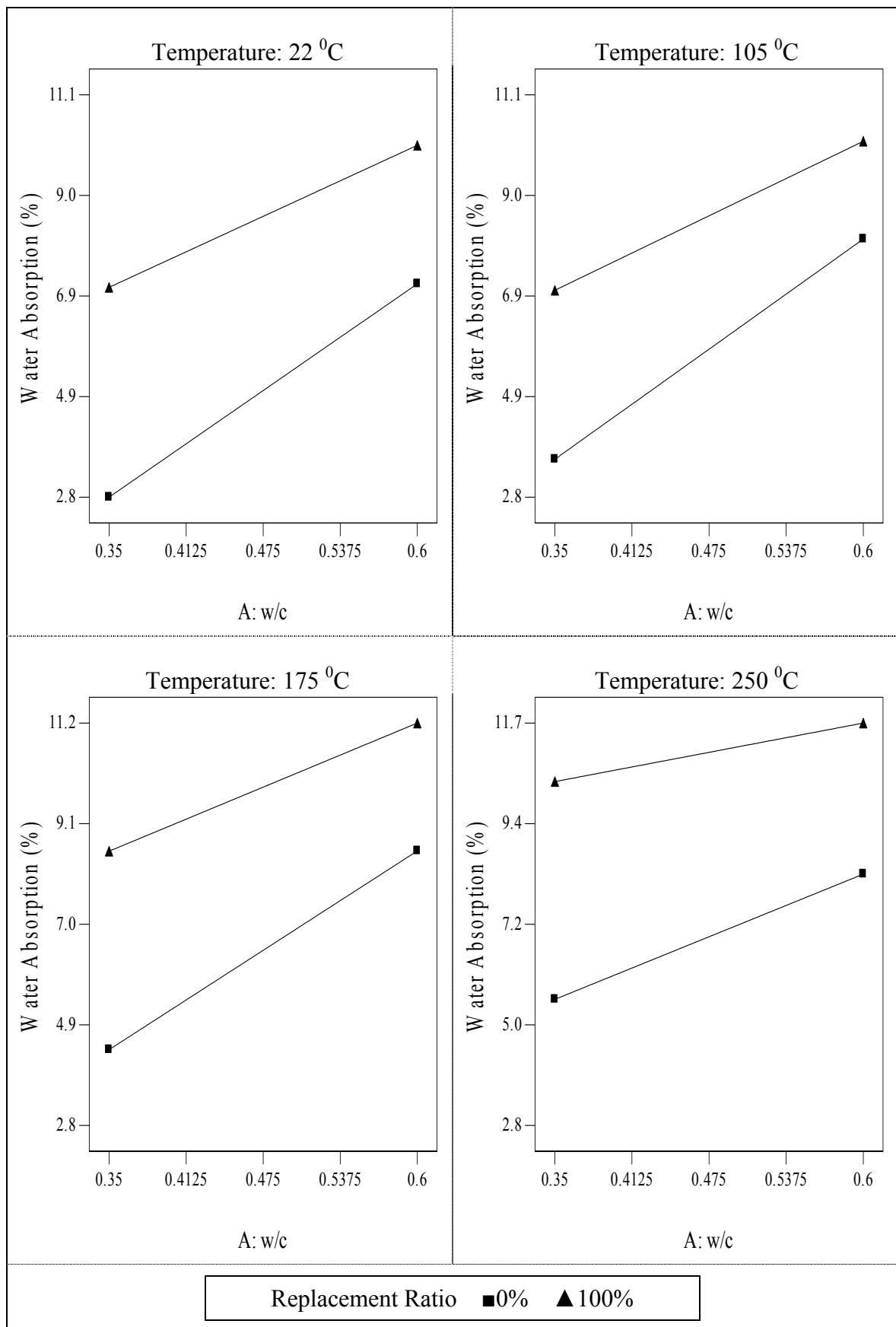


Figure 5.53. Interaction graph of w/c and replacement ratio at 28 days

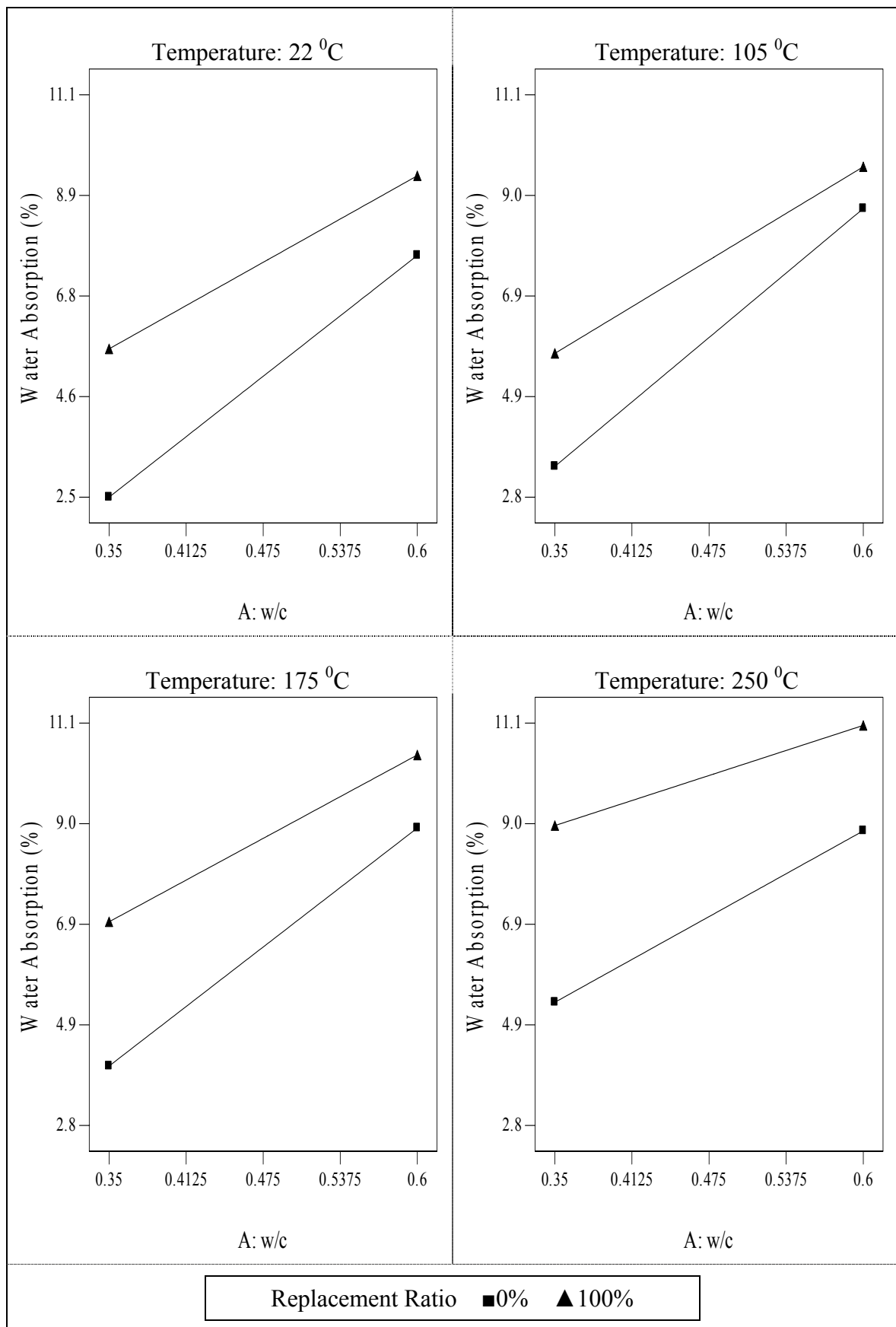


Figure 5.54. Interaction graph of w/c and replacement ratio at 90 days

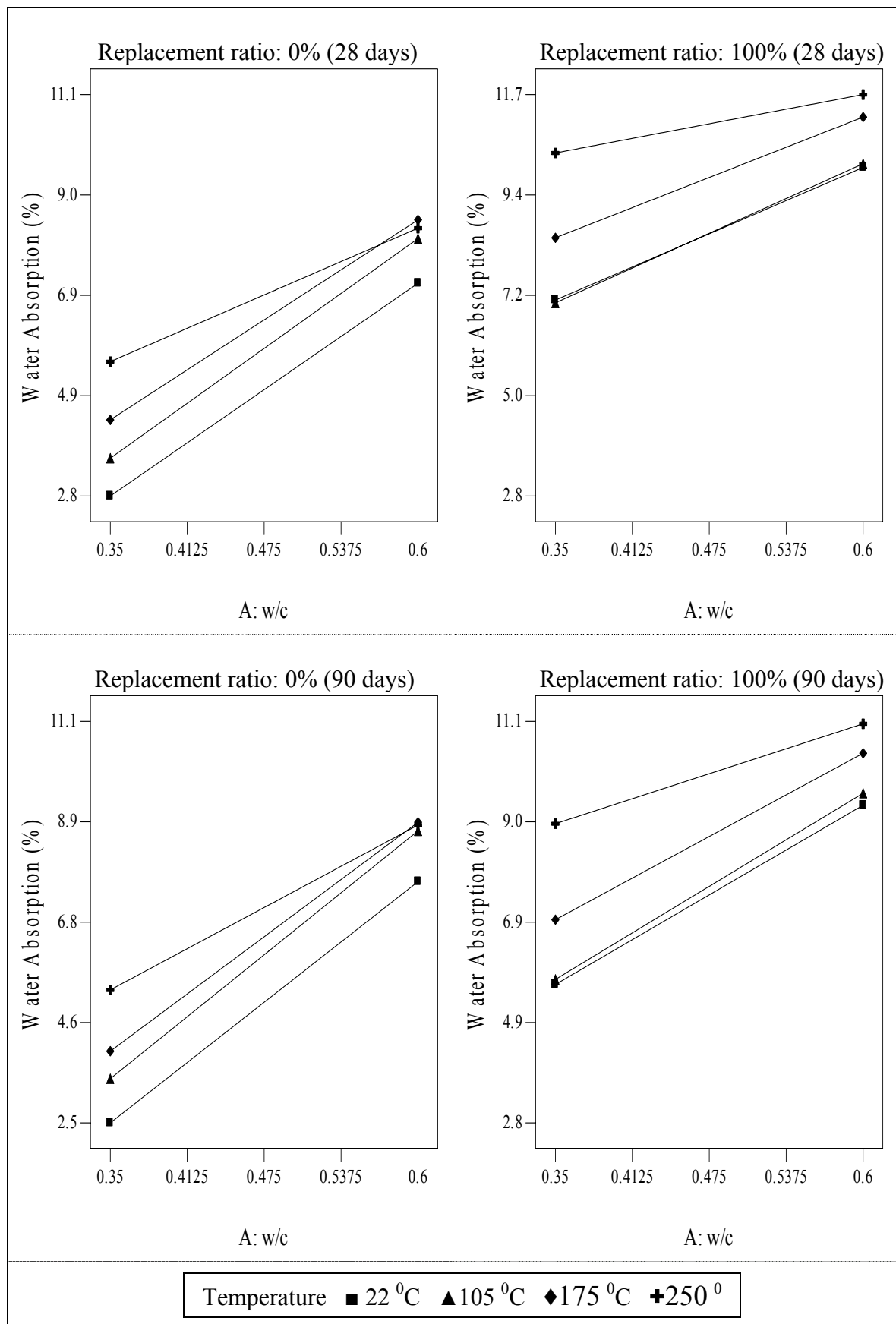


Figure 5.55. Interaction graph of w/c and temperature

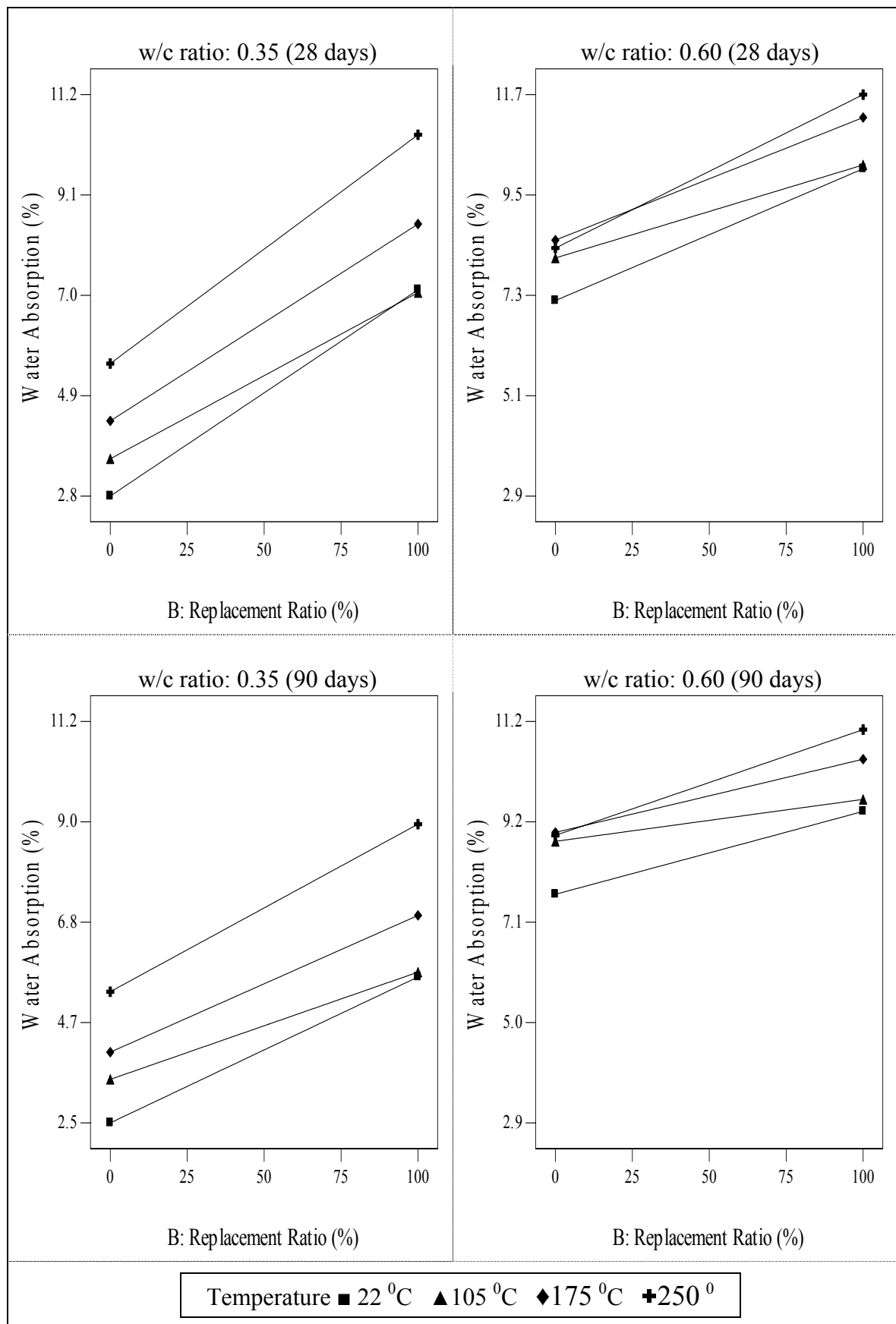


Figure 5.56. Interaction graph of replacement ratio and temperature

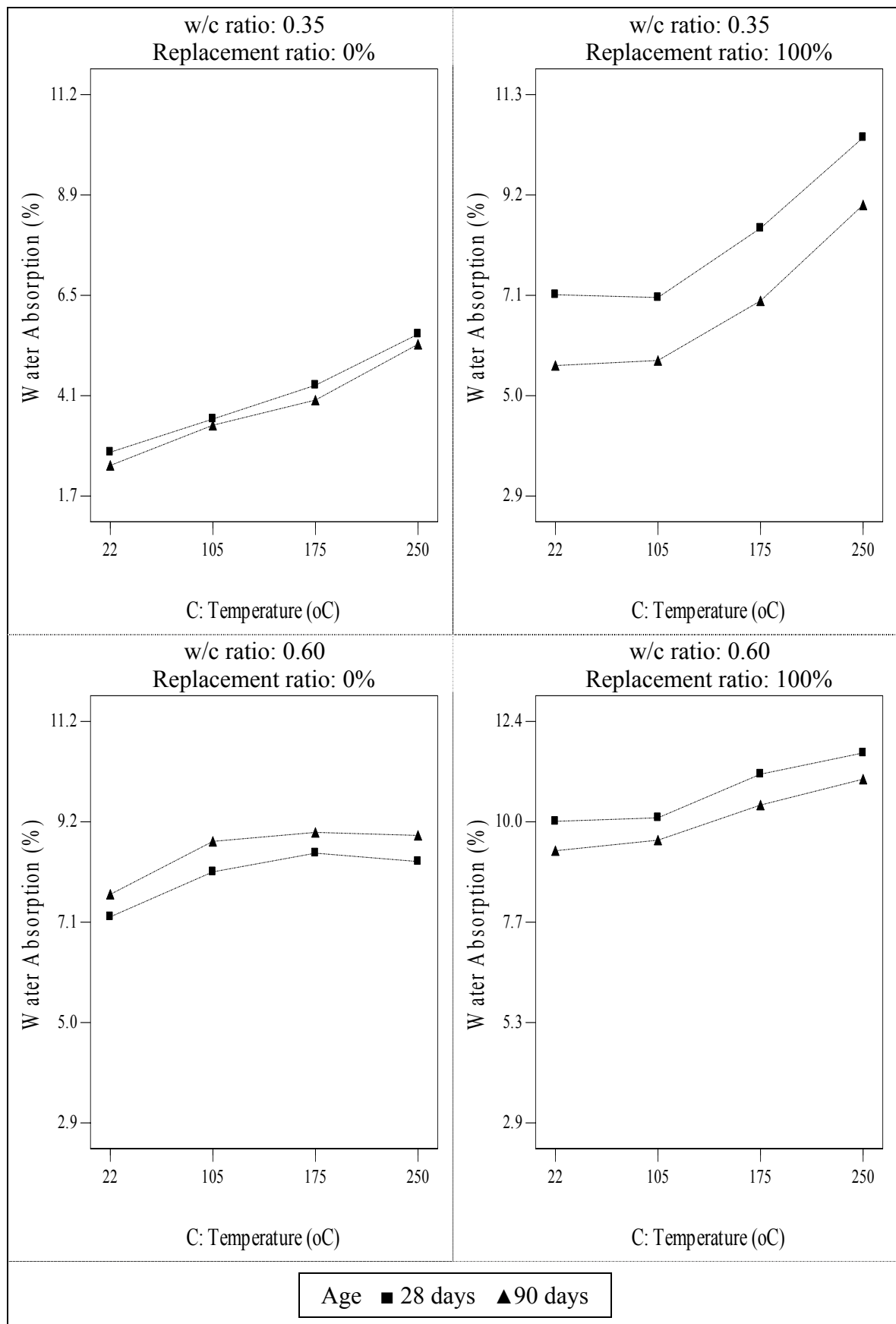


Figure 5.57. Interaction graph of temperature and age

5.4.2. Rapid Chloride Permeability Test

The rapid chloride permeability test (RCPT) results of second stage concretes, mixes 1-31, produced with various water/cement and replacement ratios and tested at 28 and 90 days at room temperature and after exposure to elevated temperatures are given in Table 5.23. 2FI model was used for analysis of rapid chloride permeability test results. The standard deviation and the R-squared values of the model were 1322 and 0.98. The equation of 2FI model for RCPT charge (coulombs) in terms of coded factors was as follows:

$$\begin{aligned}
 \text{RCPT Charge} &= 10850.3 + 3452.5 * A + 2674.3 * B - 2987.3 * C[1] \\
 (\text{Coulombs}) &= -3898.4 * C[2] + 1783.1 * C[3] - 463.2 * D - 9.9 * AB \\
 &= -1252.8 * AC[1] - 172.8 * AC[2] + 1196.5 * AC[3] \\
 &= -547.3 * AD - 617 * BC[1] - 1411.2 * BC[2] \\
 &= +1390.9 * BC[3] + 339.1 * BD - 1065.6 * C[1]D \\
 &= -39.7 * C[2]D - 301.2 * C[3]D
 \end{aligned} \tag{5.6}$$

The coded factors were used in the above equation. The values for coded factors were listed in Tables 5.7-5.10. The RCPT charge values could be calculated with the above equation by using the coded values for each factor. The predicted and actual values of the RCPT charge together with the chloride ion penetrability based on charge passed according to ASTM are seen in Table 5.24. The predicted and actual values of the RCPT charge are illustrated in Figure 5.58; and predicted values of rapid chloride permeability test results are demonstrated in Figures 5.59-5.66. The R-squared value of 0.98 shows that 2FI model is quite good for the RCPT charge prediction. The results of the ANOVA for response surface 2FI model are presented in Table 5.25. As seen from ANOVA table the model terms A, B, C, BC, and CD are significant, because the Prob>F values are less than 0.05. This means that the water/cement ratio (A), the replacement ratio (B), and temperature (C) have important effects on RCPT charge. For the RCPT charge response, the relations between water/cement and replacement ratios are given in Figures 5.67-5.68; the interaction graphs of water/cement ratio and temperature are seen in Figure 5.69; the interaction graphs of replacement ratio and temperature are seen in Figure 5.70; and the interaction graphs of temperature and age are presented in Figure 5.71.

Table 5.23. RCPT charge (coulombs) of second stage natural & recycled aggregate concretes

Run	w/c	RR (%)	T (°C)	Age (days)	RCPT ₁ (Coulombs)	RCPT ₂ (Coulombs)	$\overline{\text{RCPT}}$ (Coulombs)	σ	CV
1	0.35	0	22	90	2618	3679	3149	531	16.8%
2	0.35	0	105	28	2764	2523	2643	121	4.6%
3	0.35	0	105	28	3296	3084	3190	106	3.3%
4	0.35	0	175	28	4280	3757	4019	261	6.5%
5	0.35	0	250	28	6946	6347	6647	300	4.5%
6	0.35	50	22	28	7897	6358	7127	769	10.8%
7	0.35	50	105	90	3274	3312	3293	19	0.6%
8	0.35	100	22	90	6457	5920	6188	268	4.3%
9	0.35	100	105	28	4348	4137	4243	105	2.5%
10	0.35	100	175	90	12148	13307	12728	579	4.6%
11	0.35	100	250	90	17536	16969	17253	284	1.6%
12	0.4125	25	250	90	12504	13657	13080	576	4.4%
13	0.475	0	175	90	7233	6848	7040	193	2.7%
14	0.475	0	175	90	7683	7678	7680	2	0.0%
15	0.475	25	22	28	7454	7762	7608	154	2.0%
16	0.475	50	250	28	18178	17629	17903	275	1.5%
17	0.475	100	105	90	8242	9048	8645	403	4.7%
18	0.475	100	175	28	16643	16750	16696	53	0.3%
19	0.5375	25	175	28	14486	15533	15010	524	3.5%
20	0.5375	50	22	90	7282	7093	7187	94	1.3%
21	0.5375	75	250	90	19589	22504	21046	1457	6.9%
22	0.6	0	22	28	9731	10261	9996	265	2.6%
23	0.6	0	105	90	7337	7306	7322	15	0.2%
24	0.6	0	105	90	7800	7571	7686	115	1.5%
25	0.6	0	250	90	16725	15716	16220	504	3.1%
26	0.6	50	105	28	11011	9957	10484	527	5.0%
27	0.6	50	105	28	11958	11948	11953	5	0.0%
28	0.6	100	22	28	15937	13510	14724	1214	8.2%
29	0.6	100	175	90	18995	21128	20061	1067	5.3%
30	0.6	100	175	90	20223	20441	20332	109	0.5%
31	0.6	100	250	28	21054	19864	20459	595	2.9%

RCPT₁, RCPT₂: RCPT charge values of specimens no 1, 2

$\overline{\text{RCPT}}$: mean of RCPT₁ and RCPT₂

Table 5.24. Actual and predicted values of RCPT charge

Run	w/c	RR (%)	T (°C)	Age (days)	Actual value (coulombs)	Predicted value (coulombs)	Residual (coulombs)	Error	Chloride ion penetrability according to ASTM
1	0.35	0	22	90	3149	2276	873	27.7%	moderate
2	0.35	0	105	28	2643	2694	-51	1.9%	moderate
3	0.35	0	105	28	3190	2694	496	15.5%	moderate
4	0.35	0	175	28	4019	4466	-447	11.1%	high
5	0.35	0	250	28	6647	7798	-1151	17.3%	high
6	0.35	50	22	28	7127	6645	482	6.8%	high
7	0.35	50	105	90	3293	3717	-424	12.9%	moderate
8	0.35	100	22	90	6188	7088	-900	14.5%	high
9	0.35	100	105	28	4243	4562	-319	7.5%	high
10	0.35	100	175	90	12728	12181	547	4.3%	high
11	0.35	100	250	90	17253	17422	-169	1.0%	high
12	0.4125	25	250	90	13080	13501	-421	3.2%	high
13	0.475	0	175	90	7040	7465	-425	6.0%	high
14	0.475	0	175	90	7680	7465	215	2.8%	high
15	0.475	25	22	28	7608	8533	-925	12.2%	high
16	0.475	50	250	28	17903	15010	2893	16.2%	high
17	0.475	100	105	90	8645	8051	594	6.9%	high
18	0.475	100	175	28	16696	17124	-428	2.6%	high
19	0.5375	25	175	28	15010	14135	875	5.8%	high
20	0.5375	50	22	90	7187	7160	27	0.4%	high
21	0.5375	75	250	90	21046	20286	760	3.6%	high
22	0.60	0	22	28	9996	10430	-434	4.3%	high
23	0.60	0	105	90	7322	7589	-267	3.6%	high
24	0.60	0	105	90	7686	7589	97	1.3%	high
25	0.60	0	250	90	16220	16389	-169	1.0%	high
26	0.60	50	105	28	10484	11282	-798	7.6%	high
27	0.60	50	105	28	11953	11282	671	5.6%	high
28	0.60	100	22	28	14724	13847	877	6.0%	high
29	0.60	100	175	90	20061	20365	-304	1.5%	high
30	0.60	100	175	90	20332	20365	-33	0.2%	high
31	0.60	100	250	28	20459	22201	-1742	8.5%	high

As seen from Equation 5.6 the biggest coefficient, 3453, belongs to “A” (water/cement ratio); this highest coefficient and analysis of variance showed that the main factor for chloride ion permeability was water/cement ratio. In Equation 5.6 the coefficient of “B” (replacement ratio) was 2674 which was again a big coefficient in the equation. This higher coefficient and analysis of variance showed that the replacement ratio was a significant for chloride ion permeability, too. As reported in analysis of variance the temperature factor was again very important. It is seen from 3D surface view figures and from interaction figures that chloride ion permeability increased when water/cement ratio increased, the increment was also observed with the increase in recycled aggregate replacement and exposed temperatures after 105 °C. According to the test results chloride ion permeability was higher at early ages. The calculated total charge passed ranged from 2643 coulombs (moderate chloride ion permeability) to 21046 coulombs (high chloride ion permeability). When the water/cement ratio of natural aggregate concretes increased from 0.35 to 0.60, 28 days test results showed that the predicted chloride ion permeability values increased from 4917 to 10430 coulombs at 22 °C, 2694 to 10368 coulombs at 105 °C, from 4466 to 14878 coulombs at 175 °C, and from 7798 to 16253 coulombs at 250 °C. When the water/cement ratio of natural aggregate concretes increased from 0.35 to 0.60, 90 days test results showed that the chloride ion permeability values increased from 2276 to 5600 coulombs at 22 °C, from 2105 to 7589 coulombs at 105 °C, from 3353 to 11576 coulombs at 175 °C, and from 10101 to 16389 coulombs at 250 °C. When the water/cement ratio of concretes, which were produced with fully coarse recycled aggregates, increased from 0.35 to 0.60, 28 days test results showed that the chloride ion permeability values increased from 8373 to 13847 coulombs at 22 °C, from 4562 to 12196 coulombs at 105 °C, from 11938 to 22311 coulombs at 175 °C, and from 13763 to 22201 coulombs at 250 °C. When the water/cement ratio of concretes, which were produced with fully coarse recycled aggregates, increased from 0.35 to 0.60, 90 days test results showed that the chloride ion permeability values increased from 7088 to 10373 coulombs at 22 °C, from 5329 to 10774 coulombs at 105 °C, from 12181 to 20365 coulombs at 175 °C, and from 17422 to 23671 coulombs at 250 °C.

From the test results and analyses and evaluations of the above paragraphs, the following conclusions for chloride ion permeability could be drawn:

1. Resistance of natural aggregate concretes to chloride ion ingress was higher than that of concrete produced with recycled aggregates. Penetration of chloride ions into recycled aggregate concrete was easy because of the physical properties of recycled aggregate. Chloride ions not only diffused from the mortar matrix of concrete but also through the pores of adhered mortar of recycled aggregate. The difficulty of using recycled aggregates as replacement of natural aggregate was its higher porous structure. Moreover, the interfacial transition zones allowed the chloride ions to penetrate into the concrete more easily. Therefore, eventual aggressive environmental agents, such as deleterious salts, may easily penetrate into the concrete [126].
2. Other than recycled aggregate content, increase in water/cement ratio and temperature negatively affected the concretes durability property against chloride ions. The higher the water/cement ratio the higher the risk for chloride ingress. Concretes subjected to elevated temperatures were severely influenced by the chloride ion penetration capacity.
3. Due to the risk of chloride induced corrosion, improvements in recycled aggregate concretes should be made especially for the ones that will serve under chloride environment. Lowering the w/c ratio and replacement ratio will solve the problem to an extent.
4. Rapid chloride permeability test helped to understand the durability property of natural and recycled aggregate concretes at elevated temperatures. Although the water absorption tests gave an idea about the concrete pore volume, they did not give an exact idea for permeability.
5. RCPT charge values increased in the range of 36-285 percent when w/c ratio increased from 0.35 to 0.60. Elevating the temperature from room temperature to 250 °C caused also an increase in RCPT charge values in an amount of 56-344 percent.
6. Based on the ASTM C1202 evaluation, almost all concretes were high class permeability rating.

Table 5.25. Results of ANOVA for RCPT charge response surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	202029070	1	202029070	115.56	<0.0001
B	144802169	1	144802169	82.82	<0.0001
C	357593822	3	119197941	68.18	<0.0001
D	6226470	1	6226470	3.56	0.0836
AB	1121	1	1121	0.00	0.9802
AC	10850770	3	3616923	2.07	0.1580
AD	4224252	1	4224252	2.42	0.1461
BC	25116091	3	8372030	4.79	0.0203
BD	1852727	1	1852727	1.06	0.3236
CD	18674306	3	6224769	3.56	0.0475

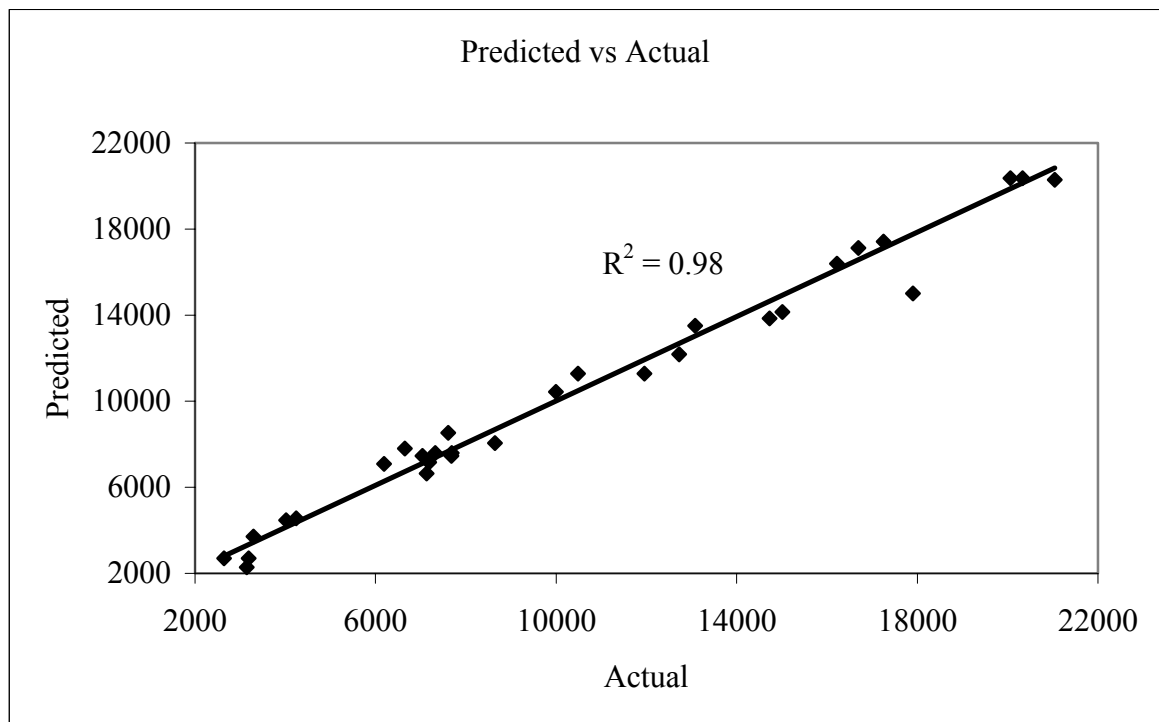


Figure 5.58. Predicted versus actual values of RCPT charge (coulombs)

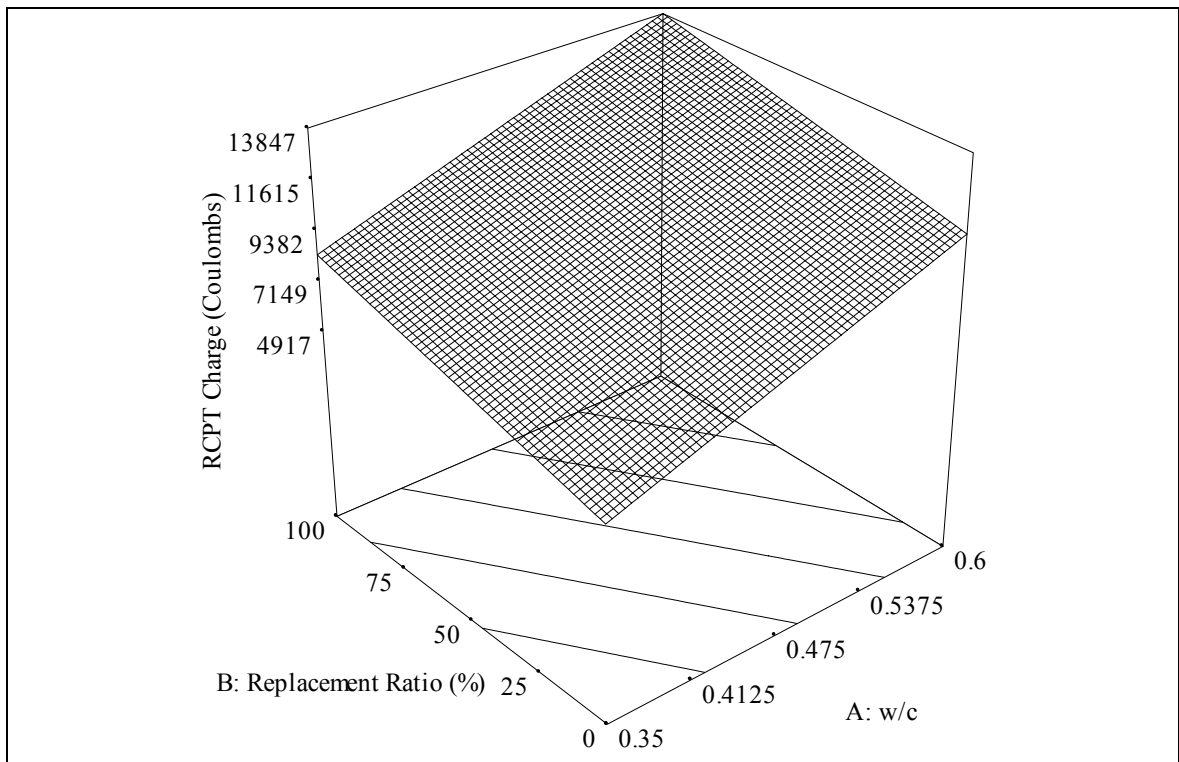


Figure 5.59. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)

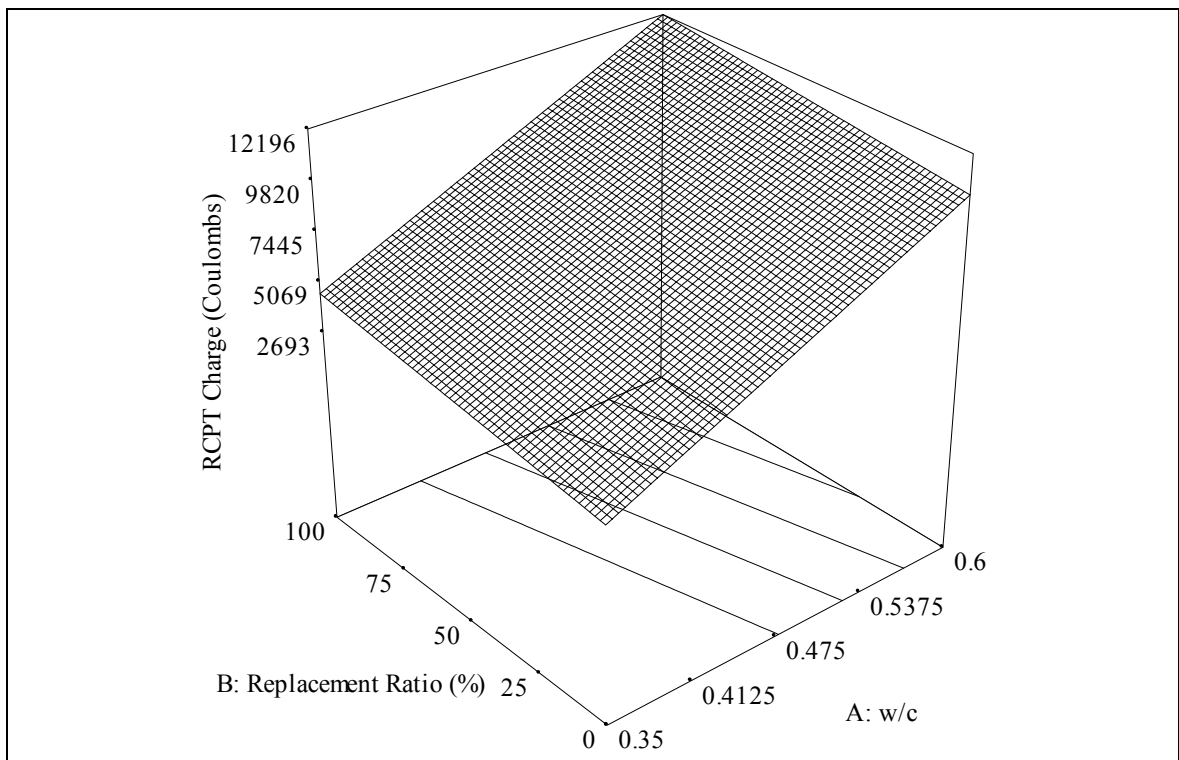


Figure 5.60. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)

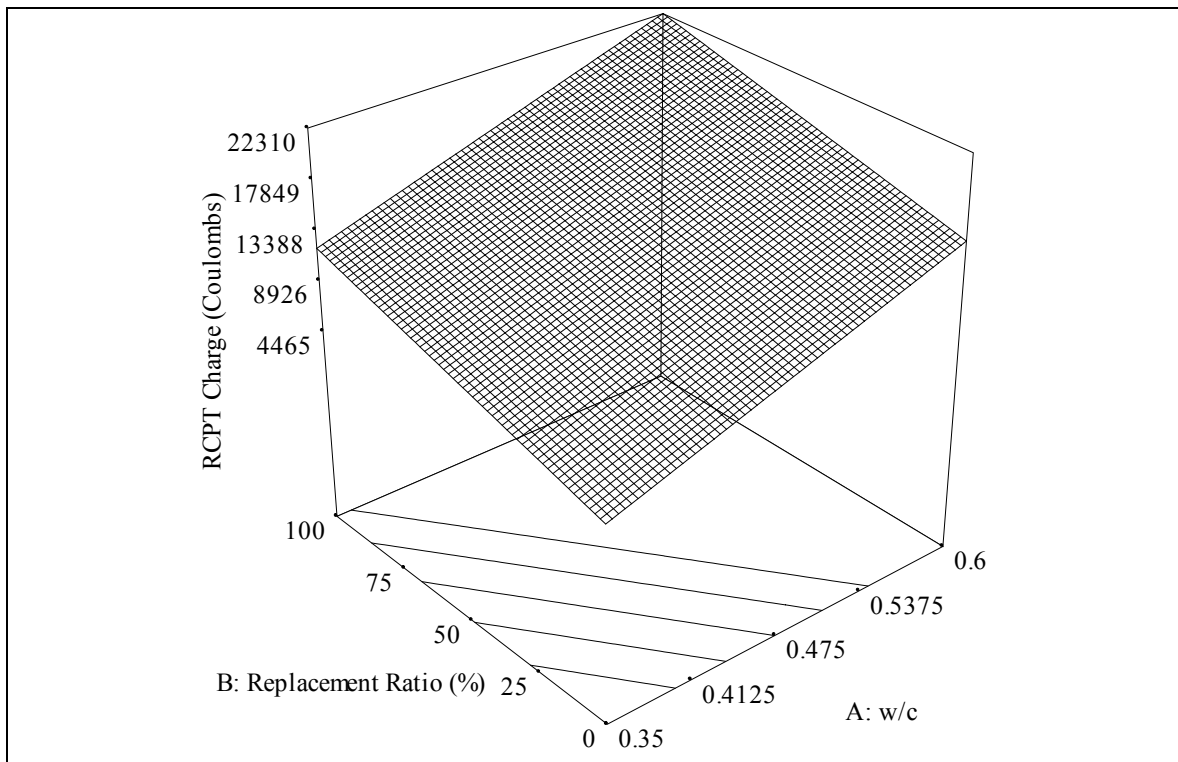


Figure 5.61. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 175 °C)

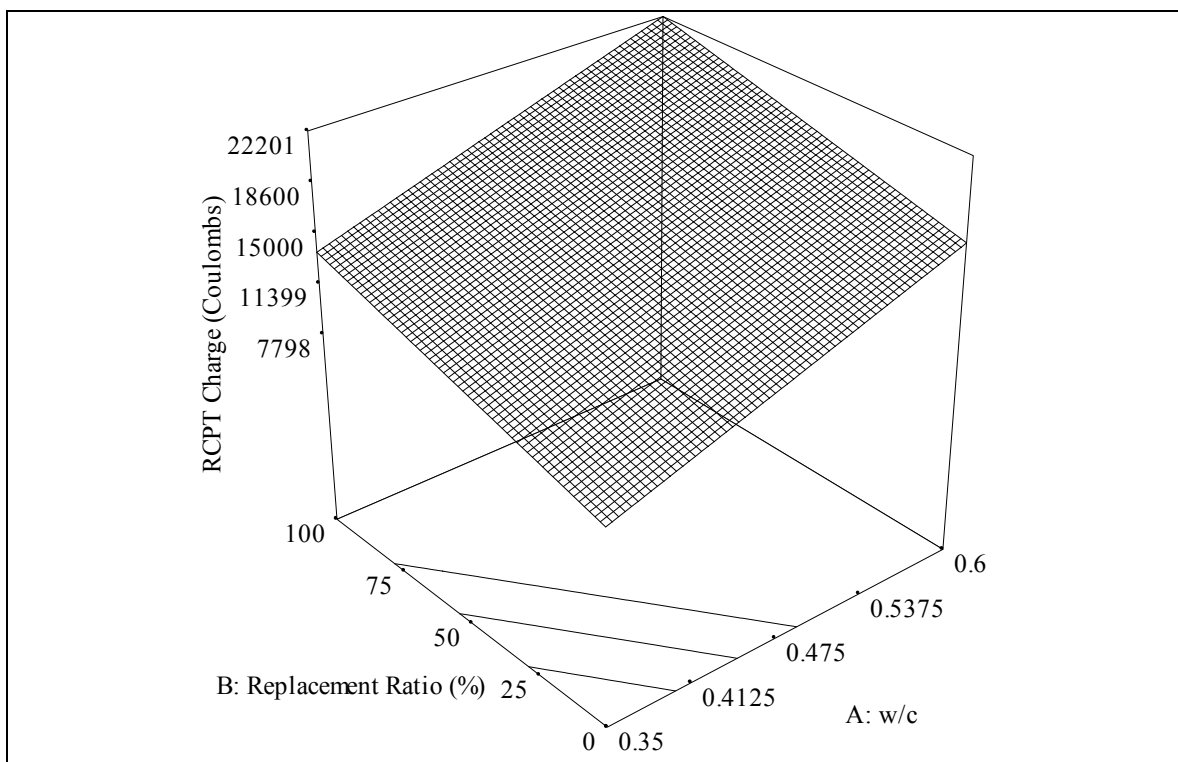


Figure 5.62. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 28 days, temperature: 250 °C)

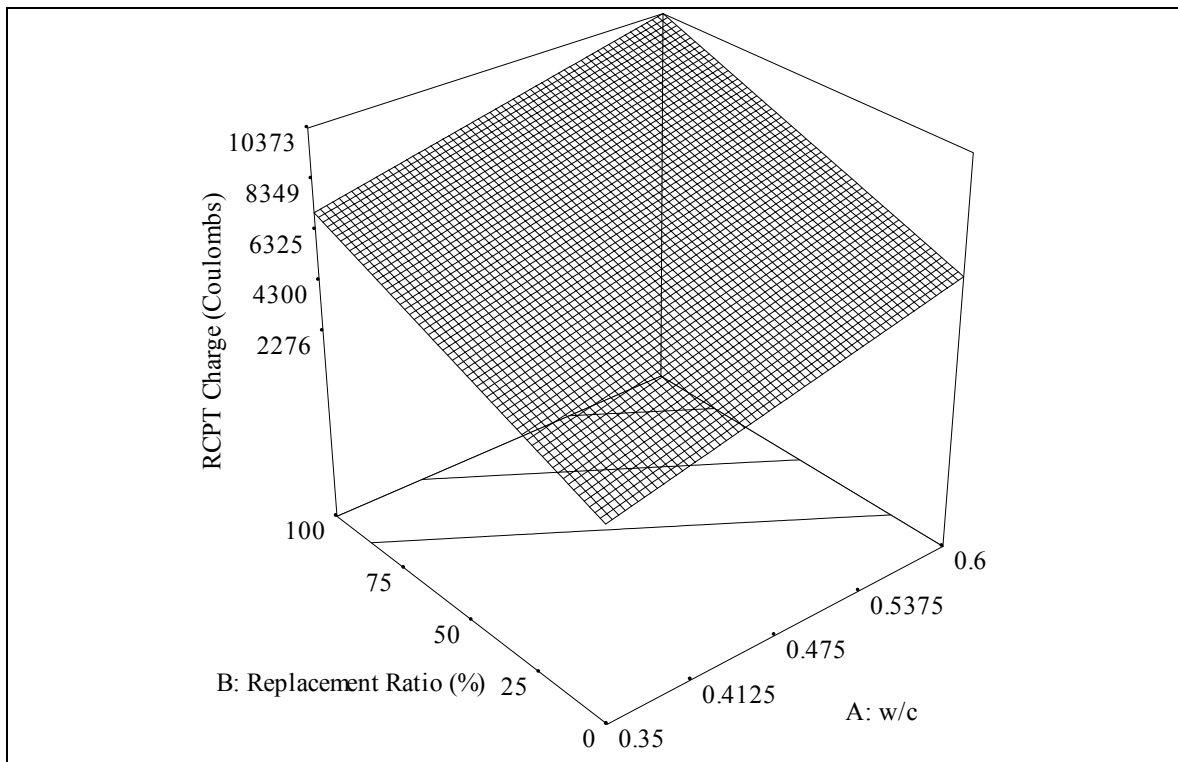


Figure 5.63. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)

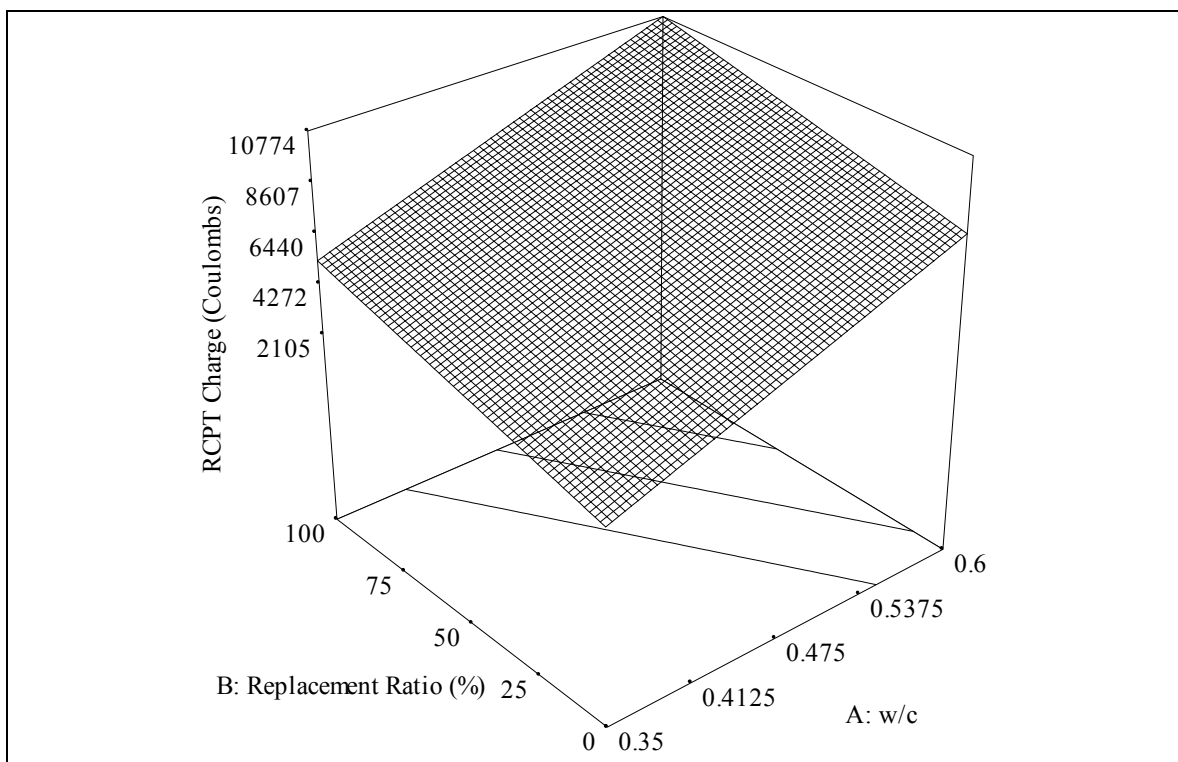


Figure 5.64. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)

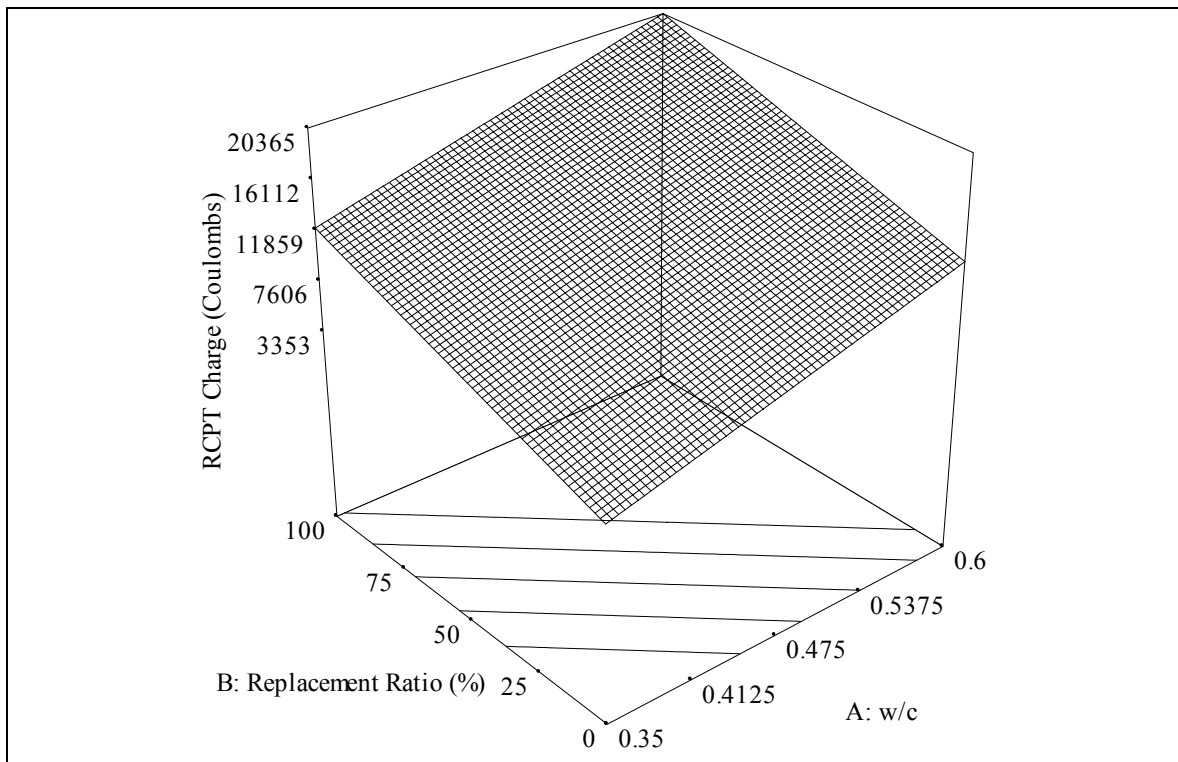


Figure 5.65. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 175 °C)

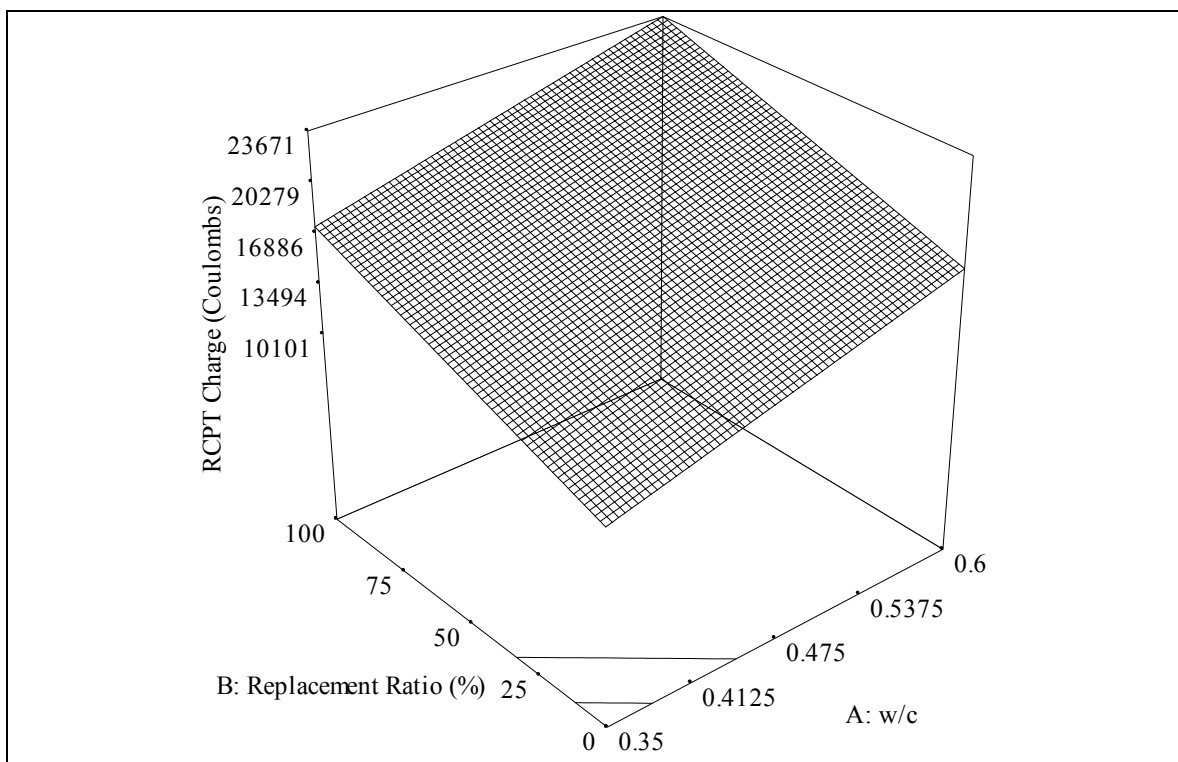


Figure 5.66. 3D surface view of RCPT charge of natural and recycled aggregate concretes (age: 90 days, temperature: 250 °C)

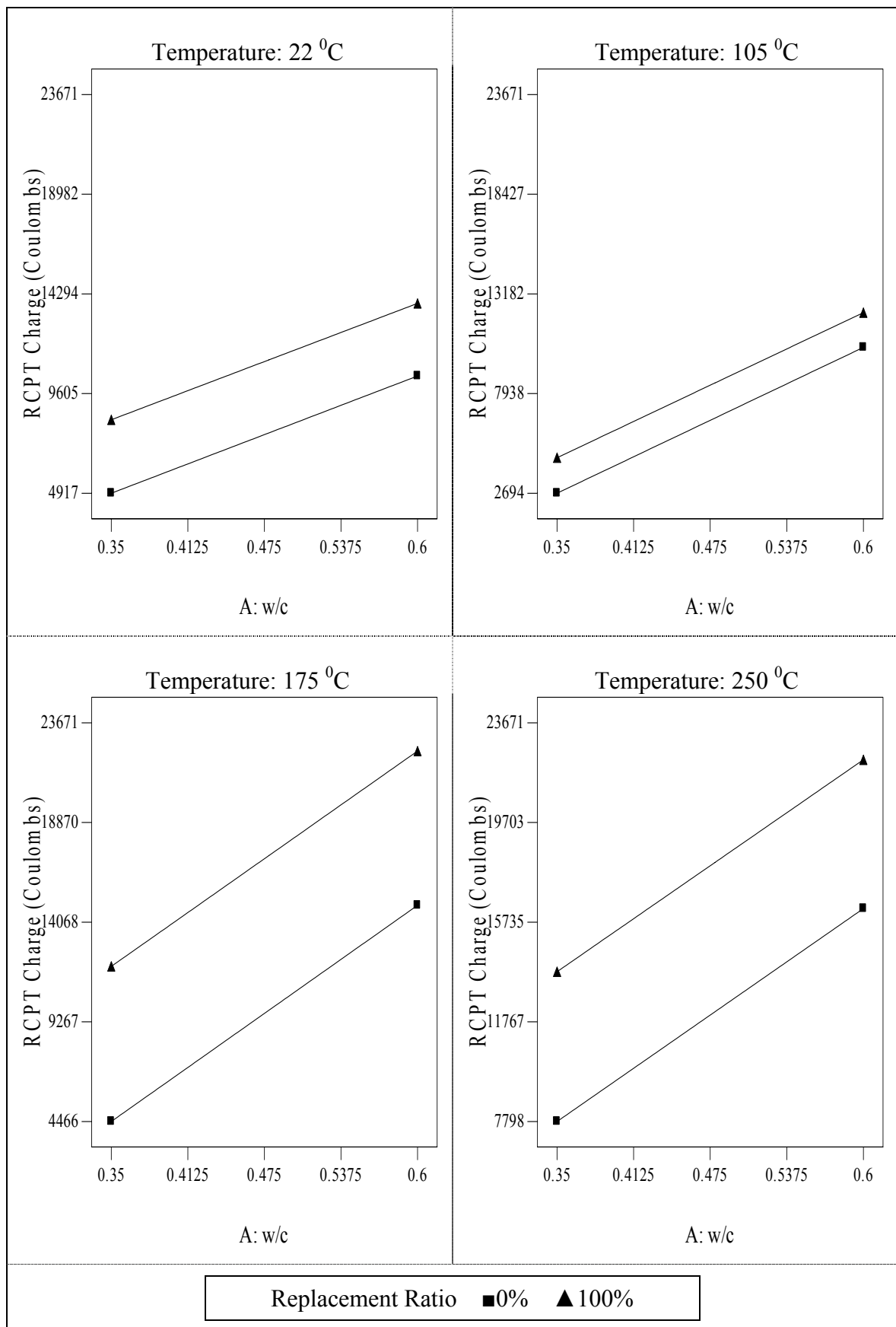


Figure 5.67. Interaction graph of w/c and replacement ratio at 28 days

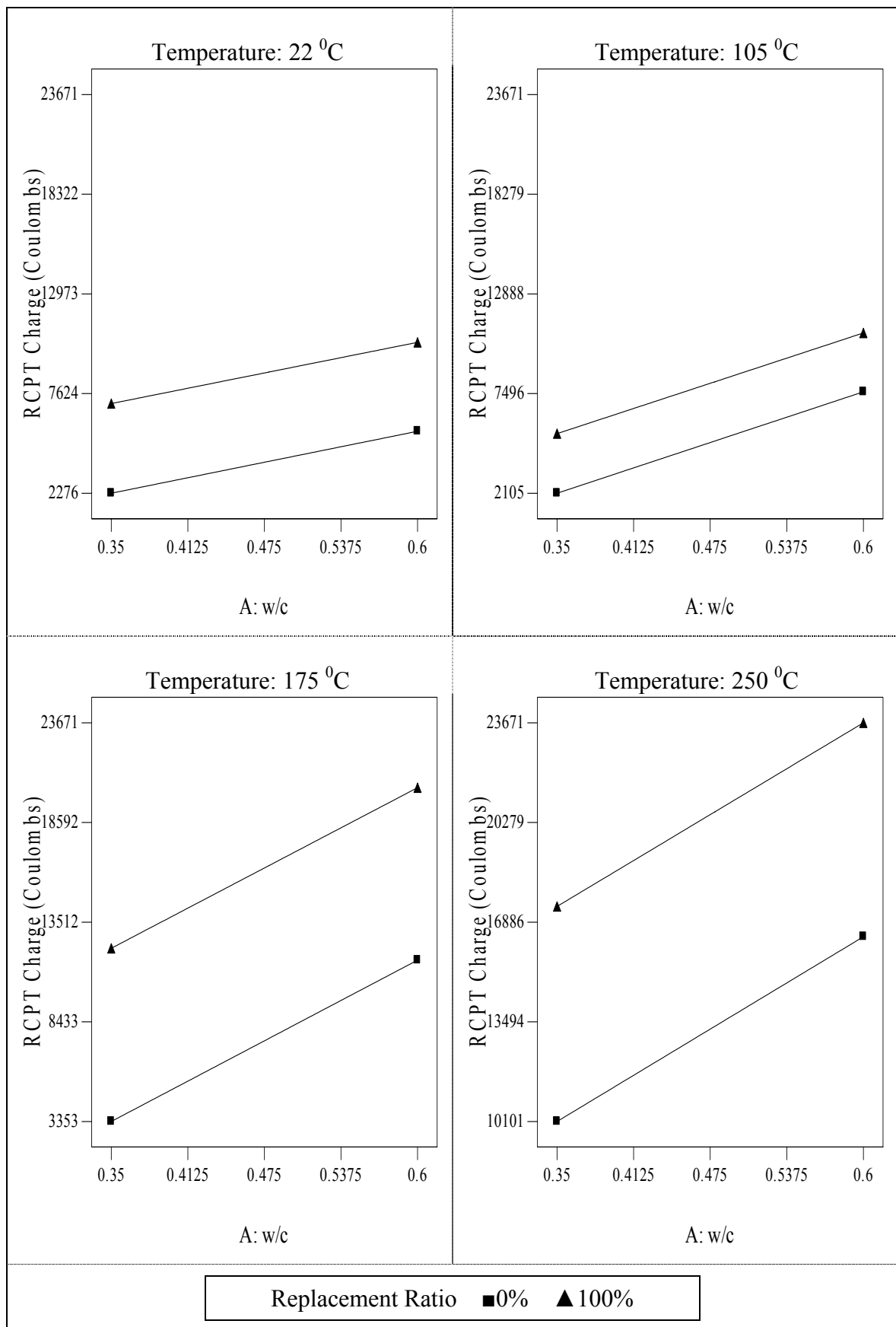


Figure 5.68. Interaction graph of w/c and replacement ratio at 90 days

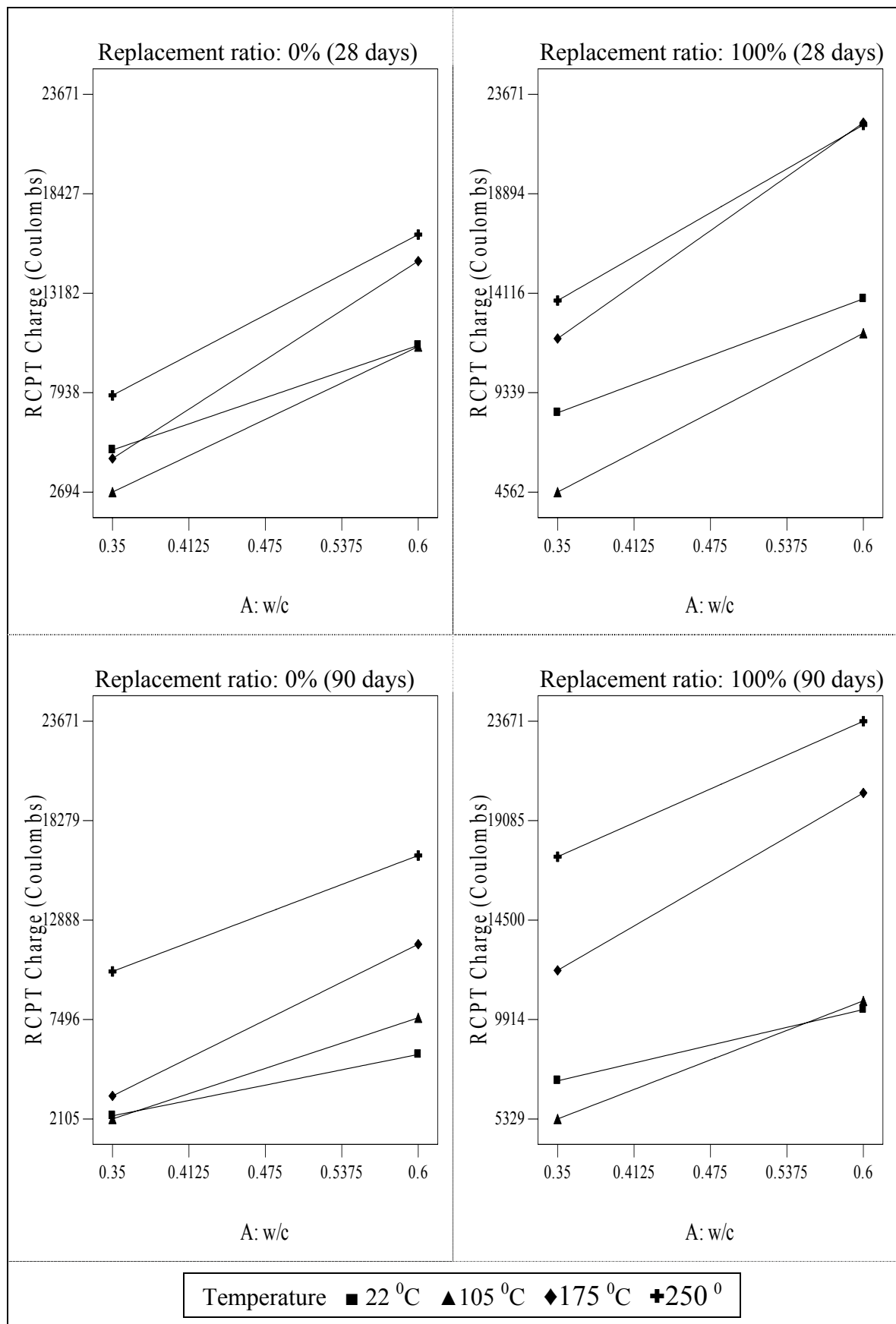


Figure 5.69. Interaction graph of w/c and temperature

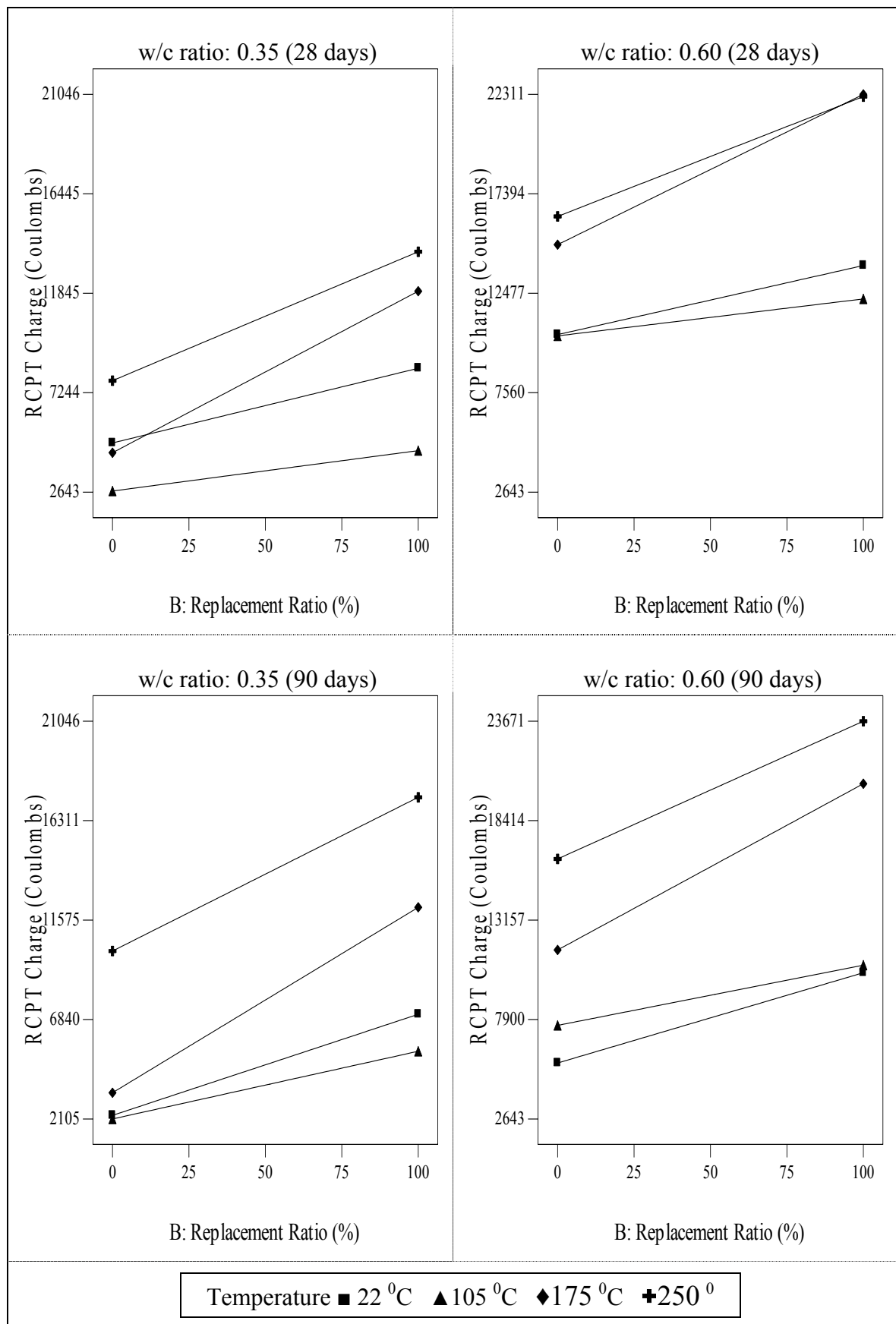


Figure 5.70. Interaction graph of replacement ratio and temperature

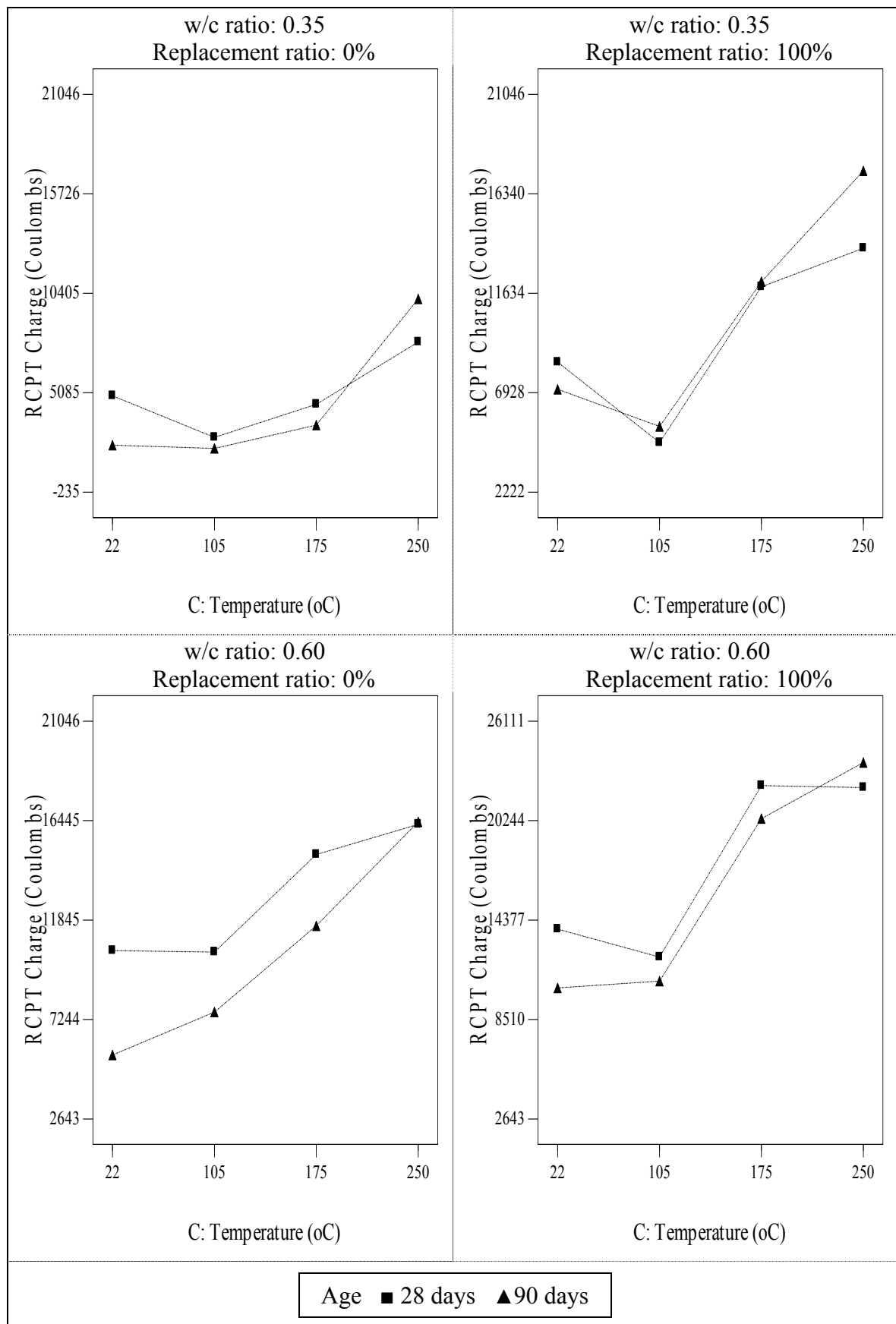


Figure 5.71. Interaction graph of temperature and age

5.4.3. Water Absorption (Sorptivity)

The initial and secondary water absorption (sorptivity) results of second stage concretes, mixes 1-31, produced with various water/cement and replacement ratios and tested at 28 and 90 days at room temperature and after exposure to elevated temperatures are given in Table 5.26. The sorptivity values ranged from 0.0002 to 0.0355 mm/sqrt (sec); ratio of max to min is 177.5; a ratio greater than 10 usually indicates a transformation is required for improved analyses. Transformations apply a mathematical function to all response data. Base 10 Log was used for the sorptivity data transformation. Linear model was used for analysis of sorptivity results. Linear model was as follows:

$$R = I + c_1A + c_2B + c_3C + c_4D \quad (5.7)$$

where:

R= response (sorptivity)

I= intercept

A= water-cement ratio, w/c

B= replacement ratio

C= temperature

D= age

c_1, c_2, c_3, c_4 = coefficients

The standard deviation and the R-squared values of the model were 0.25 and 0.87. The equation of linear model for sorptivity in terms of coded factors was as follows:

$$\begin{aligned} \text{Log}_{10}(\text{sorptivity, mm/sqrt (sec)}) = & -2.43 + 0.33 * A + 0.13 * B - 0.61 * C[1] \\ & - 0.24 * C[2] + 0.34 * C[3] + 0.057 * D \end{aligned} \quad (5.8)$$

The coded factors were used in the above equation. The values for coded factors were listed in Tables 5.7-5.10. The sorptivity values could be calculated with the above equation by using the coded values for each factor. The predicted and actual values of the sorptivity are seen in Table 5.27.

Table 5.26. Initial and secondary water absorption (sorptivity) of second stage natural and recycled aggregate concretes

Run	w/c	RR (%)	T (°C)	Age (days)	Initial absorption		Secondary absorption	
					S_i , mm/sqrt(sec)	R^2	S_s , mm/sqrt(sec)	R^2
1	0.35	0	22	90	0.0002	0.98	0.0001	0.98
2	0.35	0	105	28	0.0008	0.98	0.0002	0.94
3	0.35	0	105	28	0.0002	0.98	0.0002	0.99
4	0.35	0	175	28	0.0032	0.99	0.0022	0.98
5	0.35	0	250	28	0.0065	1	0.0036	0.97
6	0.35	50	22	28	0.0006	0.98	0.0001	0.99
7	0.35	50	105	90	0.0009	0.98	0.0006	0.98
8	0.35	100	22	90	0.0008	0.96	0.0001	0.98
9	0.35	100	105	28	0.0007	0.97	0.0008	0.99
10	0.35	100	175	90	0.0096	1	0.0027	0.98
11	0.35	100	250	90	0.0132	1	0.0045	0.99
12	0.4125	25	250	90	0.0102	1	0.0037	1
13	0.475	0	175	90	0.0059	1	0.0022	0.99
14	0.475	0	175	90	0.0039	0.99	0.0024	0.98
15	0.475	25	22	28	0.0011	0.98	0.0004	0.99
16	0.475	50	250	28	0.0032	0.98	0.0041	0.98
17	0.475	100	105	90	0.0035	1	0.0028	0.99
18	0.475	100	175	28	0.0179	1	0.0011	0.99
19	0.5375	25	175	28	0.0137	1	0.0010	0.99
20	0.5375	50	22	90	0.0020	0.99	0.0012	99
21	0.5375	75	250	90	0.0200	1	0.0013	1
22	0.6	0	22	28	0.0010	0.99	0.0004	0.99
23	0.6	0	105	90	0.0074	0.98	0.0019	0.99
24	0.6	0	105	90	0.0060	0.99	0.0026	0.99
25	0.6	0	250	90	0.0199	1	0.0006	0.98
26	0.6	50	105	28	0.0042	1	0.0019	0.98
27	0.6	50	105	28	0.0065	1	0.0029	0.98
28	0.6	100	22	28	0.0012	1	0.0016	0.98
29	0.6	100	175	90	0.0134	1	0.0017	0.99
30	0.6	100	175	90	0.0185	1	0.0015	0.98
31	0.6	100	250	28	0.0355	1	0.0005	0.99

S_i : initial sorptivity, S_s : secondary sorptivity

Table 5.27. Actual and predicted values of sorptivity

Run	w/c	RR (%)	T (°C)	Age (days)	Actual value (Log ₁₀ (Si))	Predicted value (Log ₁₀ (Si))	Residual (Log ₁₀ (Si))	Error
1	0.35	0	22	90	-3.70	-3.44	-0.26	7.0%
2	0.35	0	105	28	-3.10	-3.18	0.08	2.6%
3	0.35	0	105	28	-3.70	-3.18	-0.52	14.1%
4	0.35	0	175	28	-2.49	-2.60	0.10	4.0%
5	0.35	0	250	28	-2.19	-2.43	0.24	11.0%
6	0.35	50	22	28	-3.22	-3.43	0.21	6.5%
7	0.35	50	105	90	-3.05	-2.94	-0.10	3.3%
8	0.35	100	22	90	-3.10	-3.19	0.09	2.9%
9	0.35	100	105	28	-3.15	-2.93	-0.22	7.0%
10	0.35	100	175	90	-2.02	-2.23	0.21	10.4%
11	0.35	100	250	90	-1.88	-2.07	0.19	10.1%
12	0.4125	25	250	90	-1.99	-2.09	0.10	5.0%
13	0.475	0	175	90	-2.23	-2.16	-0.07	3.1%
14	0.475	0	175	90	-2.41	-2.16	-0.25	10.4%
15	0.475	25	22	28	-2.96	-3.16	0.21	7.1%
16	0.475	50	250	28	-2.49	-1.98	-0.51	20.5%
17	0.475	100	105	90	-2.46	-2.49	0.04	1.6%
18	0.475	100	175	28	-1.75	-2.02	0.27	15.4%
19	0.5375	25	175	28	-1.86	-2.05	0.18	9.7%
20	0.5375	50	22	90	-2.70	-2.82	0.13	4.8%
21	0.5375	75	250	90	-1.70	-1.64	-0.06	3.5%
22	0.60	0	22	28	-3.00	-2.90	-0.10	3.3%
23	0.60	0	105	90	-2.13	-2.42	0.28	13.1%
24	0.60	0	105	90	-2.22	-2.42	0.19	8.6%
25	0.60	0	250	90	-1.70	-1.66	-0.04	2.4%
26	0.60	50	105	28	-2.38	-2.40	0.03	1.3%
27	0.60	50	105	28	-2.19	-2.40	0.22	10.0%
28	0.60	100	22	28	-2.92	-2.65	-0.27	9.2%
29	0.60	100	175	90	-1.87	-1.58	-0.29	15.5%
30	0.60	100	175	90	-1.73	-1.58	-0.15	8.7%
31	0.60	100	250	28	-1.45	-1.53	0.08	5.5%

The predicted and actual values of the sorptivity are illustrated in Figure 5.72; and predicted values of sorptivity results are demonstrated in Figures 5.73-5.80. The R-squared value of 0.87 shows that even linear model is not so good for the sorptivity prediction. The results of the ANOVA for response surface linear model are presented in Table 5.28. As seen from ANOVA table the model terms A, B, and C are significant, because the Prob>F values are less than 0.05. This means that the water/cement ratio (A), the replacement ratio

(B), and temperature (C) have important effects on sorptivity. For the sorptivity response, the relations between water/cement and replacement ratios are given in Figures 5.81-5.82; and the interaction graphs of temperature and age are presented in Figures 5.83-5.86.

As seen from Equation 5.8 and from ANOVA results water/cement ratio (A), replacement ratio (B) and temperature (C) factors were all significant factors for sorptivity. It is seen from 3D surface view figures and from interaction figures that sorptivity increased when water/cement ratio increased, the increment was also observed with the increase in recycled aggregate replacement and exposure temperatures. According to the test results sorptivity was lower at early ages. The logarithmic values of test results of sorptivity ($\text{mm/s}^{1/2}$) in base 10 ranged from -3.70 (lowest value) to -1.45 (highest sorptivity value). When the water/cement ratio of natural aggregate concretes increased from 0.35 to 0.60, 28 days test results showed that the logarithmic values of sorptivity ($\text{mm/s}^{1/2}$) in base 10 increased from -3.55 to -2.90 at 22 °C, from -3.18 to -2.53 at 105 °C, from -2.60 to -1.94 at 175 °C, and from -2.43 to -1.77 at 250 °C. When the water/cement ratio of natural aggregate concretes increased from 0.35 to 0.60, 90 days test results showed that the logarithmic values of sorptivity ($\text{mm/s}^{1/2}$) in base 10 increased from -3.44 to -2.79 at 22 °C, from -3.07 to -2.42 at 105 °C, from -2.48 to -1.83 at 175 °C, and from -2.32 to -1.66 at 250 °C. When the water/cement ratio of concretes, which were produced with fully coarse recycled aggregates, increased from 0.35 to 0.60, 28 days test results showed that the logarithmic values of sorptivity ($\text{mm/s}^{1/2}$) in base 10 increased from -3.30 to -2.65 at 22 °C, from -2.93 to -2.28 at 105 °C, from -2.35 to -1.69 at 175 °C, and from -2.18 to -1.53 at 250 °C. When the water/cement ratio of concretes, which were produced with fully coarse recycled aggregates, increased from 0.35 to 0.60, 90 days test results showed that the logarithmic values of sorptivity ($\text{mm/s}^{1/2}$) in base 10 increased from -3.19 to -2.54 at 22 °C, from -2.82 to -2.17 at 105 °C, from -2.23 to -1.58 at 175 °C, and from -2.07 to -1.41 at 250 °C.

Based on the test results and analyses and comparisons of the above paragraphs, it could be concluded that the water absorption (sorptivity) values depend on age and replacement ratio factors and to a large extent on w/c ratio and temperature factors. Sorptivity values increased in the range of 22-47 percent when w/c ratio increased from 0.35 to 0.60. Elevating the temperature from room temperature to 250 °C caused also a

raise in sorptivity values in an amount of 46-80 percent. Although the incorporation of recycled aggregate into concretes increased the sorptivity values, the increase was not more than 18 percent. To finish, recycled aggregates could be used in concretes safely by lowering the water/cement ratio if the sorptivity parameter was the main issue.

Pore structure, distribution of the pores, and the connections between the pores were important parameters for sorptivity test. So in order to predict the durability property it is important to decide what property is needed, since there is not unique test method. Water absorption by immersion was an easy test and gave idea about the total pore volume; rapid chloride permeability test was a quick way to understand the permeability characteristics. From this aspect sorptivity test represents the transport mechanism that usually occurs in concrete structures exposed to repeated wetting and drying cycles, and it is important to protect the embedded reinforcement with concrete cover.

Table 5.28. Results of ANOVA for sorptivity response surface linear model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	2.32	1	2.32	37.53	<0.0001
B	0.33	1	0.33	5.39	0.0291
C	5.74	3	1.91	30.91	<0.0001
D	0.10	1	0.10	1.59	0.2195

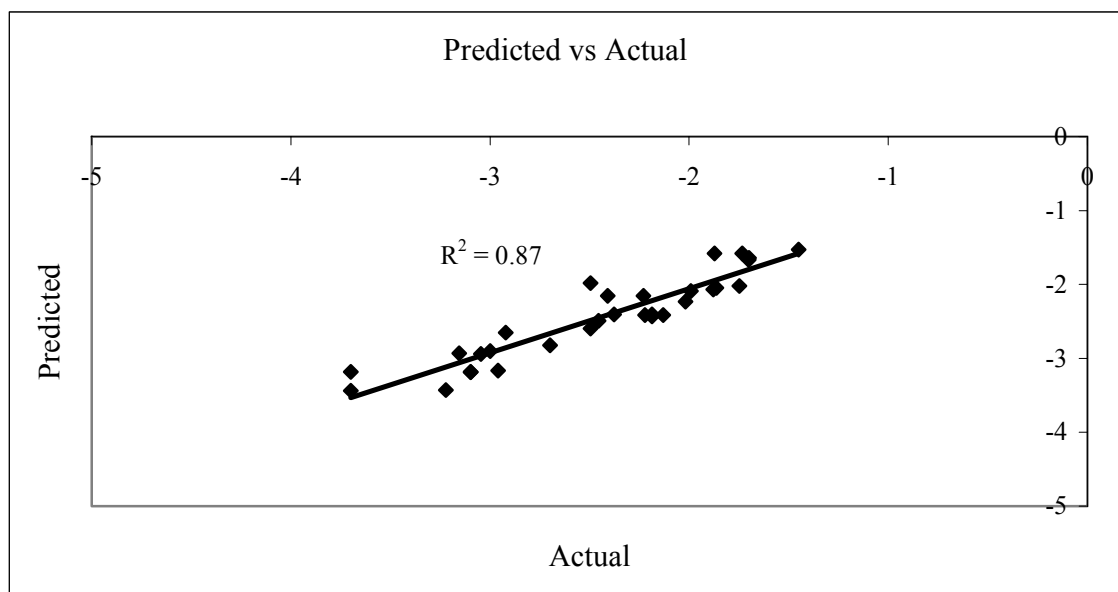


Figure 5.72. Predicted versus actual values of log10 sorptivity (mm/sqrt (sec))

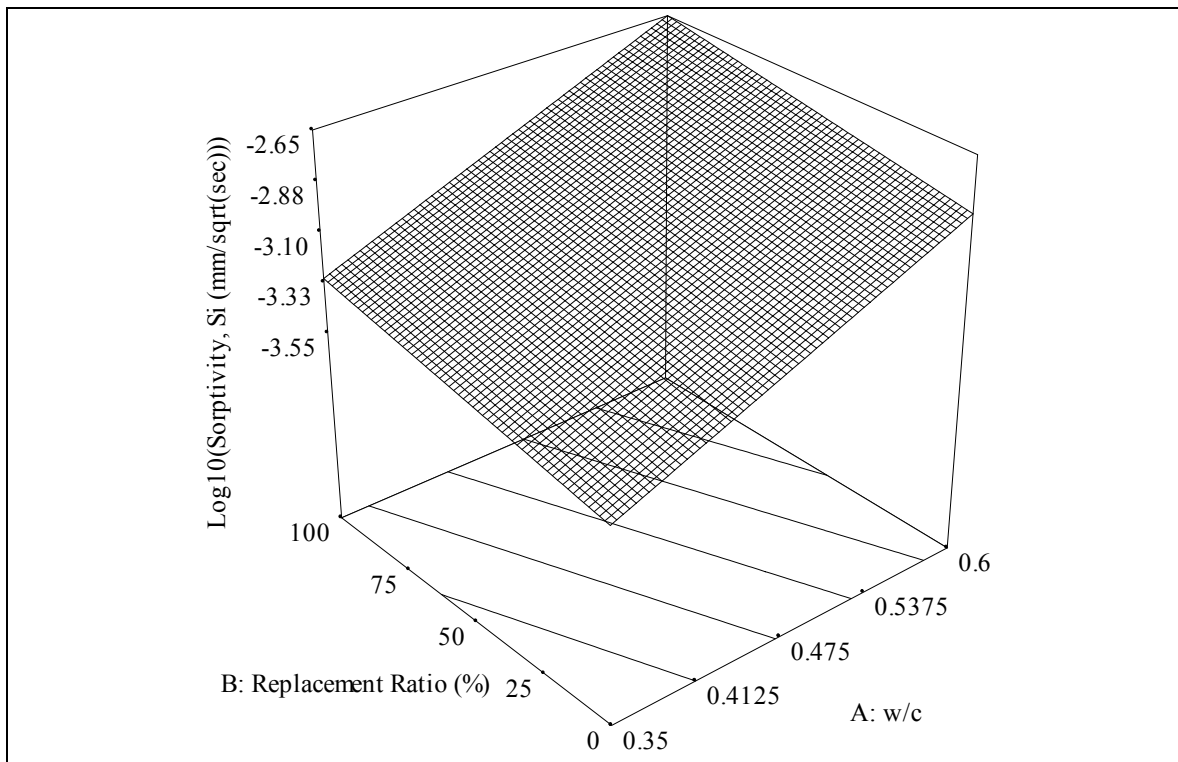


Figure 5.73. 3D surface view of sorptivity of natural and recycled aggregate concretes (age: 28 days, temperature: 22 °C)

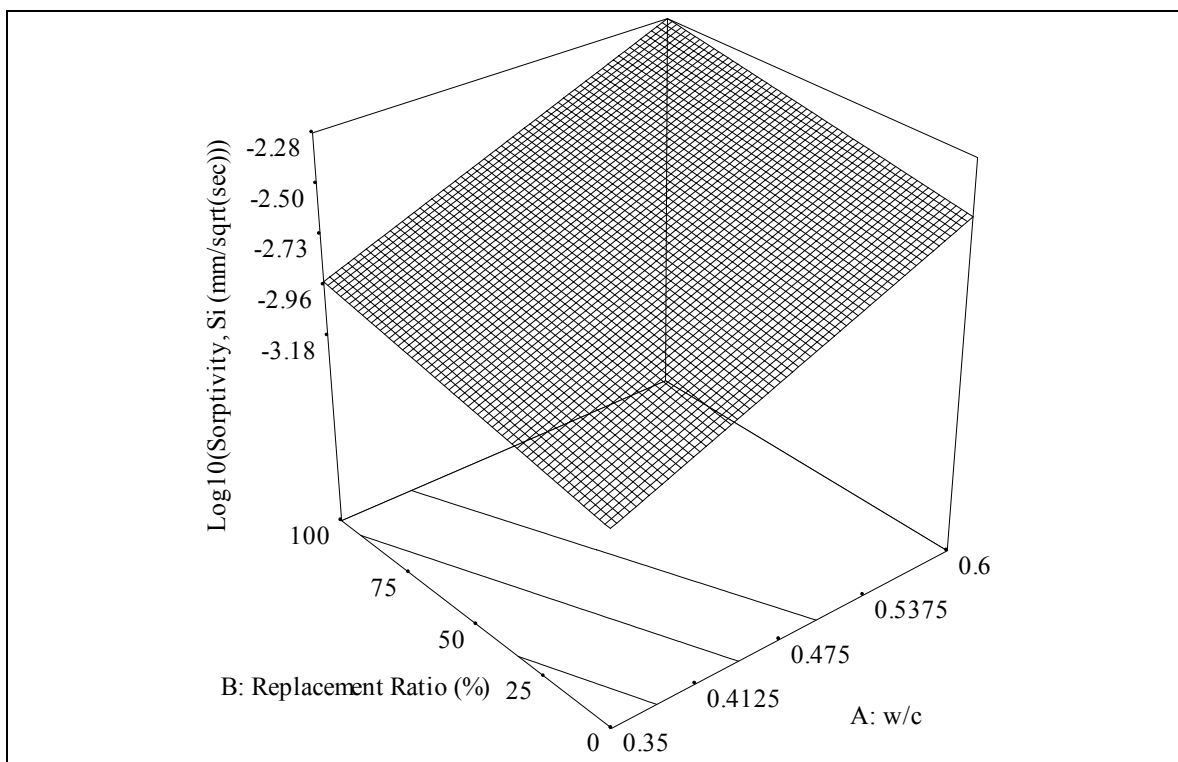


Figure 5.74. 3D surface view of sorptivity of natural and recycled aggregate concretes (age: 28 days, temperature: 105 °C)

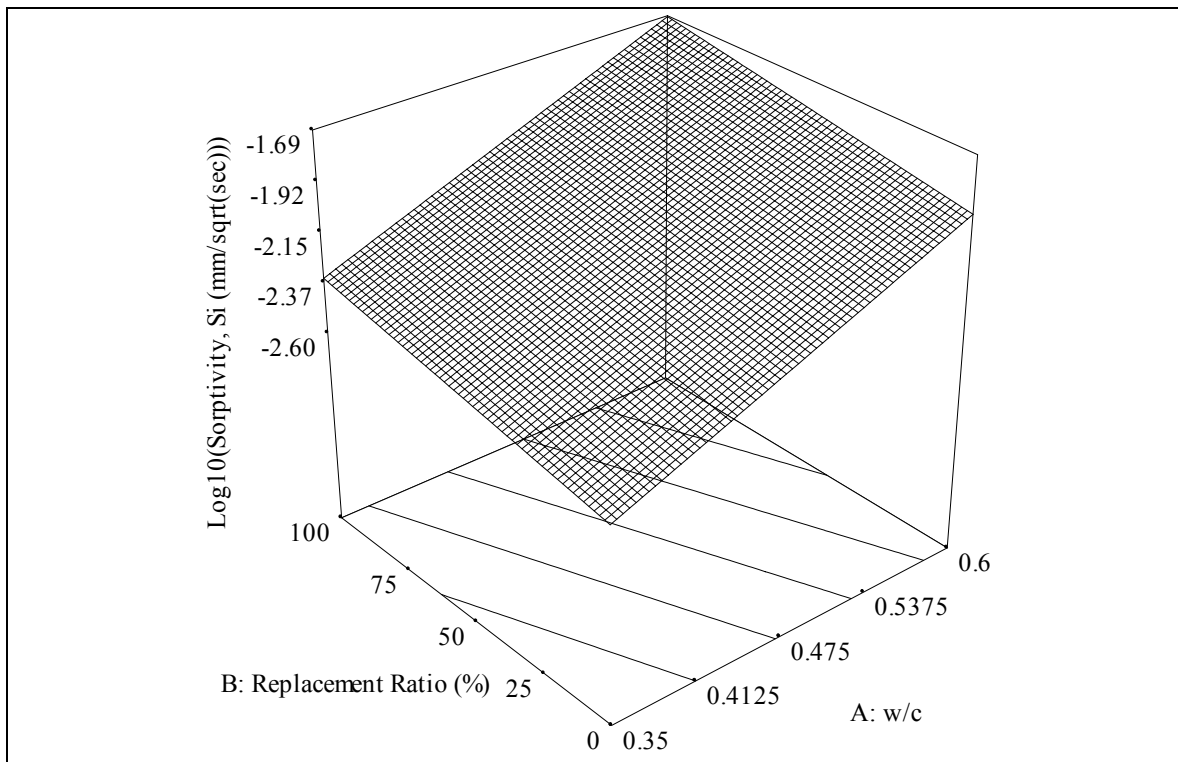


Figure 5.75. 3D surface view of sorptivity of natural and recycled aggregate concretes
(age: 28 days, temperature: 175 °C)

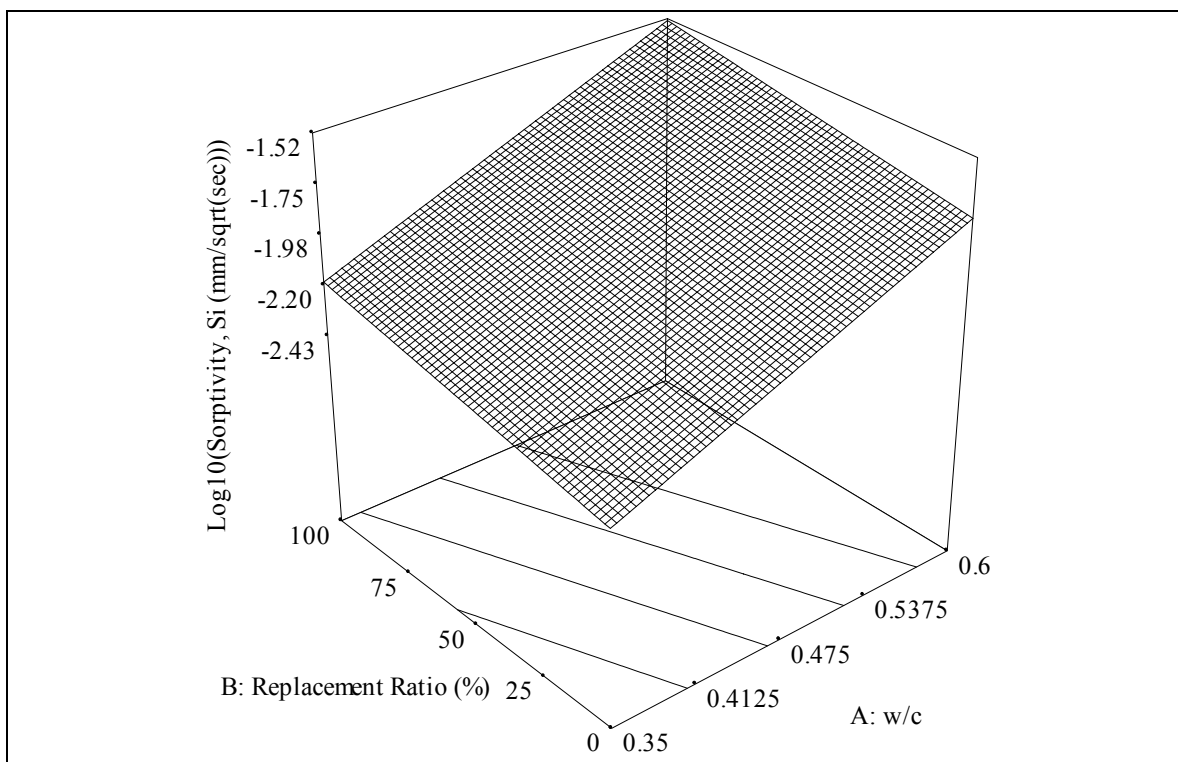


Figure 5.76. 3D surface view of sorptivity of natural and recycled aggregate concretes
(age: 28 days, temperature: 250 °C)

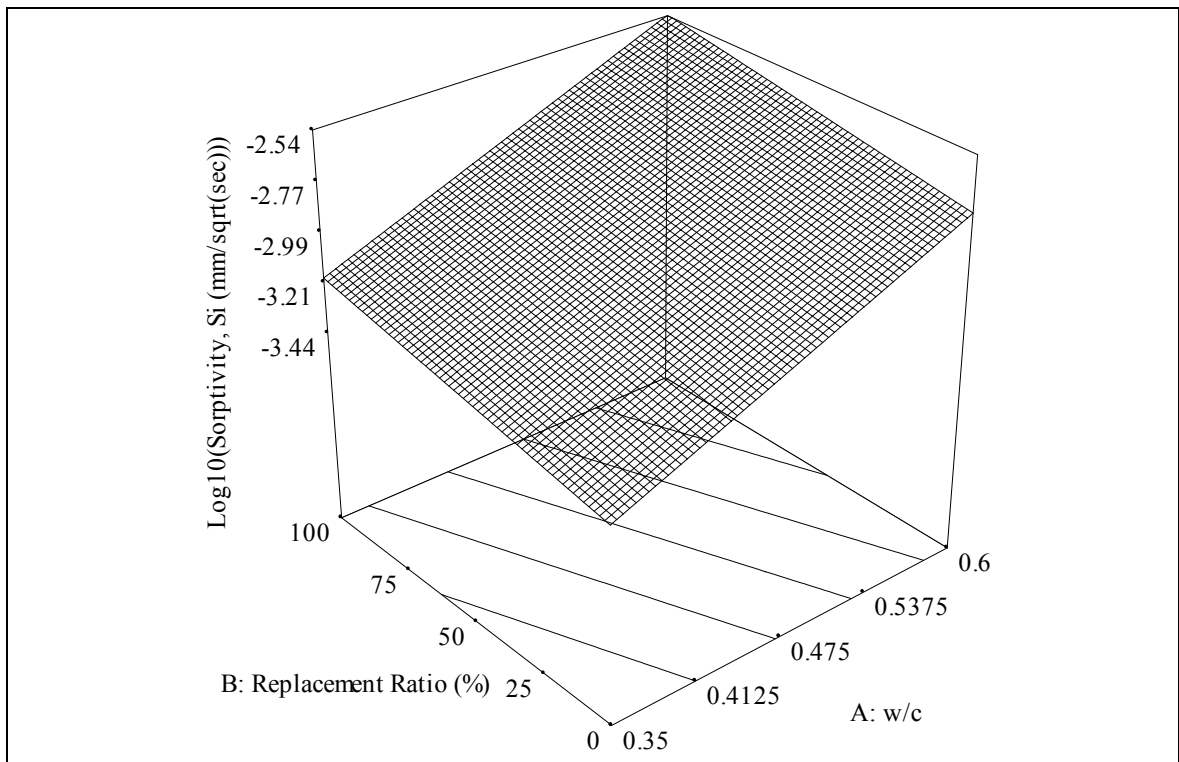


Figure 5.77. 3D surface view of sorptivity of natural and recycled aggregate concretes (age: 90 days, temperature: 22 °C)

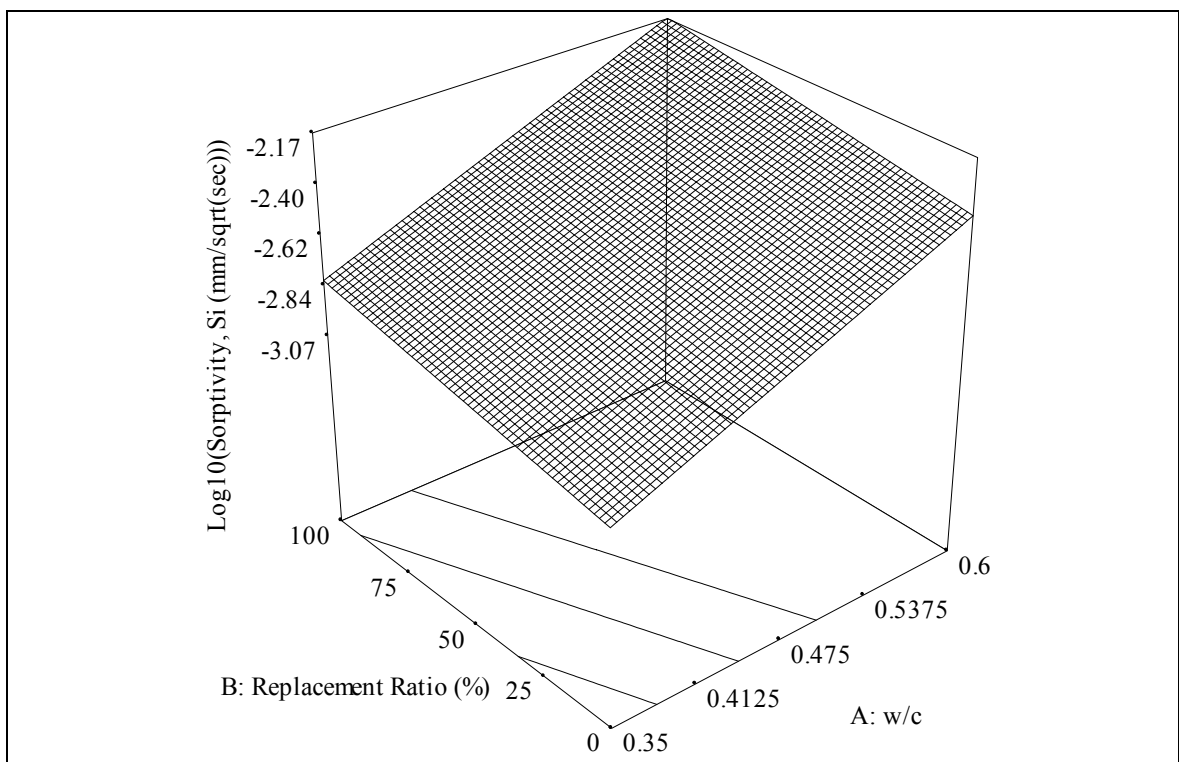


Figure 5.78. 3D surface view of sorptivity of natural and recycled aggregate concretes (age: 90 days, temperature: 105 °C)

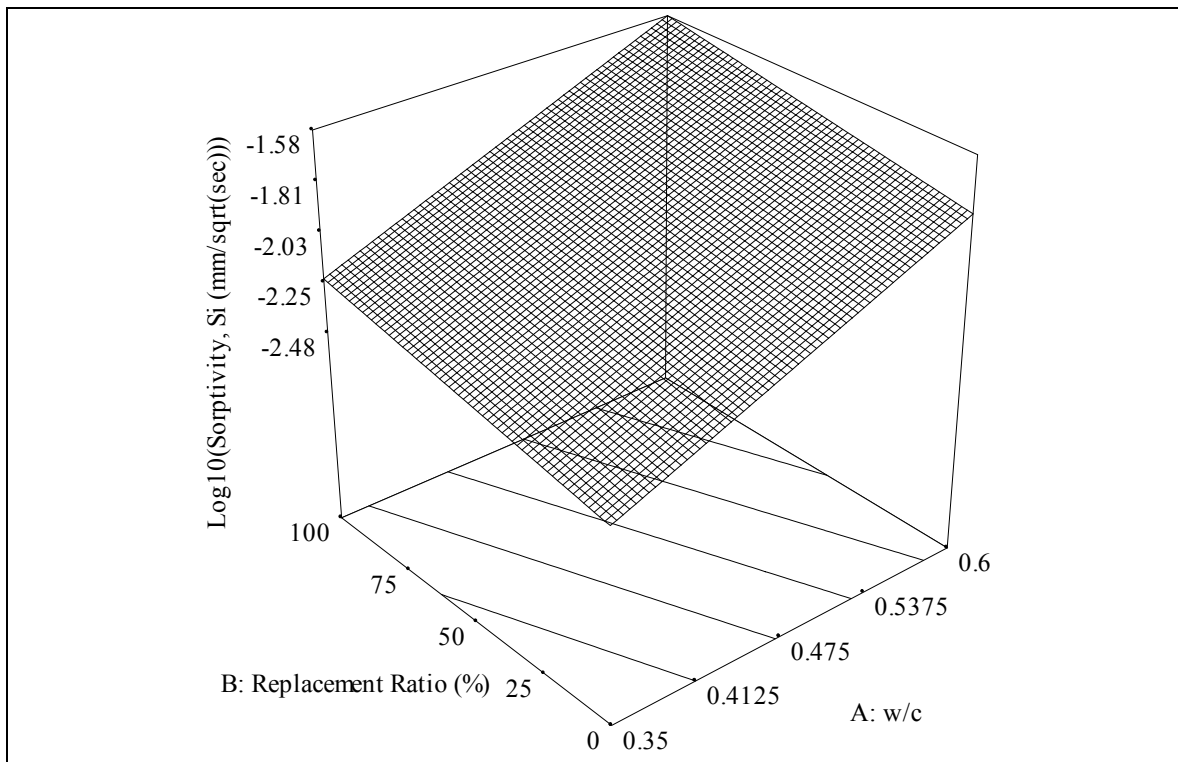


Figure 5.79. 3D surface view of sorptivity of natural and recycled aggregate concretes
(age: 90 days, temperature: 175 °C)

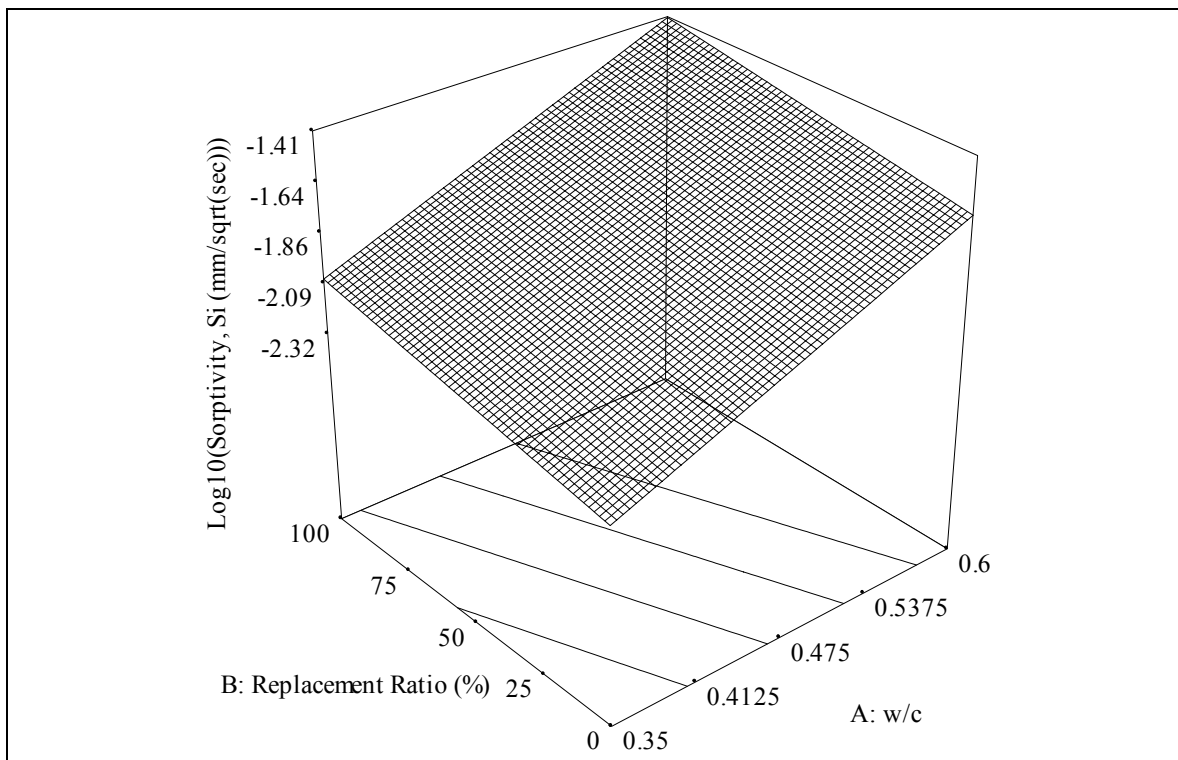


Figure 5.80. 3D surface view of sorptivity of natural and recycled aggregate concretes
(age: 90 days, temperature: 250 °C)

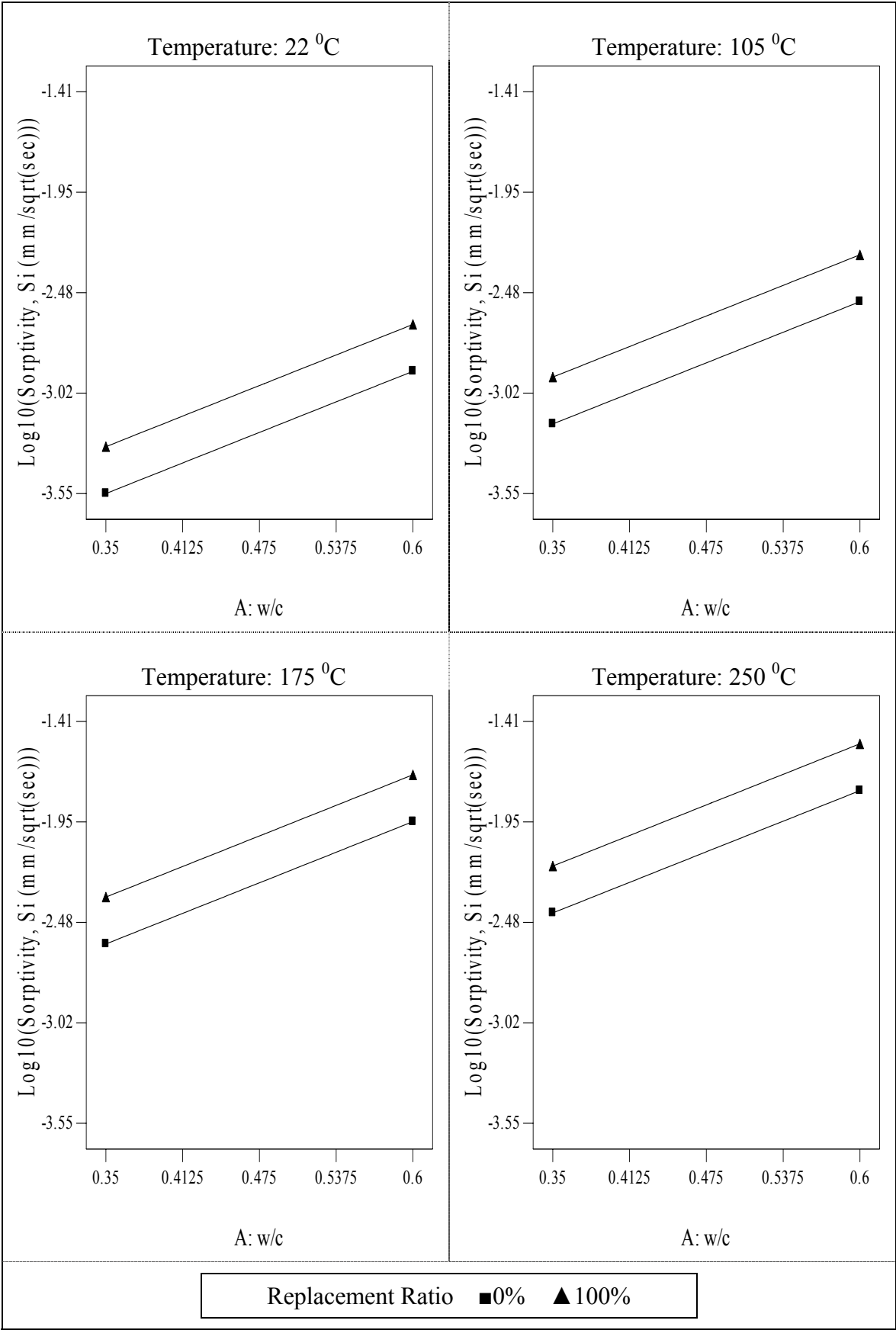


Figure 5.81. Interaction graph of w/c and replacement ratio at 28 days

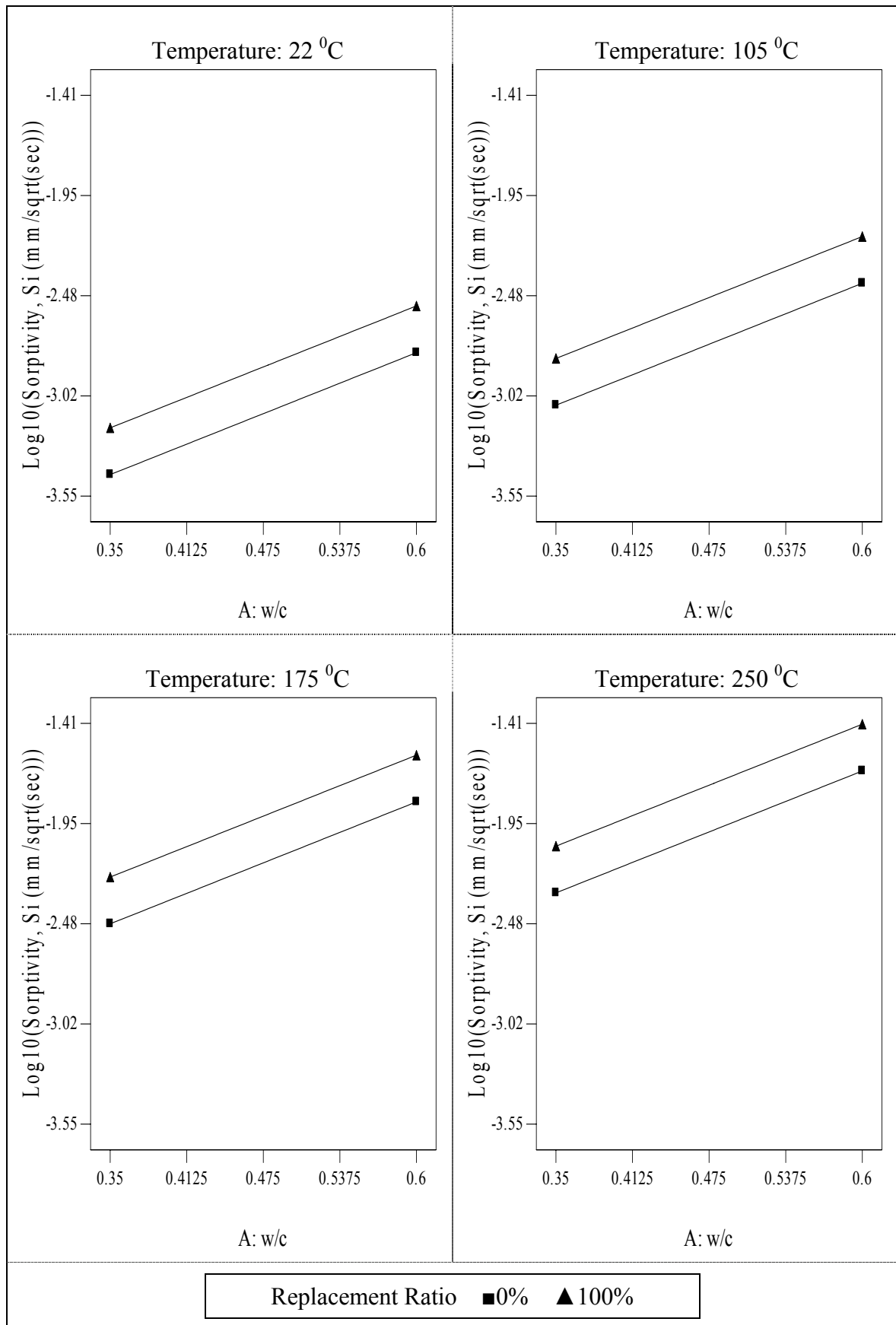


Figure 5.82. Interaction graph of w/c and replacement ratio at 90 days

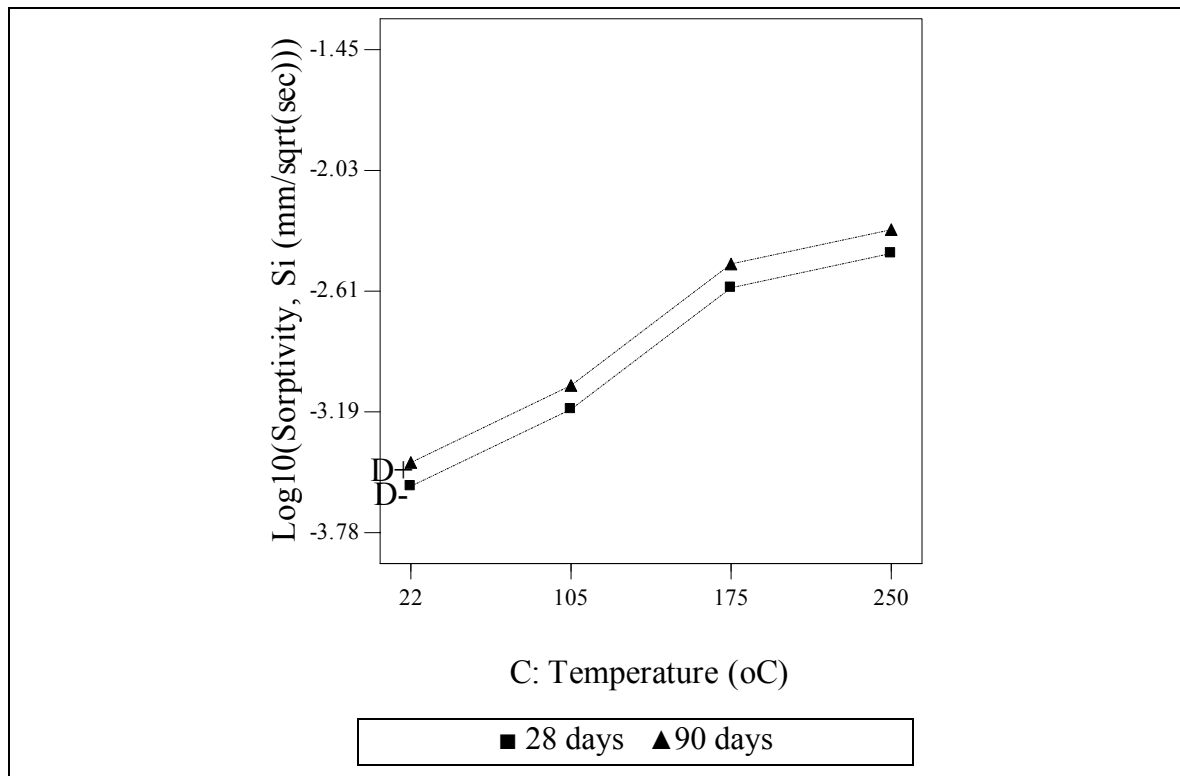


Figure 5.83. Interaction graph of temperature and age, w/c ratio =0.35
(replacement ratio =0%)

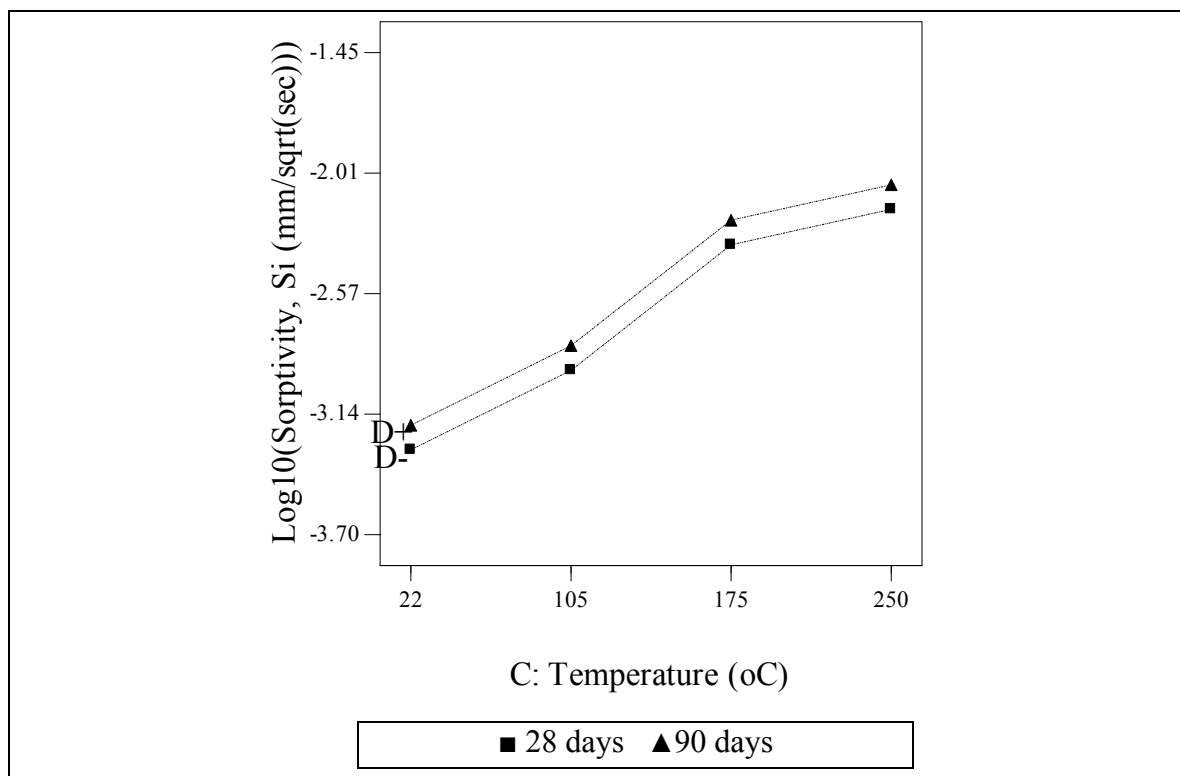


Figure 5.84. Interaction graph of temperature and age, w/c ratio =0.35
(replacement ratio =100%)

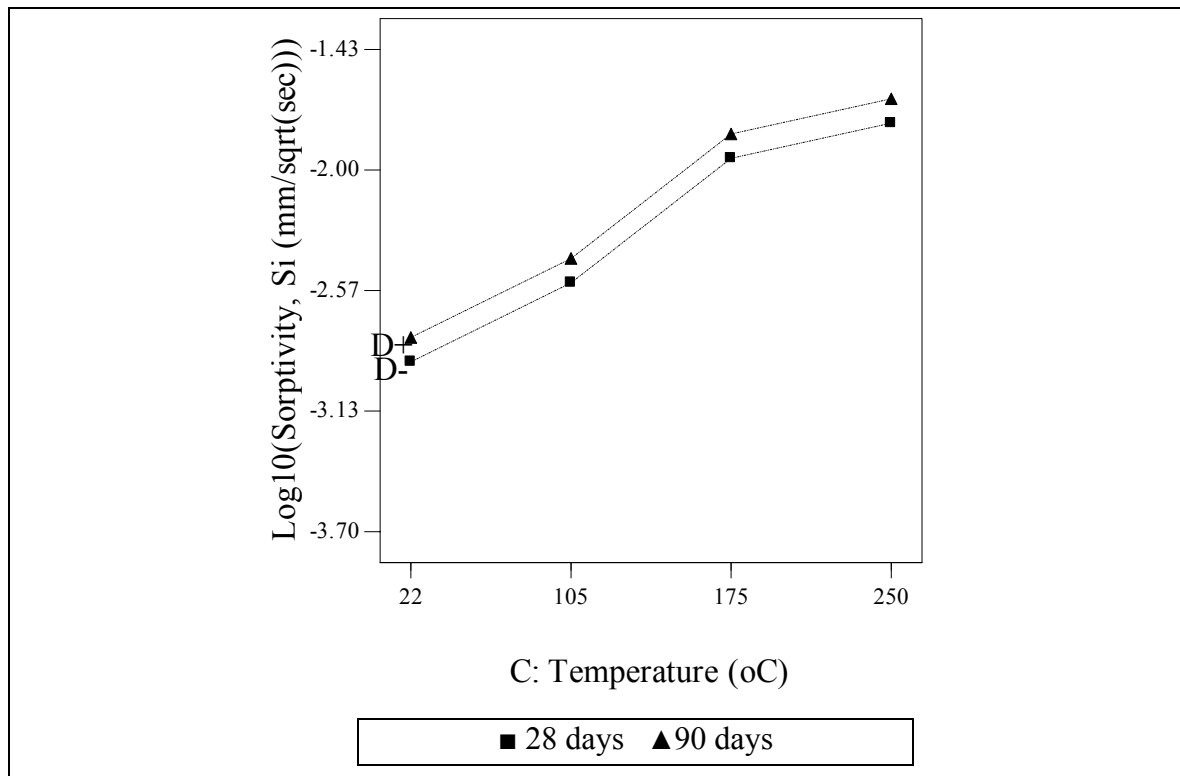


Figure 5.85. Interaction graph of temperature and age, w/c ratio = 0.60
(replacement ratio = 0%)

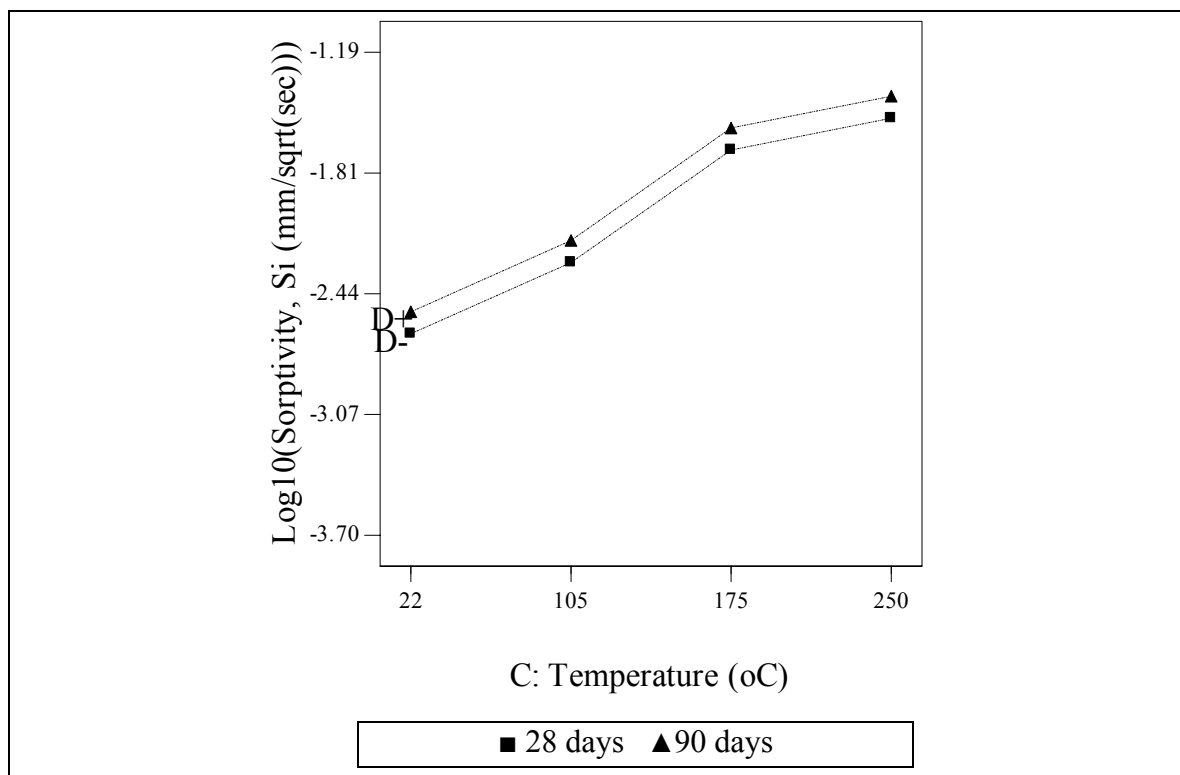


Figure 5.86. Interaction graph of temperature and age, w/c ratio = 0.60
(replacement ratio = 100%)

5.5. Adhered Mortar Content of Recycled Aggregates

The adhered mortar content of the recycled aggregates could not be determined by the hydrochloric acid dissolution method. The method represented in the literature [25] uses the hydrochloric acid dissolution to separate out the mortar and old natural aggregate from each other. In our study hydrochloric acid reacted with the aggregate, and then the aggregate size decreased by time, but HCl did not break up the bond between the adhered mortar and virgin aggregate completely. So, no result was obtained by this method.

For this reason the adhered mortar content of the recycled aggregate was determined by the heating method (thermal shock). During heating, internal forces were produced due to thermal mismatch, which finally broke the adhered mortar. After completing the thermal shock the original coarse aggregates were easily separated out from the mortar by sieving, so the mass of the aggregate and mortar obtained. Later the adhered mortar content was calculated by dividing the mass of the adhered mortar to the total mass of adhered mortar and aggregate. By this method, the adhered mortar content was found as 51.1 and 48.4 percent for the recycled aggregates (4-8mm) and (8-16mm), respectively.

On the other hand, by the equation derived in chapter four, Equation 4.16, the adhered mortar content was calculated as 47.6 and 40.9 percent for the recycled aggregates (4-8mm) and (8-16mm), respectively.

The results of the two methods above for determining the adhered mortar content were different from each other. The difference may be due to the sampling of aggregates. Another reason for differences might be the errors involved in the experiments for testing the little size and small mass aggregates.

5.6. Optimization

Mechanical and durability properties of recycled aggregate concretes exposed to elevated temperatures were tested, analyzed and reported in previous headings and

compared with the concretes that were produced with natural aggregates. Properties of concretes were examined after performing the evaluation tests such as compressive strength, split tensile strength, static modulus of elasticity, water absorption by immersion and sorptivity, and rapid chloride permeability tests. Test results showed that incorporating recycled aggregates into concrete affected the mechanical and durability properties, but the rate of effect was not same for the evaluated parts. However, none of the test results showed that the recycled aggregate concrete was unsuitable. Since the reusing of recycled aggregates in new concretes is valuable from the view point of reservation of natural sources and protection of environment, it is important to use these recycled aggregates by taking into account the final product quality. To do that, optimization of the percent replacement of recycled aggregates should be done to satisfy the required property of final product.

The optimization module in the Design Expert software searches for a combination of factor levels that simultaneously satisfy the requirements placed on each of the responses and factors. The objective of optimization is to find the “best settings” that maximize a particular response or responses. Optimization usually involves considering several responses simultaneously. A numerical optimization technique using desirability functions (d_j), which are defined for each response, can be used to optimize the responses simultaneously [127]. A desirability function (d_j) varies over the range of $0 \leq d_j \leq 1$. By using the single composite response (D) given in Equation 5.9, which is the geometric mean of the individual desirability functions, the multi objective optimization problem is solved. D is maximized over the feasible region of the design variables.

$$D = (d_1 d_2 d_3 \dots d_n)^{1/n} \quad (5.9)$$

where n is the number of responses included in the optimization. If any of the responses or factors falls outside their desirability range, the overall function becomes zero. In cases of the maximizing and minimizing of individual responses, the desirability will be defined by the formulas given in Equation 5.10 and 5.11, respectively.

$$d_j = \begin{cases} 0 & Y_j \leq \min f_j \text{ and } 0 < d_j < 1, \\ \left[\frac{Y_j - \min f_j}{\max f_j - \min f_j} \right]^{wt_j} & \min f_j < Y_j < \max f_j \\ 1 & Y_j \geq \max f_j \end{cases} \quad (5.10)$$

$$d_j = \begin{cases} 1 & Y_j \leq \min f_j \text{ and } 0 < d_j < 1, \\ \left[\frac{\max f_j - Y_j}{\max f_j - \min f_j} \right]^{wt_j} & \min f_j < Y_j < \max f_j \\ 0 & Y_j \geq \max f_j \end{cases} \quad (5.11)$$

where d_j , Y_j , $\min f_j$ and $\max f_j$ are the desirability function, the fitted value, and minimum and maximum actual (experimental) values of j th response included in the optimization, respectively. The power value wt_j is a weighting factor of the j th response.

Using the optimization module in the software, by maximizing the recycled aggregate replacement ratio it is possible to maximize compressive strength, static modulus of elasticity and splitting tensile strength, simultaneously. Thus, these three responses were considered to be of equal importance, i.e. $wt_j = 1$ and maximized simultaneously. For $n = 4$, Equation 5.9 takes the form:

$$D = (d_1 x d_2 x d_3 x d_4)^{1/4} \quad (5.12)$$

where d_1 , d_2 , d_3 and d_4 are the desirability functions of recycled aggregate replacement ratio, compressive strength, static modulus of elasticity and splitting tensile strength, respectively. The solution of this multi objective optimization at 28 days and at 22 °C, 105 °C, 175 °C, and 250 °C are shown in Figures 5.87-5.90. The optimal values of design variables and predicted compressive strength, static modulus of elasticity and splitting tensile strength values at 28 days are given in Table 5.29.

The durability properties of the concretes are also important; therefore the permeability properties of the concretes must be minimized. Numerical optimization can optimize any combination of either factors or responses. Thus, rapid chloride permeability (RCPT) charge, water absorption and sorptivity values were also considered to be of equal importance, i.e. $wt_j = 1$ and minimized simultaneously by maximizing the recycled aggregate replacement ratio. The solution of this multi objective optimization at 28 days

and at 22 °C, 105 °C, 175 °C, and 250 °C are shown in Figures 5.91-5.94. The optimal values of design variables and predicted RCPT charge, water absorption and sorptivity values at 28 days are given in Table 5.30.

Table 5.29. The optimal values of design variables and predicted response values at 28 days

Exposure temperature	Optimal values of design variables		Compressive strength (MPa)	Static modulus of elasticity (GPa)	Splitting tensile strength (MPa)	Desirability
	w/c	RR (%)				
22 °C	0.35	100	38.2	22.1	3.02	0.571
105 °C	0.35	100	36.3	21.2	3.33	0.577
175 °C	0.35	47	42.2	21.0	3.37	0.575
250 °C	0.35	37	48.9	21.9	4.29	0.713

Table 5.30. The optimal values of design variables and predicted response values at 28 days

Exposure temperature	Optimal values of design variables		RCPT charge (coulombs)	Water absorption (%)	Sorptivity (mm/s ^{1/2})	Desirability
	w/c	RR (%)				
22 °C	0.35	55	6819	5.2	0.0004	0.744
105 °C	0.35	65	3912	5.8	0.0010	0.732
175 °C	0.35	38	7320	5.9	0.0031	0.561
250 °C	0.35	31	9630	7.0	0.0044	0.462

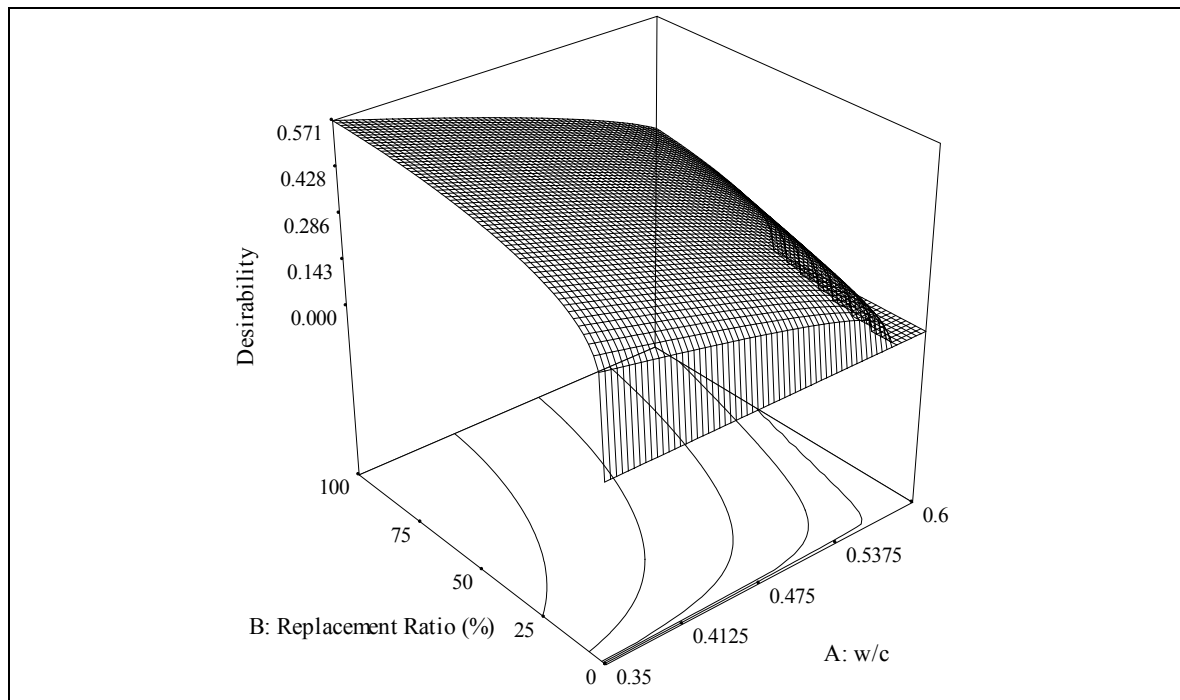


Figure 5.87. Response surface plot of the composite desirability (D) at 22 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously

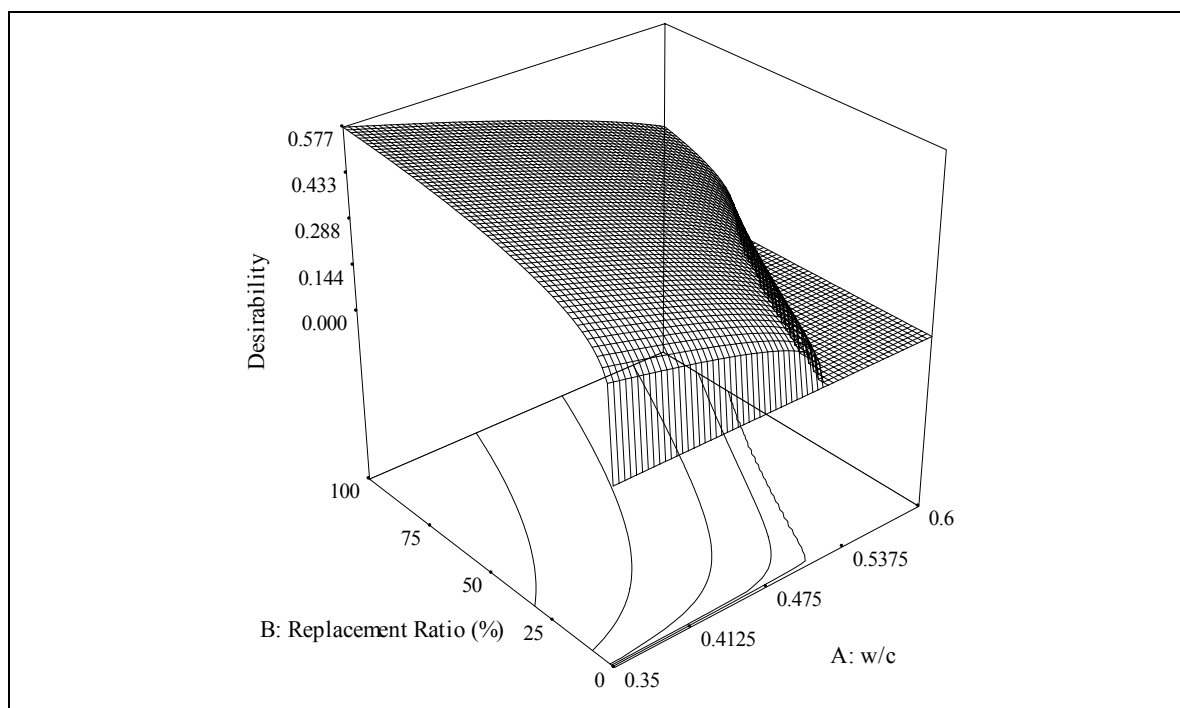


Figure 5.88. Response surface plot of the composite desirability (D) at 105 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously

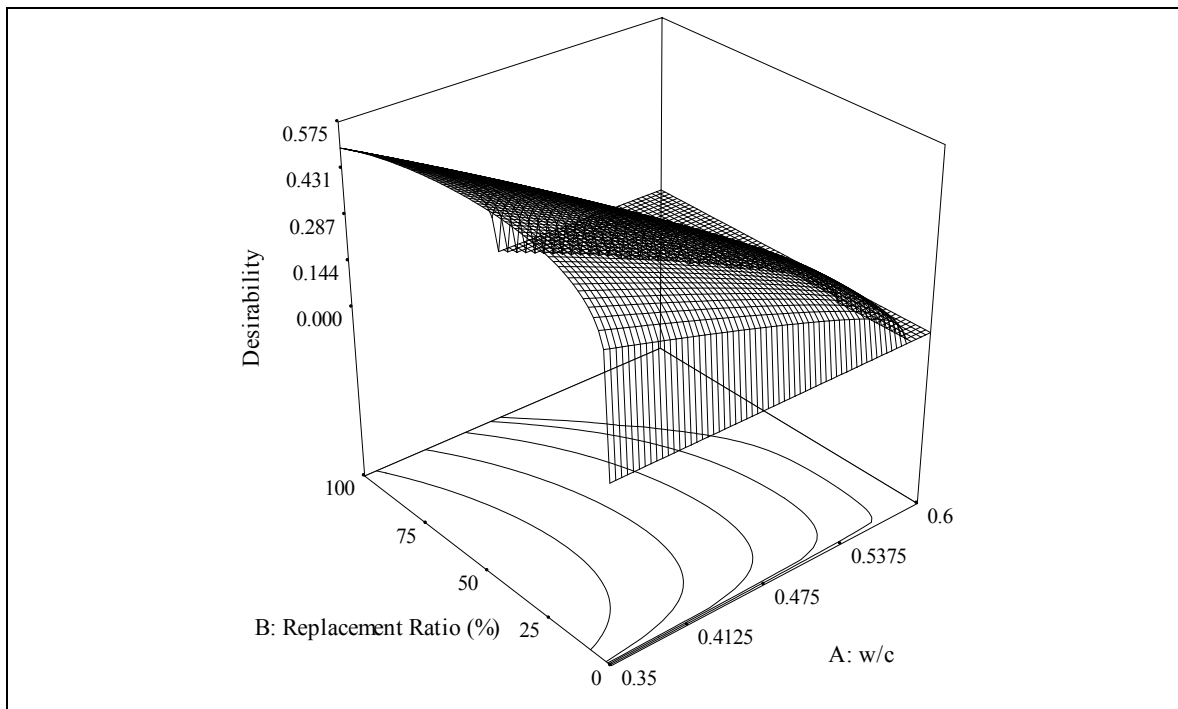


Figure 5.89. Response surface plot of the composite desirability (D) at 175 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously

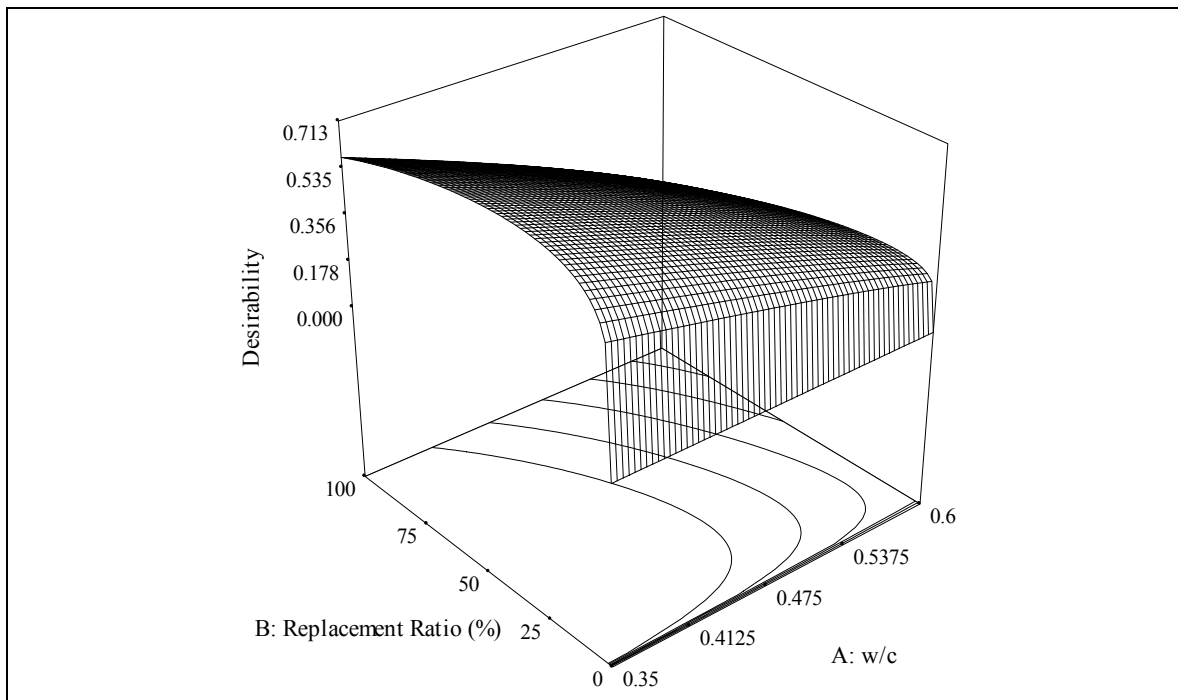


Figure 5.90. Response surface plot of the composite desirability (D) at 250 °C when compressive strength, static modulus of elasticity and splitting tensile strength are maximized simultaneously

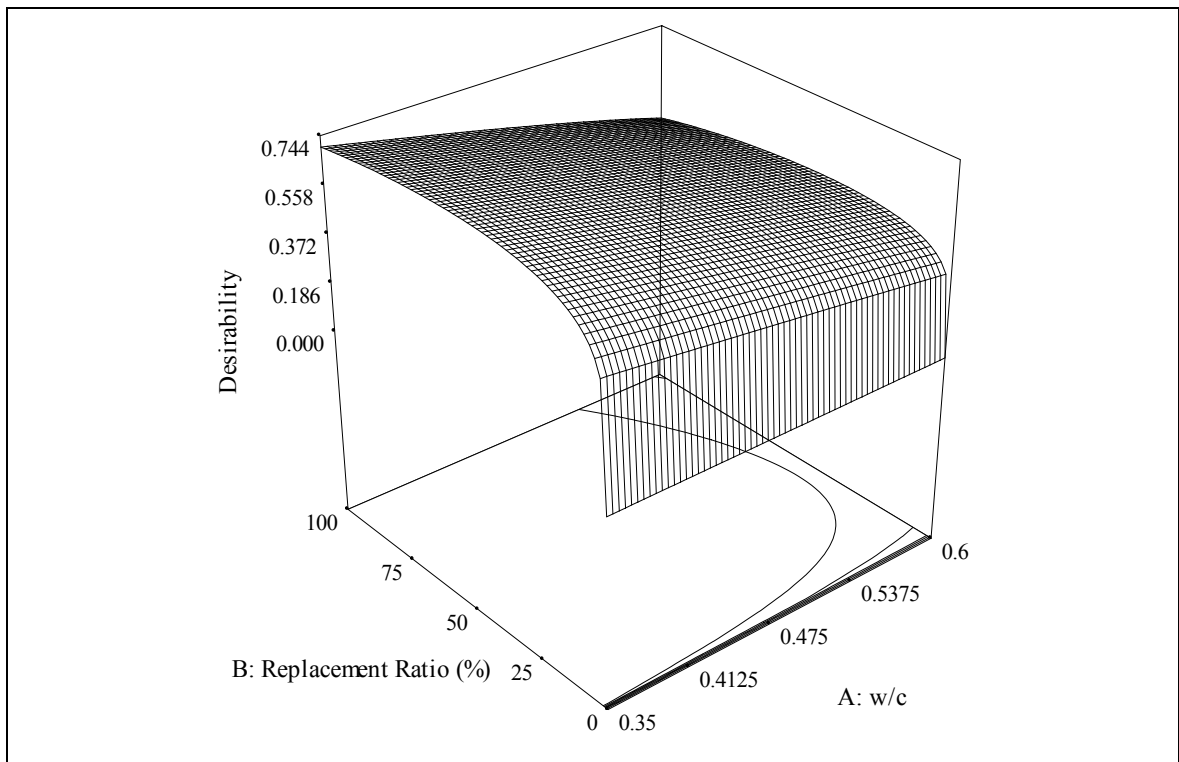


Figure 5.91. Response surface plot of the composite desirability (D) at 22 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously

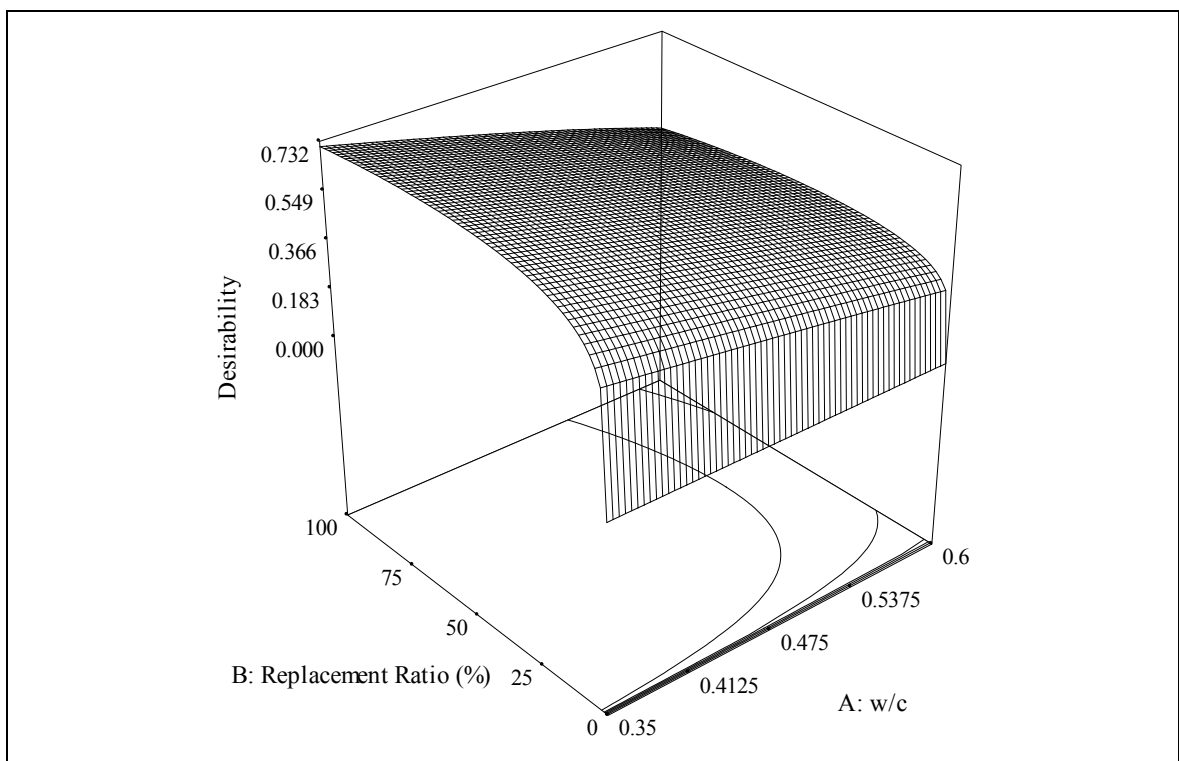


Figure 5.92. Response surface plot of the composite desirability (D) at 105 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously

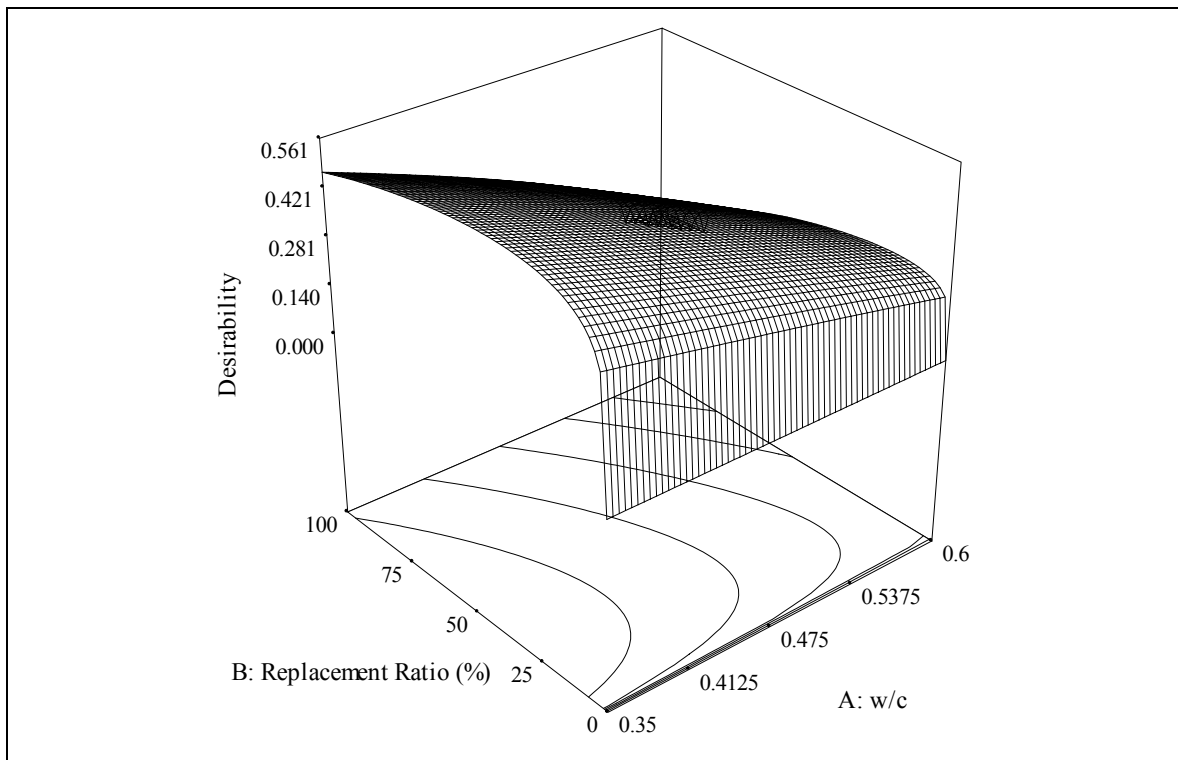


Figure 5.93. Response surface plot of the composite desirability (D) at 175 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously

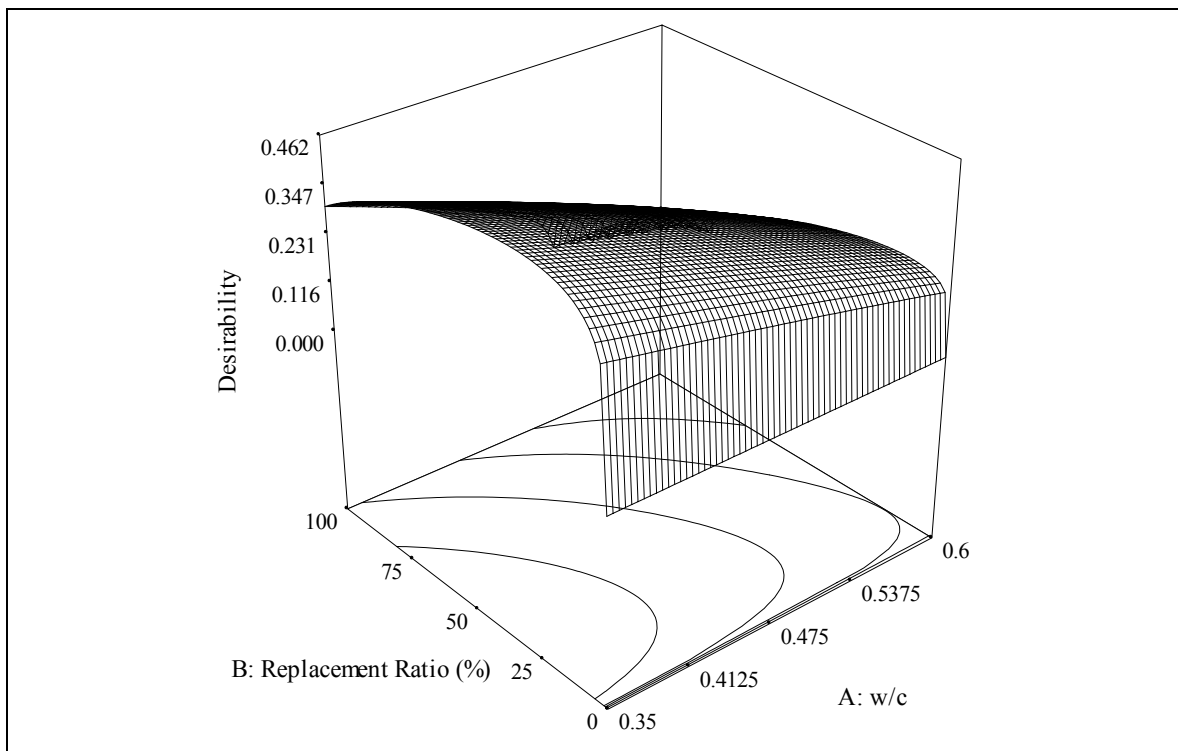


Figure 5.94. Response surface plot of the composite desirability (D) at 250 °C when RCPT charge, water absorption and sorptivity values are minimized simultaneously

Based on the criteria and the conditions that the concrete was exposed to it is seen that properties of concretes changed. Before using the recycled aggregates in concrete, the environment of concrete should be carefully examined to predict the deteriorative influences, because not only higher strength concrete is good but also concrete has to provide the desired service life without any problem like maintenance and repair cost due to degradation or deterioration.

5.7. Relationships between Concrete Properties

The interrelationships of the mechanical and durability properties of the recycled aggregate concrete are important subject for the design, construction and analysis of the structures. In the case of normal concretes, the relationships between their properties have been well established and described in detail within various design codes, standards and handbooks. However, some previous studies have indicated that such existing relationships for normal concrete may not be valid for recycled aggregate concrete [35]. The aim of this section was to perform the establishment of the relationships between the various properties of concrete. Regression analyses were performed to correlate one property to another.

5.7.1. Relationship between Compressive Strength and Splitting Tensile Strength

Among the linear [128], logarithmic, polynomial, power [35], and exponential regression models, power regression model was used in our study for statistical regression analyses to correlate the compressive strength and the splitting tensile strength. Regression analyses were performed on the actual test results obtained in this study using the power regression model which is in the form of following equation:

$$f_{st} = a.f_c^b \quad (5.13)$$

where:

f_{st} = split tensile strength

f_c = compressive strength

a, b= regression coefficients

In order to find the regression coefficients “a” and “b”, firstly the actual values were plotted using Microsoft Excel software. After the actual values plotted, the trend lines (best fitting) were added to the figures. The available trend lines in the software were in the form of above mentioned regression models. Like trend line itself, equation of the trend line was also displayed in plotted figure, Figure 5.95, which gave the regression coefficients. Hence, the following equation was proposed for the relation between the compressive strength and the splitting tensile strength of the natural and recycled aggregate concretes (mixes 1-31):

$$f_{st} = 0.19 f_c^{0.8} \quad (5.14)$$

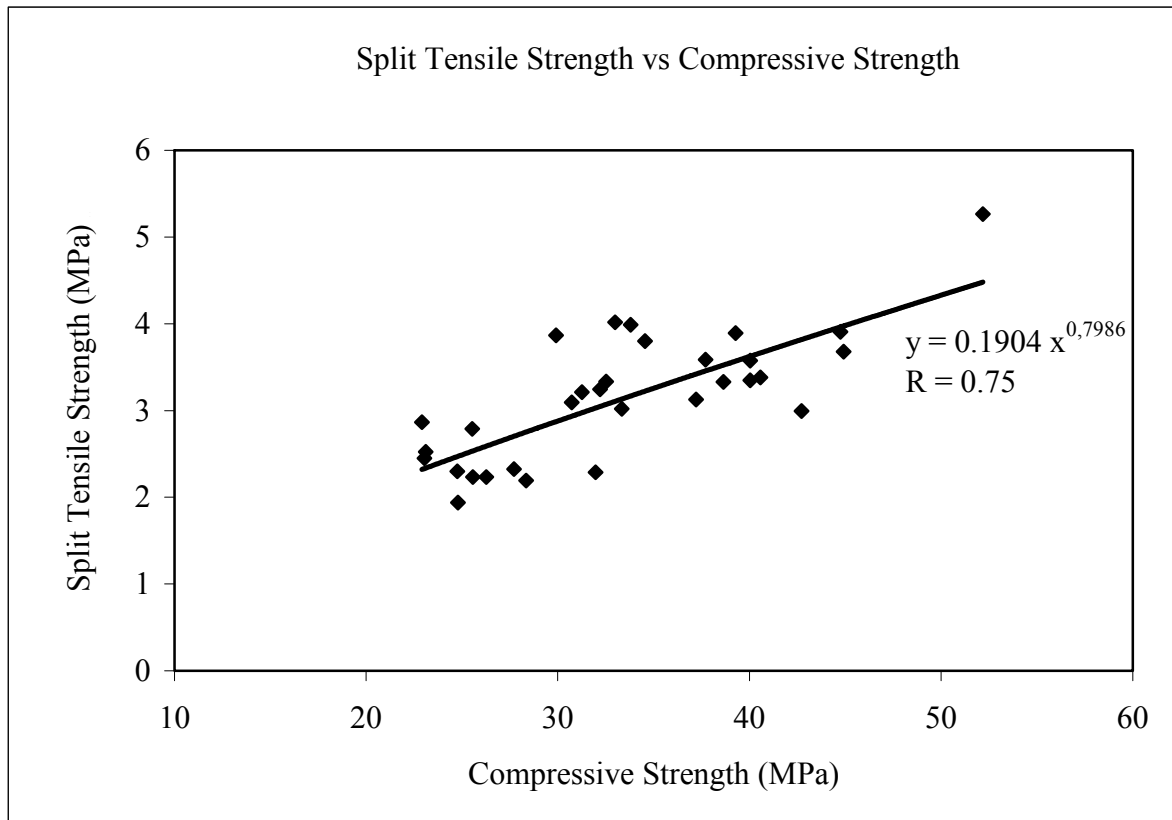


Figure 5.95. Relationship between split tensile strength and compressive strength

The relation between the splitting tensile strength and compressive strength was established from the actual values of the test results. Although R value of 0.75 indicated a considerable scatter, the positive sign of the equation showed that the tendency, which the splitting tensile strength increased with increasing compressive strength, is validated. Small R value also showed that the regression was established under poor fit, which means that there existed a remarkably high variation in the compressive strength and split tensile strength results. The reason for the variations was quite simple: although the compressive strength and split tensile strength more affected by w/c ratio, replacement ratio, exposure temperature, and age, the rate of effect was not same for the compressive strength and tensile strength. Temperature effect could be omitted if the regression analysis was performed over split tensile strength and compressive strength test results at individual temperatures. The regression analysis results at each temperature degree yielded the following equations and R values:

$$f_{st} = 0.41f_c^{0.59}, \quad R = 0.76 \quad (T=22^\circ\text{C}) \quad (5.15)$$

$$f_{st} = 0.17f_c^{0.85}, \quad R = 0.81 \quad (T=105^\circ\text{C}) \quad (5.16)$$

$$f_{st} = 0.06f_c^{1.13}, \quad R = 0.78 \quad (T=175^\circ\text{C}) \quad (5.17)$$

$$f_{st} = 0.07f_c^{1.07}, \quad R = 0.89 \quad (T=250^\circ\text{C}) \quad (5.18)$$

Lower values of R illustrated in above equations showed again that the effect of w/c ratio, replacement ratio, and age on compressive strength and split tensile strength was not same. By performing the regression analysis on particular replacement ratio data, the replacement ratio factor could be excluded. The regression analysis results at each replacement ratio gave the following equations and R values:

$$f_{st} = 0.23f_c^{0.78}, \quad R = 0.86 \quad (\text{RR}=0\%) \quad (5.19)$$

$$f_{st} = 0.15f_c^{0.85}, \quad R = 0.89 \quad (\text{RR}=25\%) \quad (5.20)$$

$$f_{st} = 0.32f_c^{0.67}, \quad R = 0.95 \quad (\text{RR}=50\%) \quad (5.21)$$

$$f_{st} = 0.12f_c^{0.89}, \quad R = 0.88 \quad (\text{RR}=100\%) \quad (5.22)$$

Finally, by performing the individual regression analysis on specific age data, 28 or 90 days, the age factor could be excluded. The regression analysis results at 28 and 90 days test results gave the following equations and R values:

$$f_{st} = 0.16f_c^{0.85}, \quad R = 0.77 \quad (28 \text{ days}) \quad (5.23)$$

$$f_{st} = 0.23f_c^{0.74}, \quad R = 0.75 \quad (90 \text{ days}) \quad (5.24)$$

The above equations showed that the relationship between split tensile strength and compressive strength differed when the circumstances changed. That is why other researchers [35, 105] gave different equations for the split tensile strength and compressive strength. The reason for the different equations was plain: the referenced authors have proposed their equations, just to best fit their own experimental results.

Instead of giving such above equations, which were different from each other and hard to keep in mind, another basic mathematical calculation was done to report the relationship between the split tensile strength and compressive strength. This basic mathematical calculation was the percentage computing. For our test results, mixes 1-31, it could be said plainly that splitting tensile strength was in the range of 7-13 percent of the compressive strength value. This latter relationship was good for practical applications, because tensile strength is often estimated from the compressive strength.

5.7.2. Relationship between Compressive Strength and Static Modulus of Elasticity

Compressive strength and static modulus of elasticity were determined from the same concrete specimen. Beyond, elasticity values reflected both stress and strain relations in linear portion, which is very important for analysis of reinforced concrete structures. The relationships between static modulus of elasticity and compressive strength were displayed in Figure 5.96 and given in Equation 5.25. Among the analyzed regression models, linear regression model gave the highest R value. That is why linear regression model was used to correlate the static modulus of elasticity and compressive strength. Equation 5.25 is derived from the results of mixes 1-31. The lower R value in Figure 5.96 showed that the static modulus of elasticity and compressive strength values scattered from

each other. This means that these two mechanical properties were affected by the w/c ratio, replacement ratio, exposure temperature, and age but in different amounts. As it is seen from Figure 5.96, the equations proposed for normal concretes by different national building codes such as ACI 318 and TS 500 overestimated the static modulus of elasticity value of recycled aggregate concretes from the given compressive strength value. Regression analyses results performed on the sub data of test results are given in Table 5.31.

$$E_c = 2115f_c^{0.6} \quad (5.25)$$

where E_c = static modulus of elasticity

f_c = compressive strength

Table 5.31. Relationship between static modulus of elasticity and compressive strength at specific situations

Equation	R	Sub data
$E_c = 905f_c^{0.85}$	0.89	Temperature= 22 °C
$E_c = 5011f_c^{0.4}$	0.83	Temperature = 105 °C
$E_c = 297f_c^{1.15}$	0.94	Temperature = 175 °C
$E_c = 110f_c^{1.35}$	0.99	Temperature = 250 °C
$E_c = 3595f_c^{0.47}$	0.57	Replacement Ratio= 0%
$E_c = 1140f_c^{0.75}$	0.86	Replacement Ratio= 25%
$E_c = 10683f_c^{0.15}$	0.24	Replacement Ratio= 50%
$E_c = 300f_c^{1.15}$	0.79	Replacement Ratio= 100%
$E_c = 1375f_c^{0.73}$	0.68	Age= 28 days
$E_c = 3680f_c^{0.44}$	0.40	Age= 90 days
$E_c = 2115f_c^{0.6}$	0.56	All data included

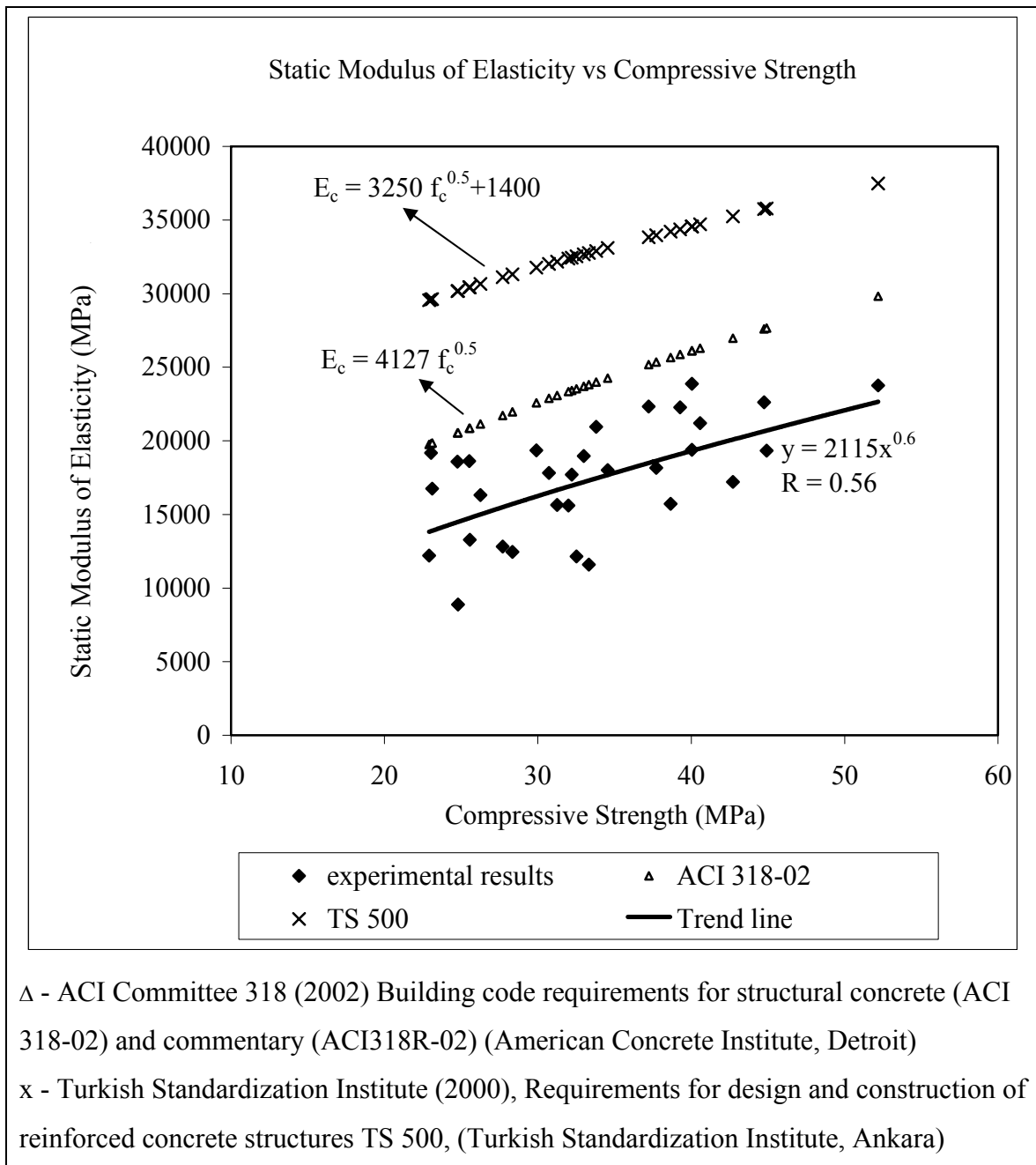


Figure 5.96. Relationship between static modulus of elasticity and compressive strength

It is seen from the Table 5.31 that when a factor was changed the equation was also changed, which means that the proposed equations were valid if only that conditions occurred. As seen from the same table, R values were higher when temperature factor was omitted, which means that the temperature factor was significant. This was the case in analysis of variance report, too. Due to this higher variety, estimation or calculation of modulus of elasticity from compressive strength will mislead the user. According to our test results, whatever the w/c ratio, replacement ratio, exposed temperature, and age, it

should be declared that static modulus of elasticity was minimum 348 folds and maximum 832 folds that of compressive strength.

5.7.3. Relationship between Compressive Strength and Saturated Bulk Density

The compressive strength of the normal concrete increases as the mass density rises [35]. With this information, the compressive strength of the concrete could be correlated to its density. The relationship between the compressive strength and saturated bulk density was investigated and plotted in Figure 5.97 and displayed in Equation 5.26.

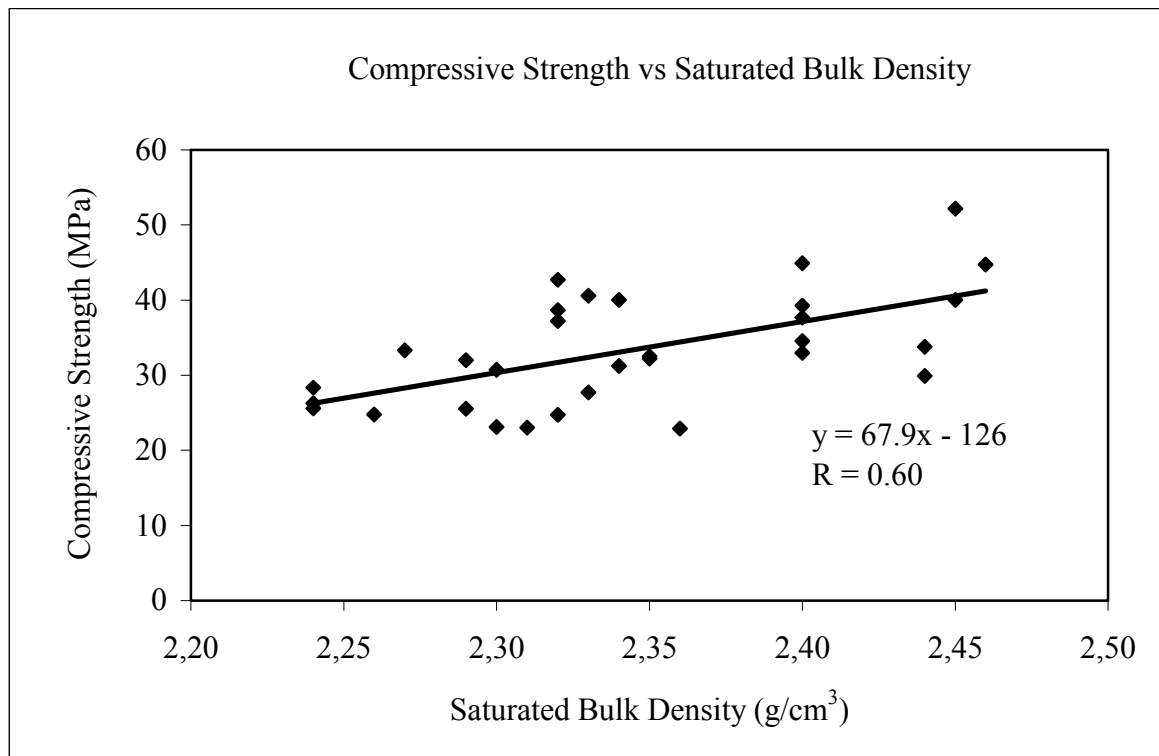


Figure 5.97. Relationship between compressive strength and saturated bulk density

$$f_c = 67.9\rho - 126 \quad (5.26)$$

where ρ = saturated bulk density (g/cm³)

The above equation and positive value of the coefficient of ρ , found by linear regression analysis of actual test results of mixes 1-31, showed that there existed a direct relation between the density and compressive strength. Besides, lower R value reflected the scattered data, as seen in Figure 5.97. Lower R value showed that factors of w/c ratio, replacement ratio, exposed temperature, and age affected the compressive strength and density but not in the same amount. Regression analyses were performed also on the sub data of compressive strength and saturated bulk density; analyses results of that data are given in Table 5.32. The equations and correlation coefficients given in Table 5.32 illustrated that there was a large difference between the compressive strength and density even at specific conditions. According to the correlation coefficients found it was really hard to rely on the relation and to use the equations to predict the compressive strength of concrete from its density. Despite the high variety in equations, for practical applications it could be easily said that the density value of the concrete, in g/cm^3 , was in the range of 5-10 percent that of compressive strength, in MPa.

Table 5.32. Relationship between compressive strength and saturated bulk density at specific situations

Equation	R	Sub data
$f_c = 61.7\rho - 112$	0.59	Temperature= 22 °C
$f_c = 51.8\rho - 92$	0.53	Temperature = 105 °C
$f_c = 62.2\rho - 112$	0.75	Temperature = 175 °C
$f_c = 114.8\rho - 230$	0.86	Temperature = 250 °C
$f_c = 129.3\rho - 276$	0.78	Replacement Ratio= 0%
$f_c = 247.4\rho - 548$	0.99	Replacement Ratio= 25%
$f_c = 113\rho - 232$	0.77	Replacement Ratio= 50%
$f_c = 155.6\rho - 323$	0.91	Replacement Ratio= 100%
$f_c = 74.4\rho - 142$	0.64	Age= 28 days
$f_c = 63\rho - 113$	0.57	Age= 90 days
$f_c = 67.9\rho - 126$	0.60	All data included

5.7.4. Relationship between Compressive Strength and Permeable Pore Volume

Compressive strength of the concrete was related to the porosity of concrete in general: the lower the occupied pores the higher the strength. The established relationship between the compressive strength and volume of permeable pore space using the actual test results of mixes 1-31 was plotted in Figure 5.98 and presented in Equation 5.27. The negative value of the coefficient of v in below equation showed that there existed a reverse relation between the compressive strength and volume of permeable pore space, as one increases the other decreases. The equation was established over the actual test results of compressive strength and volume of permeable pore space by linear regression analysis.

$$f_c = -0.85v + 47 \quad (5.27)$$

where v = volume of permeable pore space (%)

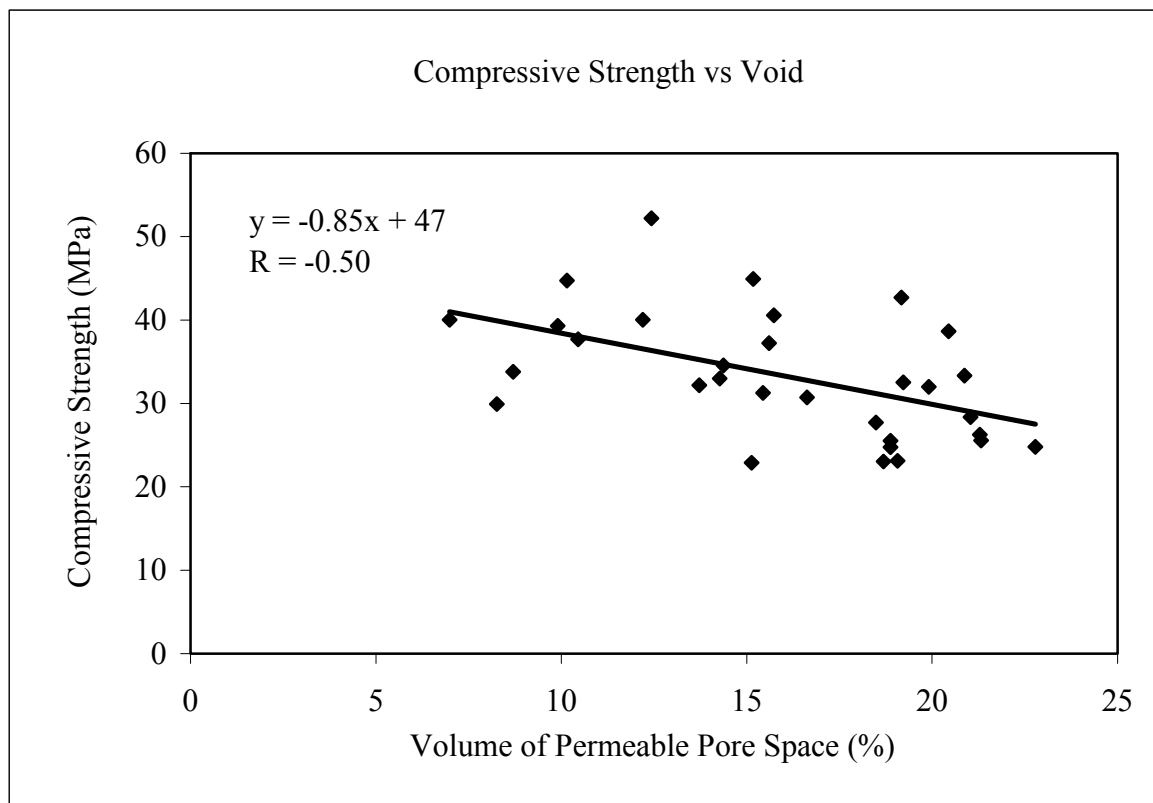


Figure 5.98. Relationship between compressive strength and volume of permeable pore space

R value of -0.50 was not high enough to use the established correlation to calculate the compressive strength from the permeable pore volume. Results showed that the factors w/c ratio, replacement ratio, exposed temperature and age have significant but different effects on the two related parameters. Regression analyses were performed also on the sub data of compressive strength and volume of permeable pore space. Regression analyses results of sub data are given in Table 5.33. The large variation between the compressive strength and volume of permeable pore space even at specific conditions are seen easily from the equations and correlation coefficients given in Table 5.33. In the reality of variation, it was hard to predict one property from another.

Table 5.33. Relationship between compressive strength and volume of permeable pore space at specific situations

Equation	R	Sub data
$f_c = -1.18v + 49$	-0.78	Temperature= 22 °C
$f_c = -0.90v + 43$	-0.69	Temperature = 105 °C
$f_c = -1.43v + 58$	-0.86	Temperature = 175 °C
$f_c = -2.31v + 81$	-0.91	Temperature = 250 °C
$f_c = -1.04v + 48$	-0.51	Replacement Ratio= 0%
$f_c = -1.68v + 61$	-0.46	Replacement Ratio= 25%
$f_c = -0.87v + 46$	-0.56	Replacement Ratio= 50%
$f_c = -1.52v + 61$	-0.75	Replacement Ratio= 100%
$f_c = -0.83v + 46$	-0.48	Age= 28 days
$f_c = -0.91v + 49$	-0.56	Age= 90 days
$f_c = -0.85v + 47$	-0.50	All data included

5.7.5. Relationship between Compressive Strength and Total Charge Passed

Total charge passed value obtained from the rapid chloride permeability test (RCPT) was related to the compressive strength of concrete; since RCPT measures the chloride permeability, which is related to the porosity too. The relationship between the total charge

passed and compressive strength for the mixes 1-31 was plotted in Figure 5.99 and displayed in Equation 5.28. Linear model was chosen for regression analysis. As obvious from the figure and negative sign of the compressive strength coefficient, total charge passed decreased linearly with increasing compressive strength. However, very small, -0.26, R value demonstrated the poor relation between the total charged passed and compressive strength of the concrete. Regression analyses were performed also on the sub data of total charged passed and compressive strength; analyses results of sub data are given in Table 5.34. The equations and lower R values showed that there are great discrepancies in the data series. Although it was clear from ANOVA report that w/c ratio, replacement ratio, exposed temperature and age affected the concrete properties, the relationships established here confirmed that the degree of effect of above factors was totally different for the total charged passed and compressive strength properties. So predicting the total charge passed value from compressive strength was very hard under these circumstances.

$$Q = -211f_c + 17724 \quad (5.28)$$

where Q= total charge passed (coulombs)

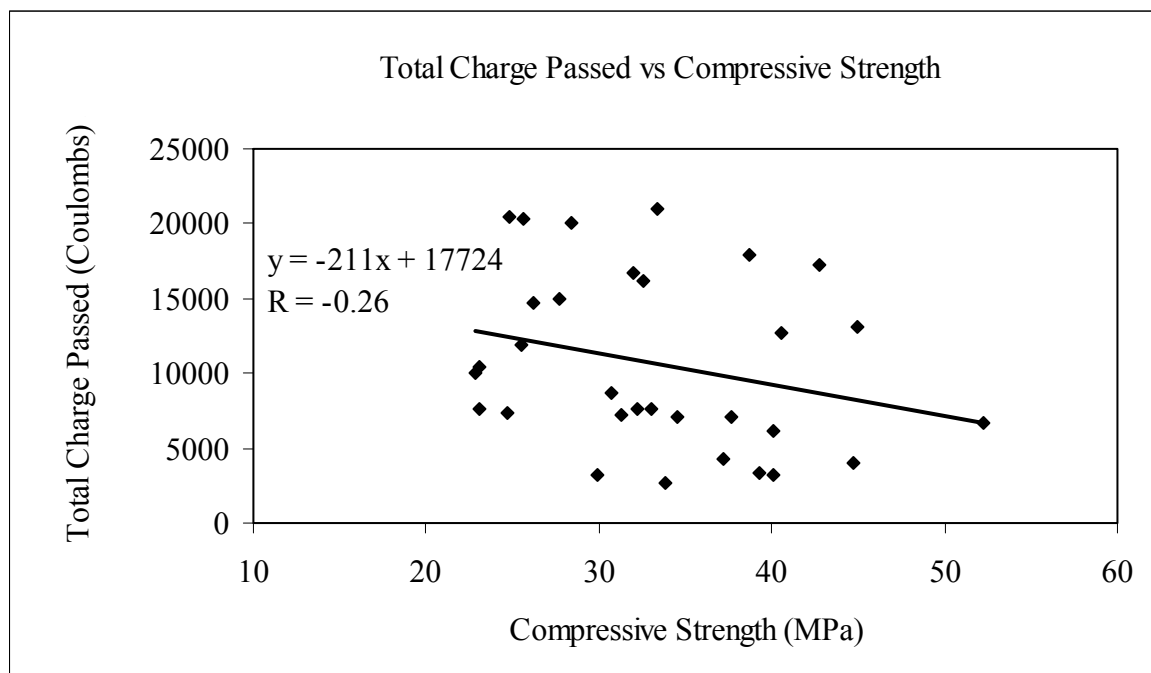


Figure 5.99. Relationship between total charge passed and compressive strength

Table 5.34. Relationship between total charged passed and compressive strength at specific situations

Equation	R	Sub data
$Q = -421f_c + 21864$	-0.79	Temperature= 22 °C
$Q = -416f_c + 18951$	-0.74	Temperature = 105 °C
$Q = -720f_c + 36960$	-0.77	Temperature = 175 °C
$Q = -463f_c + 33866$	-0.85	Temperature = 250 °C
$Q = -125f_c + 11082$	-0.30	Replacement Ratio= 0%
$Q = -6f_c + 11671$	-0.01	Replacement Ratio= 25%
$Q = -106f_c + 13097$	-0.14	Replacement Ratio= 50%
$Q = -477f_c + 29789$	-0.54	Replacement Ratio= 100%
$Q = -285f_c + 19451$	-0.41	Age= 28 days
$Q = -125f_c + 15420$	-0.14	Age= 90 days
$Q = -211f_c + 17724$	-0.26	All data included

5.7.6. Relationship between Total Charge Passed and Water Absorption

Rapid chloride permeability and water absorption tests are related with durability properties, so the relationship between them was worth to quantify. The established relationship between the total charge passed and water absorption using the actual test results of mixes 1-31 was plotted in Figure 5.100 and displayed in Equation 5.29. The positive value of the coefficient of A in Equation 5.29 illustrated that there existed a straight relation between the total charge passed and water absorption. When the total charge passed increased the water absorption increased. The equation was established over the actual test results of mixes 1-31 by linear regression analysis.

$$Q = 2170A - 5402 \quad (5.29)$$

where A = water absorption by immersion (%)

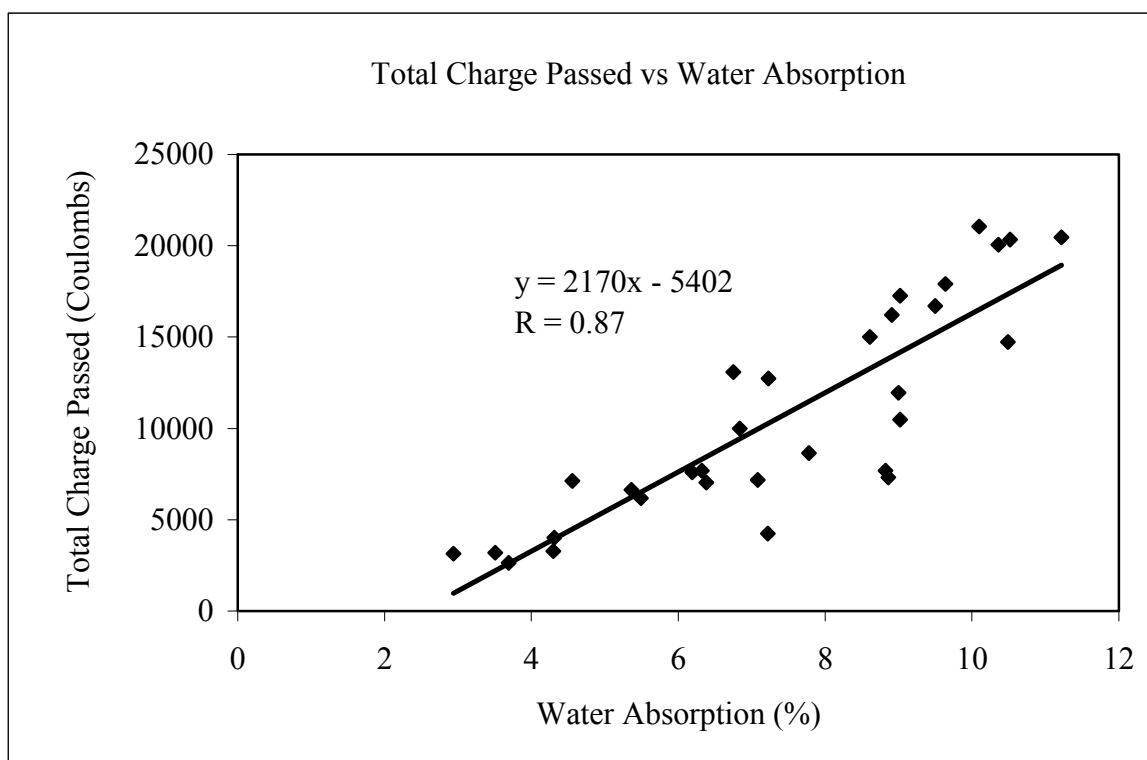


Figure 5.100. Relationship between total charge passed and water absorption

R value of 0.87 was higher, so using the proposed equation it would be simple to predict the total charge passed from water absorption. Results showed that the factors w/c ratio, replacement ratio, exposed temperature and age affected the both property in the same way. Regression analyses performed also on the sub data of total charge passed and water absorption. Regression analyses results of sub data are given in Table 5.35. The value of R given in Table 5.35 ranging between 0.71 and 0.98 showed that calculation of one property from another was not so easy, because equation was valid under that specified circumstances.

5.7.7. Relationship between Total Charge Passed and Sorptivity

Two durability properties of concrete namely chloride permeability and sorptivity values were investigated to set up a relationship between them, despite the rapid chloride permeability and sorptivity tests results give different conclusions to the researcher. The plots of these two test results obtained from mixes 1-31 were illustrated in Figure 5.101. The equation for this relationship was given in Equation 5.30.

Table 5.35. Relationship between total charge passed and water absorption at specific situations

Equation	R	Sub data
$Q = 1435A - 941$	0.94	Temperature= 22 °C
$Q = 1240A - 1965$	0.87	Temperature = 105 °C
$Q = 2749A - 8783$	0.98	Temperature = 175 °C
$Q = 2365A - 4522$	0.96	Temperature = 250 °C
$Q = 1388A - 1451$	0.79	Replacement Ratio= 0%
$Q = 2551A - 6427$	0.71	Replacement Ratio= 25%
$Q = 1837A - 3687$	0.84	Replacement Ratio= 50%
$Q = 2804A - 10778$	0.88	Replacement Ratio= 100%
$Q = 2031A - 4601$	0.91	Age= 28 days
$Q = 2339A - 6488$	0.83	Age= 90 days
$Q = 2170A - 5402$	0.87	All data included

$$Q = 562033S + 6496 \quad (5.30)$$

where S= sorptivity (mm/sqrt(sec))

The above equation was established over the actual test results of mixes 1-31 by linear regression analysis. The positive value of the coefficient of S in Equation 5.30 showed that there existed a straight relation between the total charge passed and sorptivity. From this equation it could be concluded that chloride permeability increased linearly with the increasing sorptivity. However, R value of 0.79 showed that there was not a close relationship between the chloride permeability and sorptivity. Therefore using the proposed equation it would be hard to rely on predicted value of the total charge passed from sorptivity. Regression analyses were performed also on the sub data of total charge passed and sorptivity. Regression analyses results of sub data are given in Table 5.36. The value of R given in Table 5.36 ranging between 0.42 and 0.99 exhibited that the variation of data between the measured properties was because of w/c ratio, replacement ratio,

exposed temperature, and age factors. Beyond that reason, relation of these two properties does not reflect the same durability property in engineering.

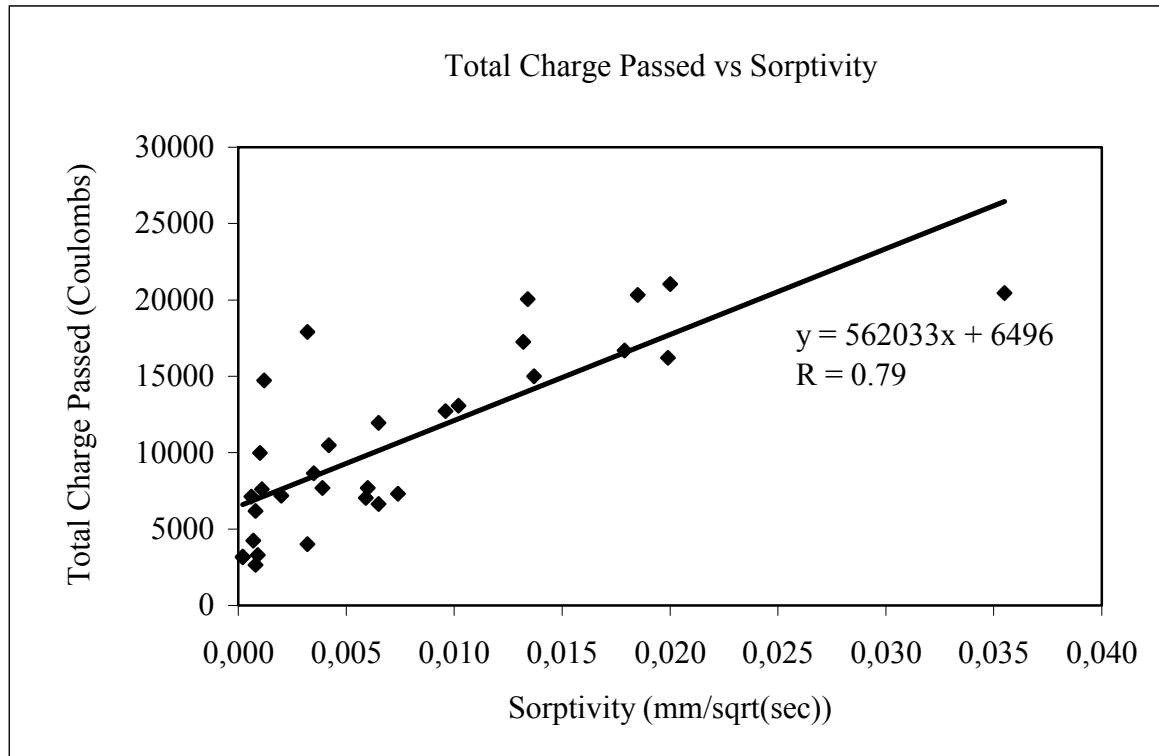


Figure 5.101. Relationship between total charge passed and sorptivity

Table 5.36. Relationship between total charge passed and sorptivity at specific situations

Equation	R	Sub data
$Q = 2692353S + 5343$	0.42	Temperature= 22 °C
$Q = 967102S + 3361$	0.80	Temperature = 105 °C
$Q = 944010S + 2786$	0.93	Temperature = 175 °C
$Q = 264036S + 11994$	0.58	Temperature = 250 °C
$Q = 588764S + 3928$	0.85	Replacement Ratio= 0%
$Q = 590243S + 6980$	0.99	Replacement Ratio= 25%
$Q = 1311775S + 5854$	0.58	Replacement Ratio= 50%
$Q = 427478S + 9247$	0.78	Replacement Ratio= 100%
$Q = 415961S + 7510$	0.69	Age= 28 days
$Q = 831631S + 4144$	0.93	Age= 90 days
$Q = 562033S + 6496$	0.79	All data included

5.7.8. Relationship between Sorptivity and Water Absorption

Sorptivity and water absorption values were evaluated to generate a relationship among them. It should be considered that although both tests measure water absorption, sorptivity measurements return capillary uptake absorption whereas water absorption measurements provide absorption by total immersion of concrete in water. The generated relationship between the sorptivity and water absorption using the actual test results of mixes 1-31 was plotted in Figure 5.102 and displayed in Equation 5.31. The equation was established over the actual test results of mixes 1-31 by regression analysis with power model. Power model was selected because of high correlation coefficient achieved with that model. As it was clear from the positive value of the coefficient of A in Equation 5.31 and from Figure 5.102, the sorptivity increased with the water absorption rising. On the other hand, R value of 0.77 was an evidence for the poor relationship between the chloride permeability and sorptivity. Consequently, by using the proposed equation calculating the value of the sorptivity from water absorption value would not give precise results.

$$S = 1E - 05 \cdot A^{2.97} \quad (5.31)$$

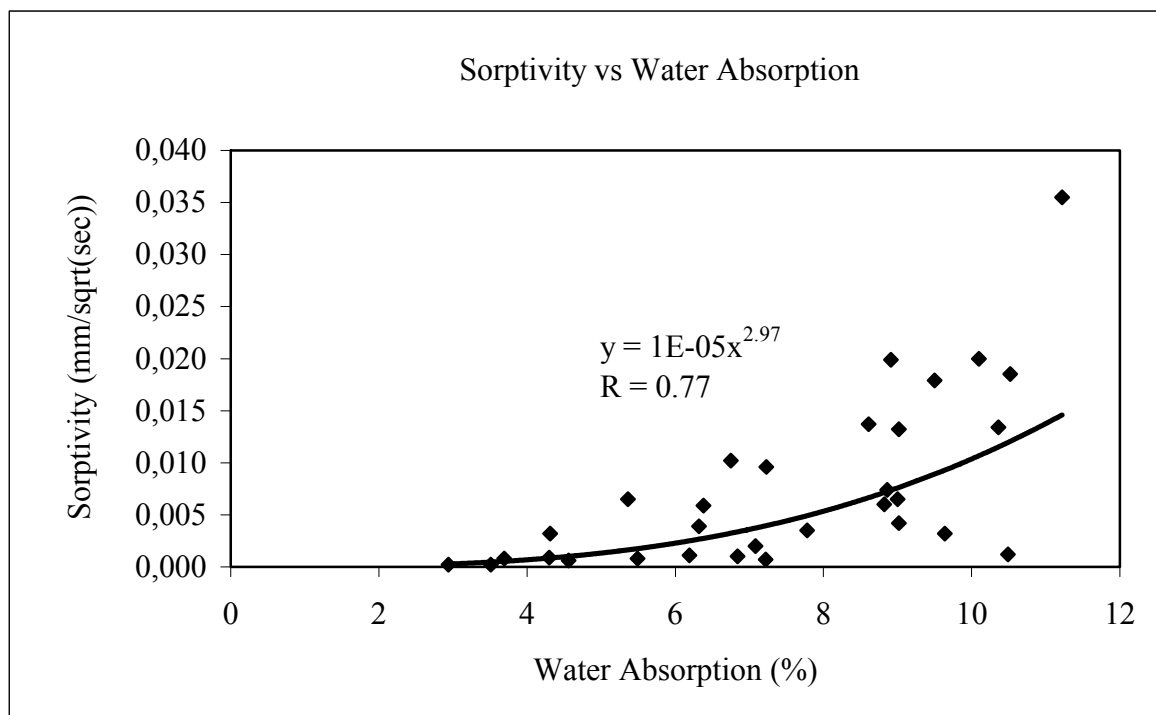


Figure 5.102. Relationship between sorptivity and water absorption

Regression analyses were performed also on the sub data of sorptivity and water absorption. Regression analyses results of that sub data are given in Table 5.37. The value of R given in Table 5.37 ranging between 0.47 and 0.94 proved the effects of the w/c ratio, replacement ratio, exposed temperature, and age factors.

Table 5.37. Relationship between sorptivity and water absorption at specific situations

Equation	R	Sub data
$S = 5E - 05 \cdot A^{1.58}$	0.87	Temperature= 22 °C
$S = 1E - 05 \cdot A^{2.75}$	0.88	Temperature = 105 °C
$S = 1E - 04 \cdot A^{2.10}$	0.94	Temperature = 175 °C
$S = 5E - 04 \cdot A^{1.45}$	0.47	Temperature = 250 °C
$S = 1E - 05 \cdot A^{3.17}$	0.83	Replacement Ratio= 0%
$S = 3E - 08 \cdot A^{6.25}$	0.77	Replacement Ratio= 25%
$S = 2E - 05 \cdot A^{2.38}$	0.93	Replacement Ratio= 50%
$S = 8E - 07 \cdot A^{4.12}$	0.66	Replacement Ratio= 100%
$S = 2E - 05 \cdot A^{2.42}$	0.66	Age= 28 days
$S = 5E - 06 \cdot A^{3.52}$	0.91	Age= 90 days
$S = 1E - 05 \cdot A^{2.97}$	0.77	All data included

5.7.9. Relationship between Total Charge Passed and Initial Current

Total charge passed and initial current were viewed in rapid chloride permeability test. Total charged passed considered the six hour recorded data whereas the initial current recorded immediately after the power supply turned on consider only the first second data on the same specimen. It was worth to investigate the relationship between total charge passed and initial current. The established relationship between the total charge passed and initial current using the actual test results of mixes 1-31 was plotted in Figure 5.103 and displayed in Equation 5.32. The equation was established by linear regression analysis.

$$Q = 5.73 \cdot IC - 2816 \quad (5.32)$$

where IC= initial current (ampere, A)

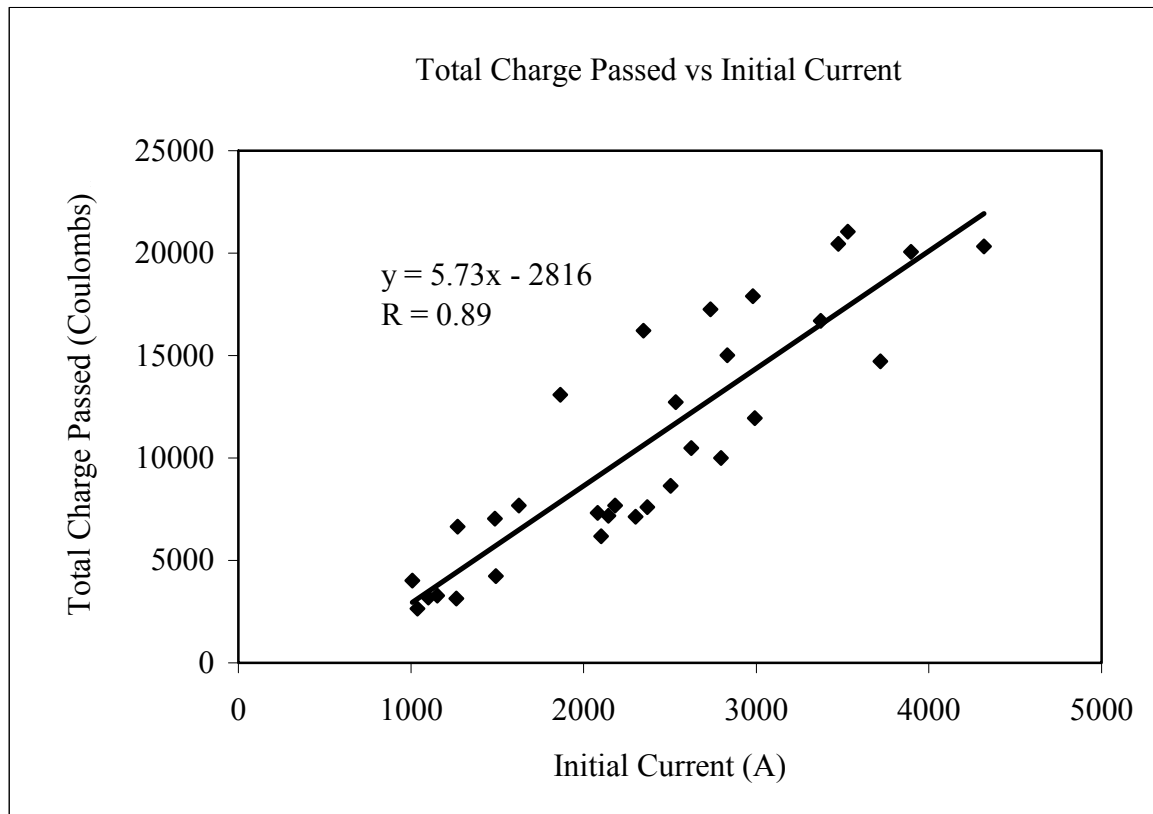


Figure 5.103. Relationship between total charge passed and initial current

The positive value of the coefficient of IC in Equation 5.32 illustrated that the total charge passed and initial current directly related to each other, if initial current was higher the total charge passed would be higher. The higher R value of 0.89 was encouraging the possibility of predicting the total charged passed from initial current, which will bring some advantages. Because total charged passed value could be obtained after six hours whereas initial current could be measured instantly. Regression analyses were performed also on the sub data of total charge passed and initial current. Regression analyses results of sub data are given in Table 5.38. The value of R given in Table 5.38 was ranging between 0.79 and 0.99 if the R value of 0.22 excluded. This wide range correlation coefficient showed that calculation of total charge passed from initial current was not that easy, since the circumstances changed the equation at every time.

Table 5.38. Relationship between total charge passed and initial current at specific situations

Equation	R	Sub data
$Q = 4.78 \cdot IC - 3403$	0.99	Temperature= 22 °C
$Q = 4.63 \cdot IC - 2221$	0.99	Temperature = 105 °C
$Q = 5.12 \cdot IC - 541$	0.99	Temperature = 175 °C
$Q = 5.76 \cdot IC + 1100$	0.97	Temperature = 250 °C
$Q = 5.07 \cdot IC - 1517$	0.79	Replacement Ratio= 0%
$Q = 1.81 \cdot IC + 7637$	0.22	Replacement Ratio= 25%
$Q = 6.41 \cdot IC - 5514$	0.88	Replacement Ratio= 50%
$Q = 6.04 \cdot IC - 4066$	0.89	Replacement Ratio= 100%
$Q = 5.50 \cdot IC - 2792$	0.91	Age= 28 days
$Q = 5.96 \cdot IC - 2883$	0.88	Age= 90 days
$Q = 5.73 \cdot IC - 2806$	0.89	All data included

Based on the relationships evaluated so far with our actual test results, it can be concluded that:

1. The effect of water/cement ratio, replacement ratio, exposed temperature, and age factors was obvious for the measured parameters in the thesis study. However, the degree of effect of each factor was changing from property to property.
2. No good correlation was established between the concrete properties satisfying the researcher to calculate one property from another. In fact in order to give reliable relations many data should be included to the analysis.
3. Relationships given in the literature [105, 116, and 127] were different from the ones found in topic 5.7; this reason was obvious because the researchers evaluate their equations to best fit their own experimental results. Further than that, it was observed in our study that the proposed equation changed every time depending on the selected data whether all or sub. The other reason for different equations was the

selected regression method; linear or power or exponential or polynomial and sometimes logarithmic model.

4. Though approximate relations presented either by ACI or other researchers used for reference, comparison and practical application purposes it is very important that relying on the relations may cause sometimes problem. Because interrelationships found between the mechanical and durability properties were differed from case to case and study to study. That is why, if one property is vital, instead of predicting it from the relationship equation, testing that property under specified service environment would be safe and sound. Maybe for this reason, there is a sentence put by the machine producers onto the Universal Testing Machine which reads “One experiment worth one thousand experts’ opinions”.

5.8. Microstructural Studies and Image Analysis on Hardened Concrete

5.8.1. Determination of Pore Structure by Mercury Intrusion Porosimetry (MIP)

Mercury Intrusion Porosimetry (MIP) test result revealed the applied pressure and corresponding cumulative intruded mercury volume at each step. Mercury intrusion data by itself provided no information at all on the distribution of pore sizes of the concrete specimens. For such information to be generated some calculations must be done. First calculation was the converting of pressure unit from psi to MPa. Washburn equation, Equation 4.28, was applied to calculate the diameter of cylindrical pores intruded at each pressuring step. However, it is well known that MIP does not give a complete and exact representation of pore structure [119-120]. According to Yang [120] this is because mercury must pass through the narrowest pores connecting the pore network before entering the samples, large pores accessible through narrow necks can be recorded as small pores; in addition to that because of the high mercury pressures used to access fine pores, C-S-H gel can collapse and therefore lead to an incorrect estimation of pore volume. Diamond [119] extensively stated the reasons for mercury porosimetry as an inappropriate method for the measurement of pore size distributions in cement based materials. According to him, applied Washburn equation to calculate the diameter from pressure was

not appropriate, because Washburn equation assumes that pores are cylindrical, but the pores in the cement based materials do not fulfill this requirement. Secondly, there is no distinction in the model between intrusion into a single long and continuous cylinder and intrusion into a multitude of shorter cylinders of the same diameter. Nevertheless, MIP remains a powerful tool mainly for comparative analysis of hydrated cements of various ages and w/c ratios, or subjected to different external conditions [120].

The range of the pore diameters, threshold diameter, and mean diameter were all obtained from mercury porosimetry test results. Minimum diameter of the pores was determined from the largest exerted pressure value, while the maximum diameter was calculated from the pressure at which intruded mercury pore volume was initiated in the data records. Threshold diameter was the largest pore diameter at which significant intruded volume was detected. Mean diameter was calculated from the following equation:

$$d = \frac{\sum_{i=1}^{i=n} V_i * d_i}{\sum_{i=1}^{i=n} V_i} \quad (5.33)$$

where:

V_i = incremental cumulative pore volume corresponding to i th pore size range represented by mean diameter “ d_i ”

d = mean pore size

Since the pore sizes vary logarithmically, mean distribution pore diameter was also calculated as given in Equation 5.34.

$$\ln d = \frac{\sum_{i=1}^{i=n} V_i * \ln d_i}{\sum_{i=1}^{i=n} V_i} \quad (5.34)$$

The analysis results of pore characteristics in concrete mixes 1-31 are summarized in Table 5.39

Table 5.39. Analysis results of MIP tests

Run	A:w/c	B:RR (%)	C:T (°C)	Age (days)	Pore diameter range (nm)		Threshold diameter (nm)	Mean diameter (nm)	Mean distribution diameter (nm)
					min	max			
1	0.35	0	22	90	6.16	762.97	204.83	36.69	21.03
2-3	0.35	0	105	28	6.21	40.70	37.53	15.85	13.90
4	0.35	0	175	28	6.24	9871.55	86.78	31.09	18.81
5	0.35	0	250	28	6.15	1466.95	99.67	25.59	18.09
6	0.35	50	22	28	6.17	9871.56	122.24	31.43	19.18
7	0.35	50	105	90	6.15	8319.65	10.63	26.55	10.80
8	0.35	100	22	90	6.21	2075.01	8.27	27.23	16.96
9	0.35	100	105	28	6.41	10141.82	177.54	33.86	20.54
10	0.35	100	175	90	6.16	10.33	10.18	7.75	7.66
11	0.35	100	250	90	6.19	1566.18	62.88	24.01	16.67
12	0.4125	25	250	90	6.21	9871.53	1320.53	220.94	38.39
13-14	0.475	0	175	90	6.19	2511.03	7.46	20.57	15.04
15	0.475	25	22	28	6.16	7962.03	281.81	47.53	21.74
16	0.475	50	250	28	6.15	2545.55	1084.67	57.09	23.26
17	0.475	100	105	90	6.21	6671.49	27.15	25.52	16.96
18	0.475	100	175	28	6.22	5485.91	14.50	18.10	12.05
19	0.5375	25	175	28	6.17	13.30	13.15	9.24	9.01
20	0.5375	50	22	90	6.26	10141.79	303.51	51.35	21.73
21	0.5375	75	250	90	6.19	1527.44	455.36	46.00	21.97
22	0.6	0	22	28	6.21	352.29	91.46	26.97	18.44
23-24	0.6	0	105	90	6.22	6.74	6.30	6.44	6.43
25	0.6	0	250	90	6.19	10141.84	199.75	64.49	21.60
26-27	0.6	50	105	28	6.16	8319.65	19.04	13.88	11.88
28	0.6	100	22	28	6.20	1726.80	350.96	45.57	22.06
29-30	0.6	100	175	90	6.19	76.75	46.67	20.53	16.33
31	0.6	100	250	28	6.21	370.25	37.09	23.35	16.51

As seen from Table 5.39, the smallest size of pore into which mercury could be intruded was about 6 nm (nanometer). Thus, the pressure was adequate to ensure intrusion of the mercury into almost all capillary pores, as the reported capillary pore in Figure 4.16 was in the range of 3 nm to 10 microns. On the other hand, majority of gel pores remained nonintruded. The range and distribution of the pores are important pore structure characteristics determining the performance and properties of concrete. Distribution of the pore sizes could be best represented through pore size distribution curves, as given in Figures 5.104-5.108. As seen from these figures, pore size distributions of mixes 1-31 were different from each other, which clearly demonstrated that differences in pore structure caused different mechanical and durability properties. It was again observed that some mixes (mix 2, 10, etc) showed a sharp pore size distribution; this sudden changes may be resulted from applied pressure rate or expansion of samples under pressure. The mix to mix variation in pore size distributions mainly occurred because of the mix class, and it was very hard from these figures to make simultaneous comparisons between the mixes. That is why analysis of variance (ANOVA) was used to better understand which factor affected the pore size distribution. In order to do that maximum pore diameter, threshold diameter, and mean diameter values entered to the Design Expert software. The results of the ANOVA for each pore characteristic are presented in Tables 5.40-5.42. Based on the ANOVA results listed in Tables 5.40-5.42, it could be said that both evaluated maximum pore diameter and mean diameter were varied with temperature (C) changes, while threshold diameter changed together with replacement ratio and temperature (BC) variable; that means there was an interaction. In addition mean distribution pore diameter was studied extensively. 2FI model was used for analysis of mean distribution pore diameter values, base 10 Log was used for the data transformations. The standard deviation and the R-squared values of the model were 0.091 and 0.89. The R-squared value of 0.89 showed that 2FI model was good for mean distribution pore diameter analysis. The results of the ANOVA for response surface 2FI model are presented in Table 5.43. As seen from ANOVA table the model terms C, AB and BC are significant, because the Prob>F values are less than 0.05. This means that the temperature factor has important effects on mean distribution pore diameter. Besides AB and BC factors were significant but their interactions showed that the effect of one factor depends on the level of other. The mean distribution pore diameter values are demonstrated in Figures 5.109-5.116.

The water/cement ratio was not found as a significant factor by analysis of variance results, the role of w/c ratio on mean distribution pore diameter is seen from Figures 5.109-5.116. In fact it is well known that the pore structure is affected by w/c ratio very much, because more pores and diffusing paths may form as the w/c ratio increases. However it should be considered that MIP test results depends on some factors. First of all, the possible largest dimension of the sample cell limits the size of the concrete sample used in MIP. The small concrete sample, about 1 g, may not be representing the overall quality of concrete in the mixes 1-31. Secondly as stated above maybe the concrete samples expanded under pressure or compressed under pressure. Thirdly, if there was an error in the test results it could not be found because only one sample tested for each mix.

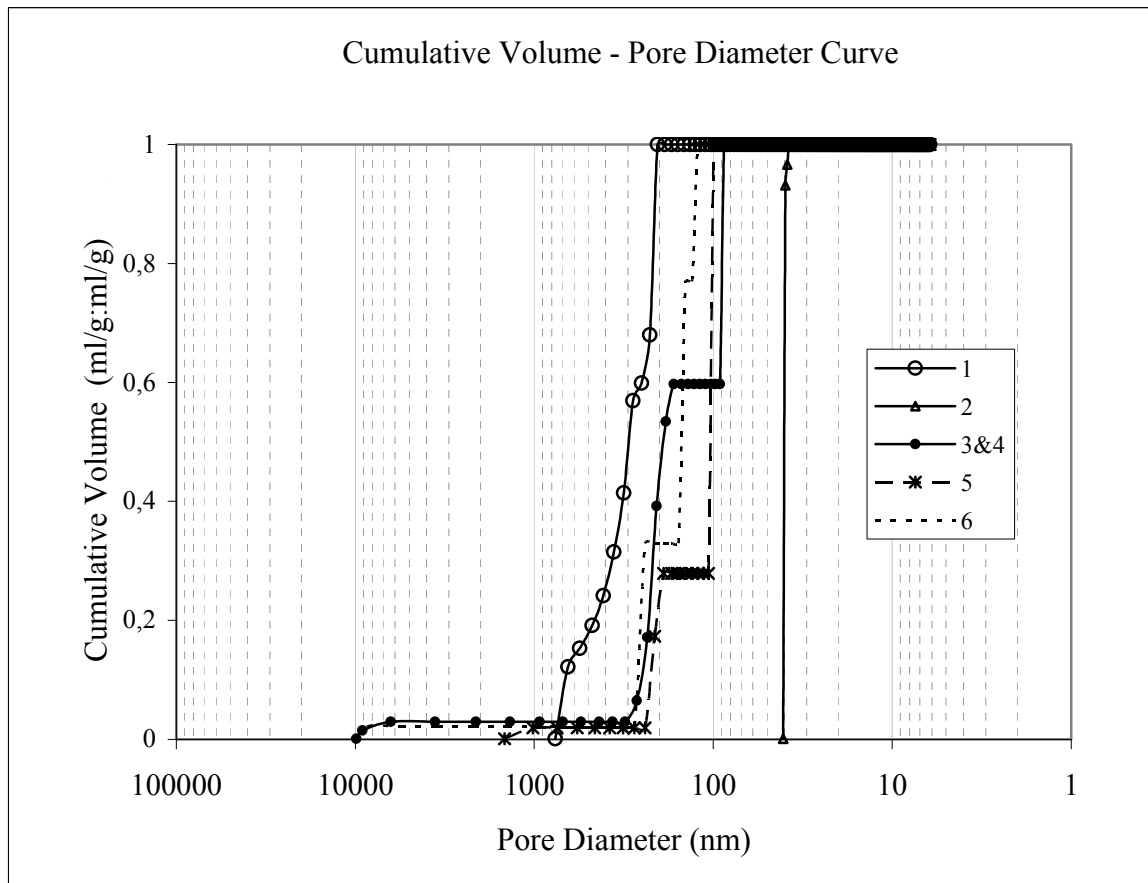


Figure 5.104. Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 1-6

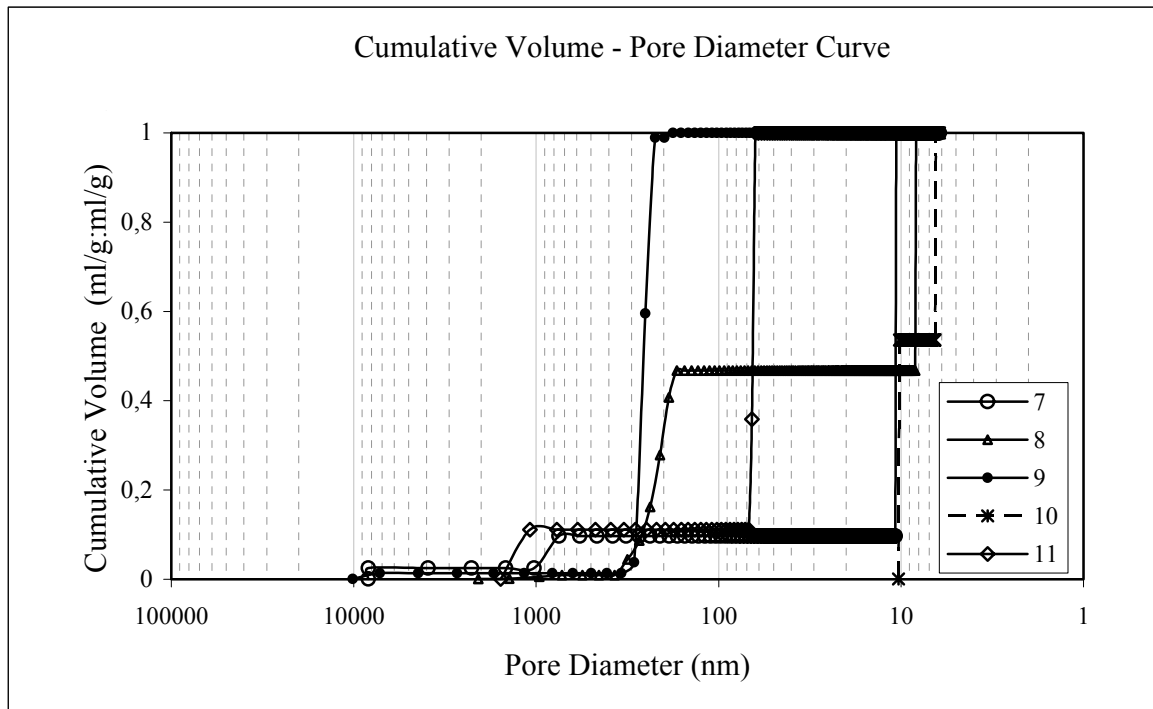


Figure 5.105. Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 7-11

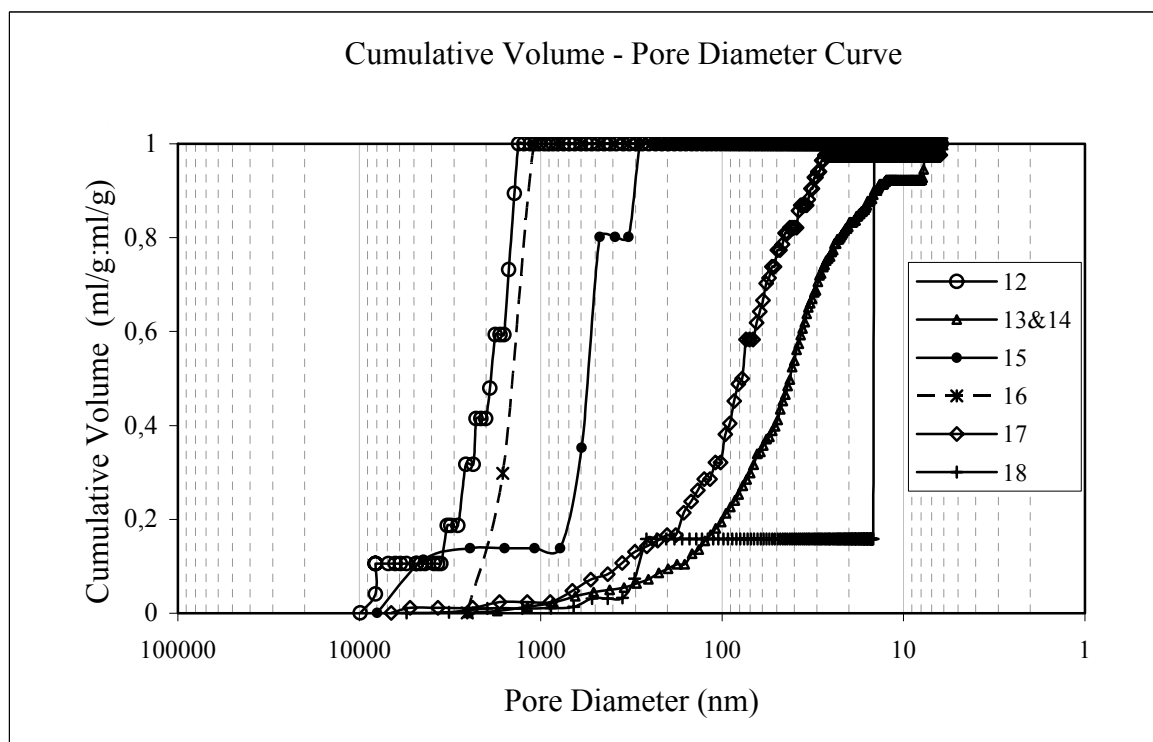


Figure 5.106. Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 12-18

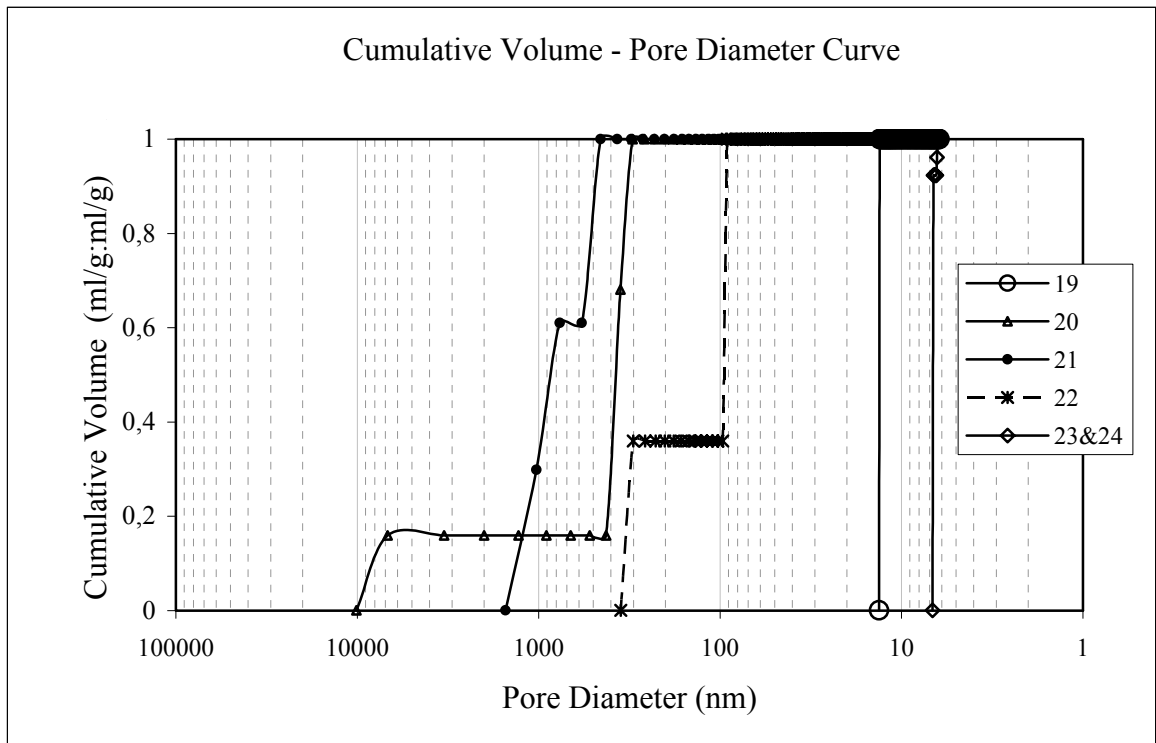


Figure 5.107. Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 19-24

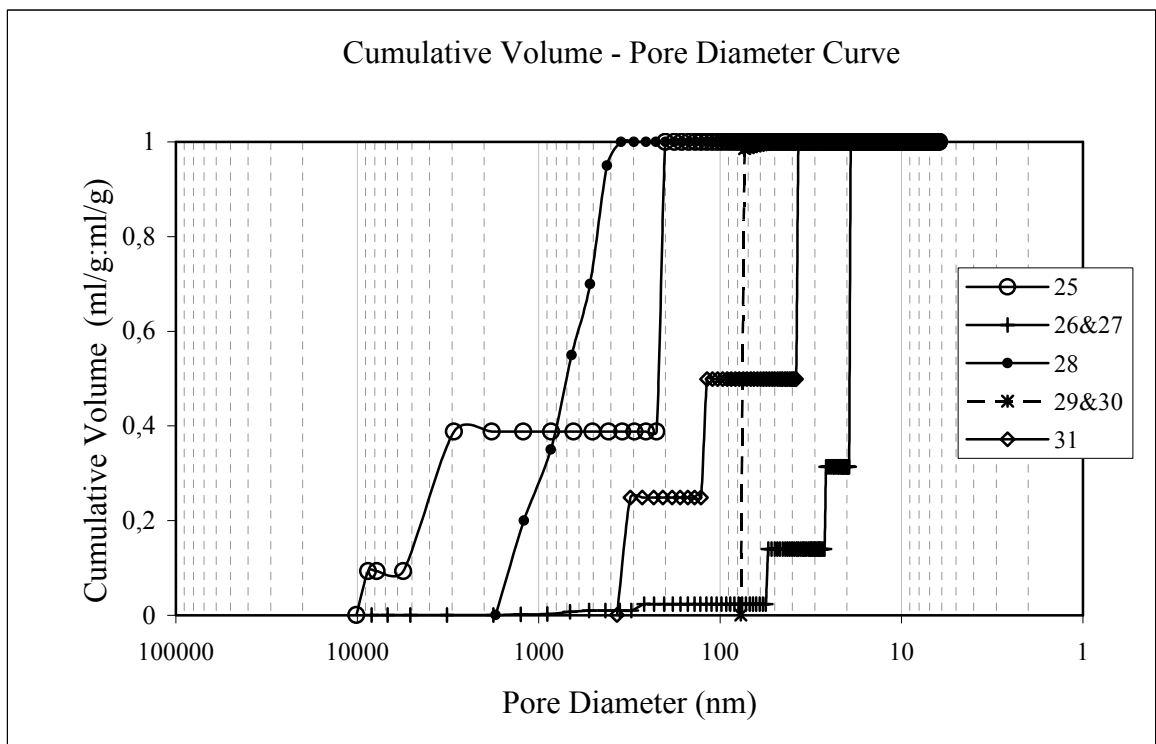


Figure 5.108. Normalized cumulative intruded pore volume versus pore size distribution curves obtained from MIP for mixes 25-31

Table 5.40. Results of ANOVA for maximum pore diameter surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	0.25	1	0.25	0.28	0.6069
B	0.64	1	0.64	0.71	0.4149
C	3.26	3	1.09	1.20	0.3513
D	0.04	1	0.04	0.04	0.8464
AB	1.21	1	1.21	1.34	0.2701
AC	0.57	3	0.19	0.21	0.8879
AD	0.02	1	0.02	0.02	0.8945
BC	13.24	3	4.41	4.88	0.0191
BD	0.77	1	0.77	0.85	0.3747
CD	2.00	3	0.67	0.74	0.5503

Table 5.41. Results of ANOVA for threshold pore diameter surface linear model

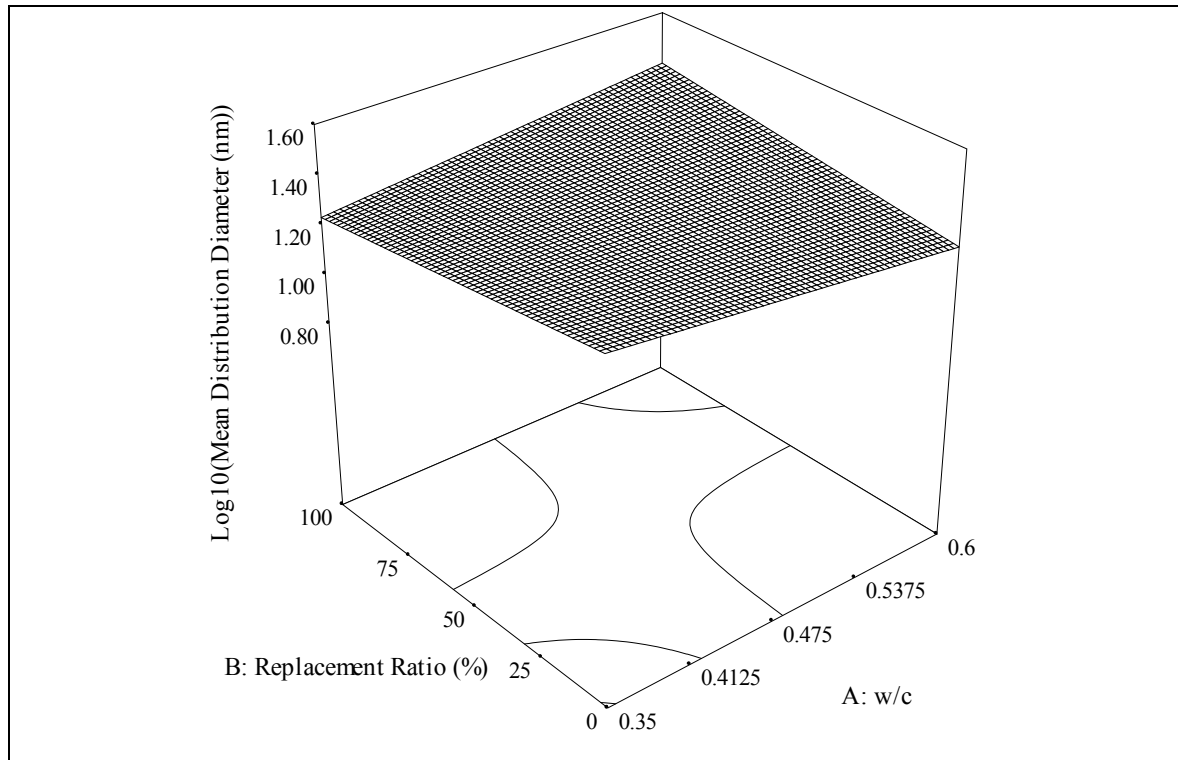
Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	0.01	1	0.01	0.05	0.8225
B	0.00	1	0.00	0.02	0.9025
C	6.60	3	2.20	8.12	0.0007
D	0.42	1	0.42	1.56	0.2239

Table 5.42. Results of ANOVA for mean pore diameter surface linear model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	0.053	1	0.05	0.82	0.3739
B	0.001	1	0.00	0.01	0.9247
C	1.347	3	0.45	6.98	0.0015
D	0.006	1	0.01	0.09	0.7698

Table 5.43. Results of ANOVA for mean distribution pore diameter surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	0.003	1	0.003	0.331	0.5758
B	0.002	1	0.002	0.193	0.6680
C	0.389	3	0.130	15.672	0.0002
D	0.003	1	0.003	0.360	0.5596
AB	0.076	1	0.076	9.124	0.0107
AC	0.013	3	0.004	0.531	0.6694
AD	0.015	1	0.015	1.818	0.2025
BC	0.144	3	0.048	5.796	0.0110
BD	0.002	1	0.002	0.219	0.6485
CD	0.072	3	0.024	2.888	0.0795

Figure 5.109. 3D surface view of mean distribution pore diameter
(age: 28 days, temperature: 22 °C)

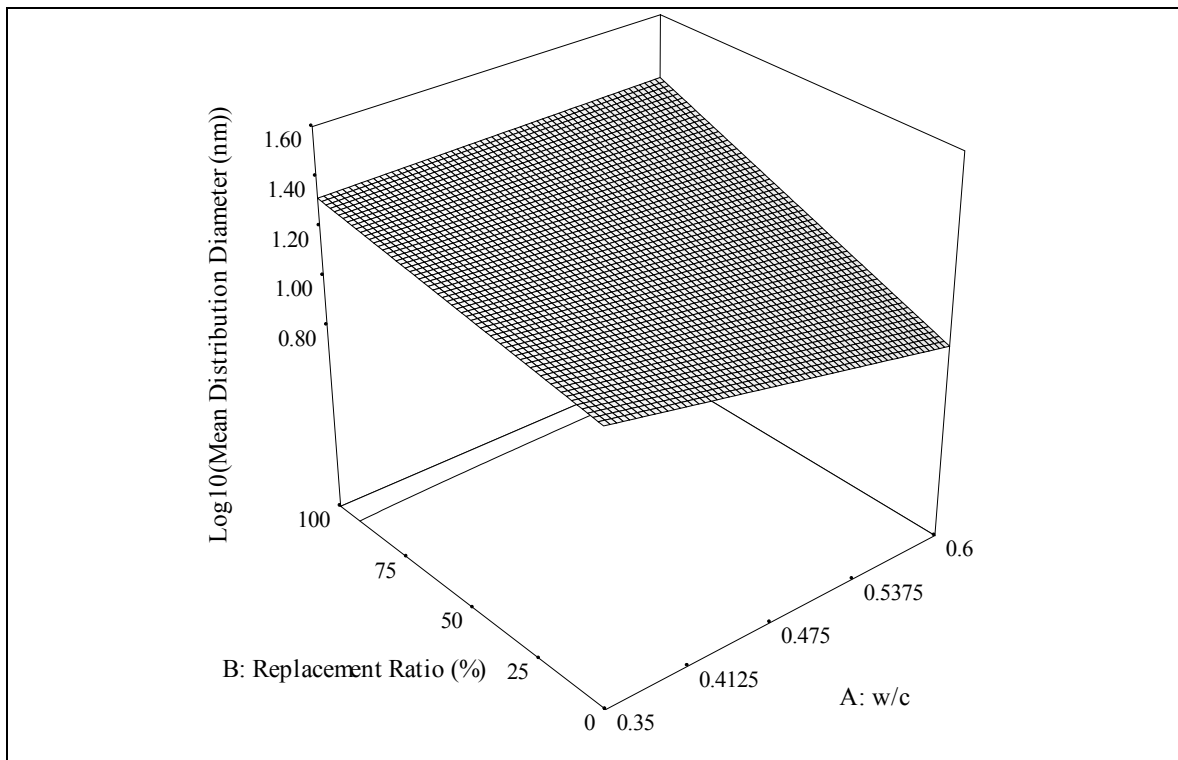


Figure 5.110. 3D surface view of mean distribution pore diameter
(age: 28 days, temperature: 105 °C)

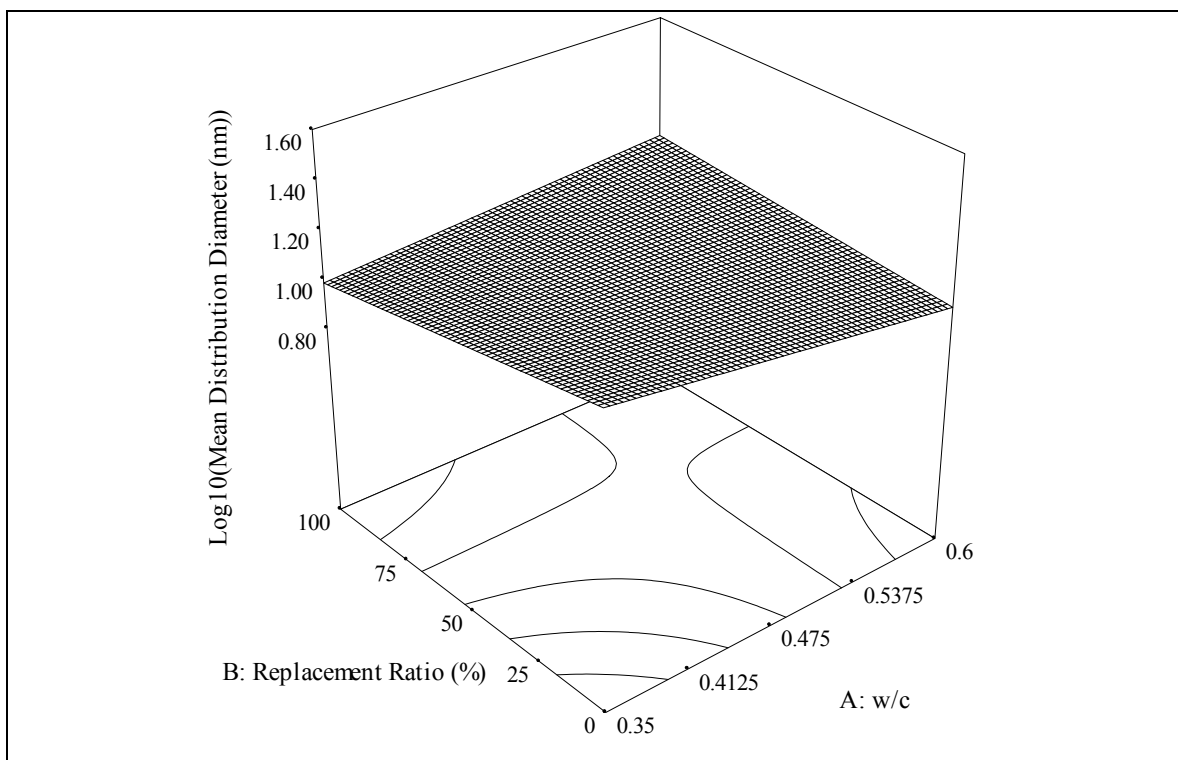


Figure 5.111. 3D surface view of mean distribution pore diameter
(age: 28 days, temperature: 175 °C)

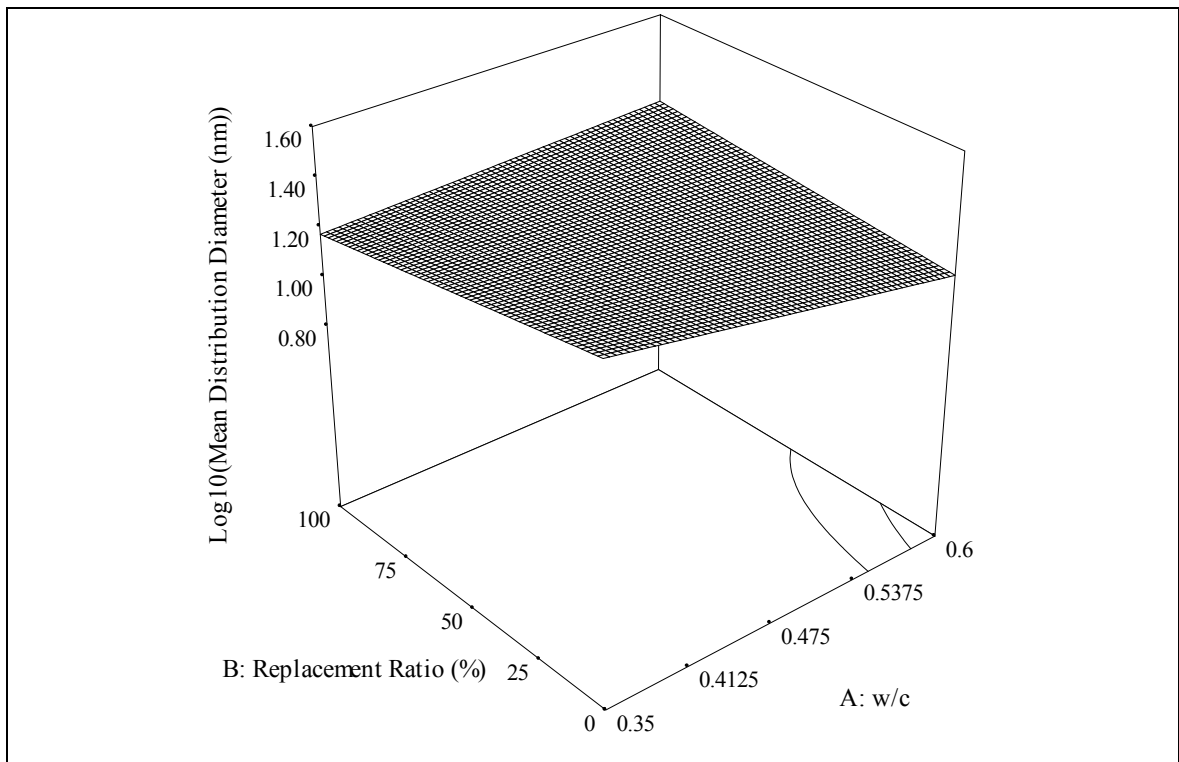


Figure 5.112. 3D surface view of mean distribution pore diameter
(age: 28 days, temperature: 250 °C)

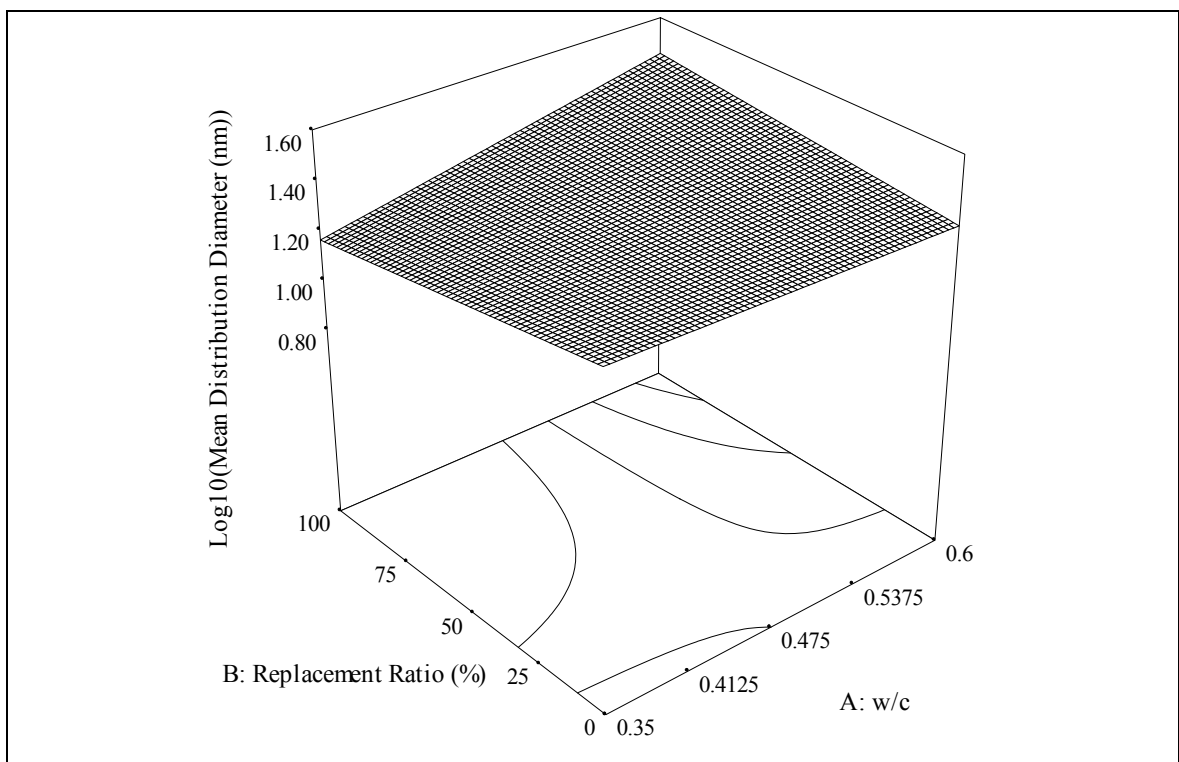


Figure 5.113. 3D surface view of mean distribution pore diameter
(age: 90 days, temperature: 22 °C)

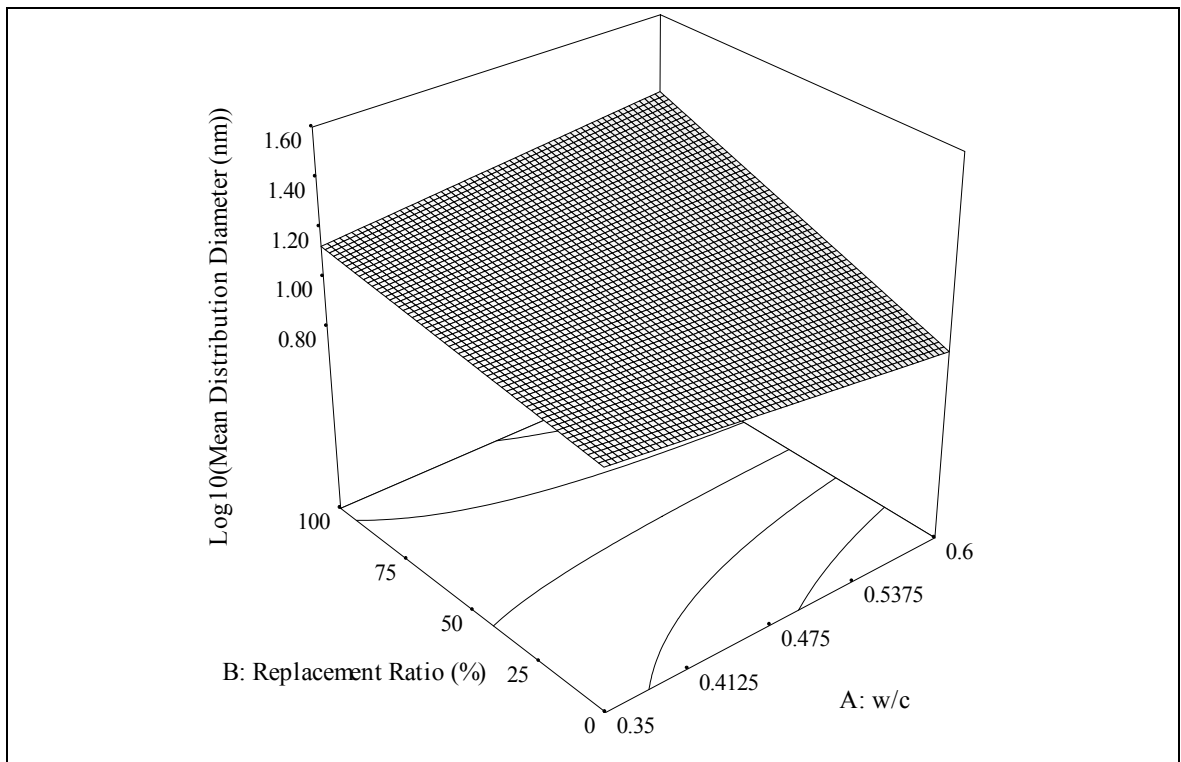


Figure 5.114. 3D surface view of mean distribution pore diameter
(age: 90 days, temperature: 105 °C)

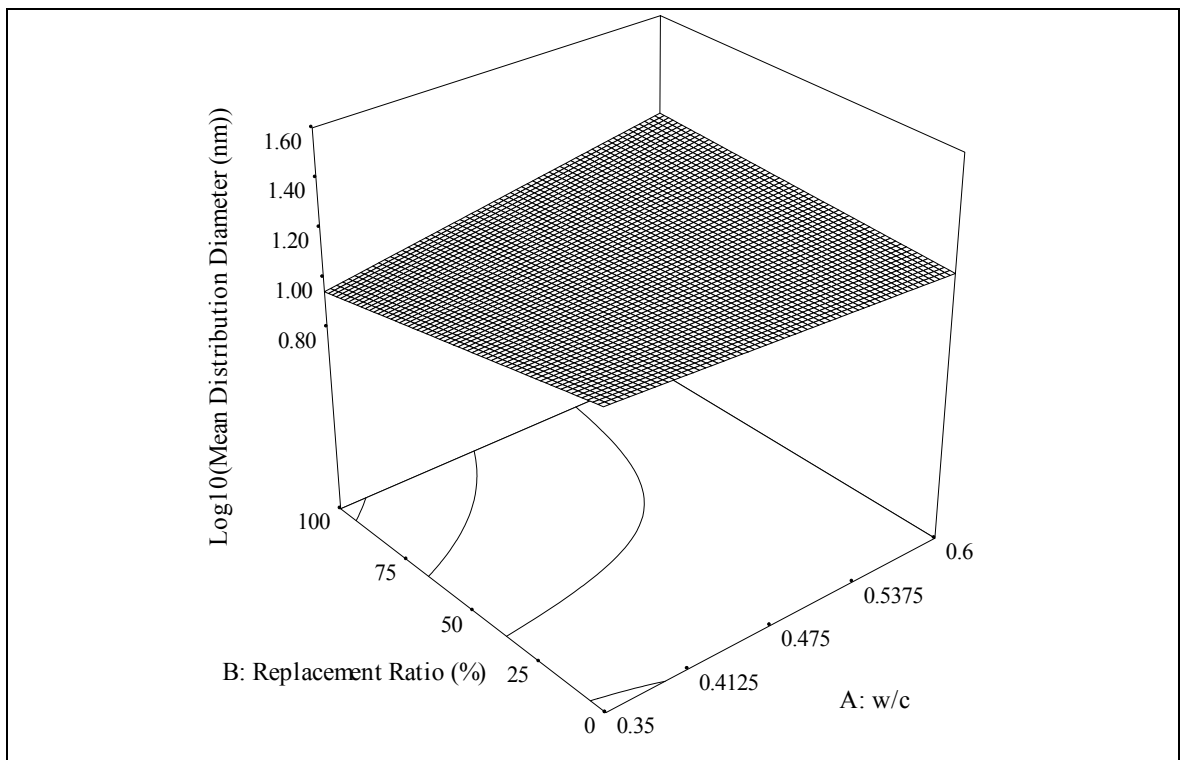


Figure 5.115. 3D surface view of mean distribution pore diameter
(age: 90 days, temperature: 175 °C)

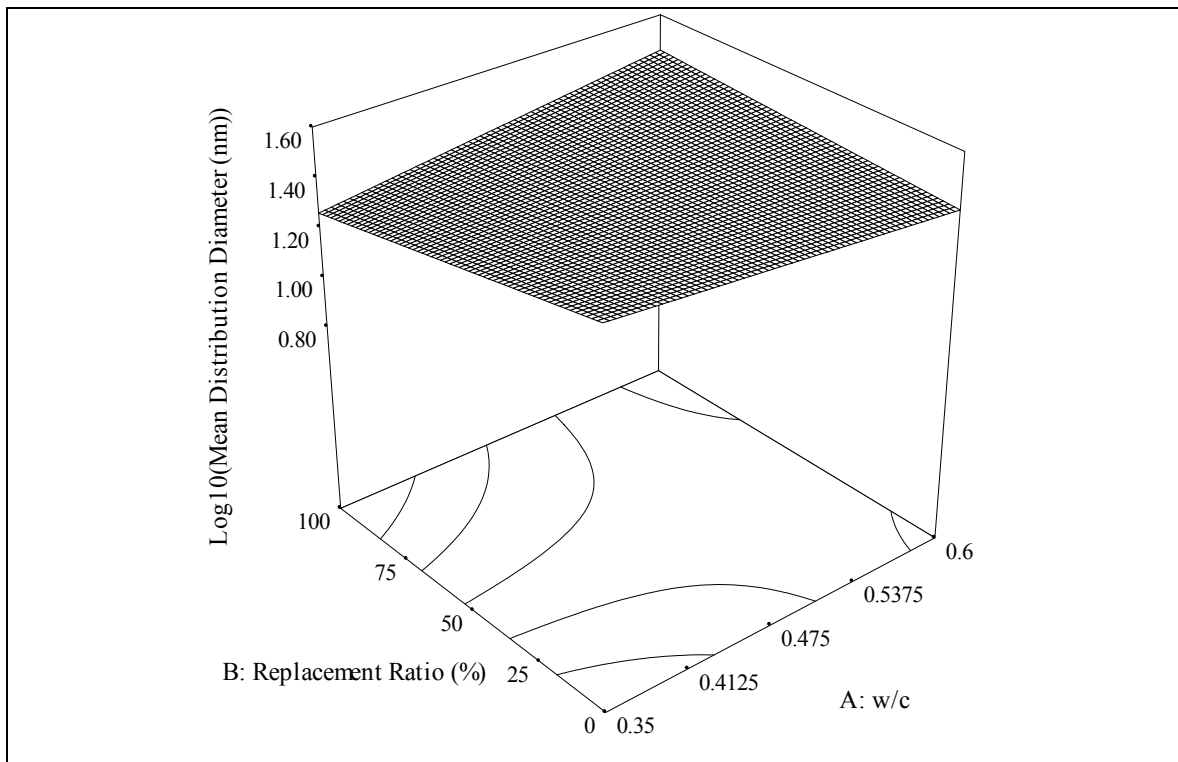


Figure 5.116. 3D surface view of mean distribution pore diameter
(age: 90 days, temperature: 250 °C)

5.8.2. Examination of Interfaces Using Optical Microscope (OM) and Scanning Electron Microscope (SEM)

Optical microscope (OM) and scanning electron microscope (SEM) observations were conducted on studying the interfacial transition zone of concrete samples of mixes 1-31. Entrained airs, capillary pores, boundary cracks and paste cracks were all evaluated with the use of the micrographs taken through both OM and SEM devices. The summary of this evaluation is seen in Table 5.44. In the table “+” sign was used if the observed property existed in that interface and minus sign was used if it did not exist. The size of the entrained air was larger than 10 microns according to Figure 4.16; according to this information entrained airs were observed at OM micrographs, which provided that all pieces of concrete regardless of interface contains these larger voids. Secondly it was observed that recycled aggregate concrete pieces were more porous than natural aggregate concrete pieces, due to the attached mortar. No crack was observed at 150x magnification using OM. An example of OM micrograph is illustrated in Figure 5.117. Capillary pore, crack and crack patterns were examined using SEM micrographs. Paste crack and

boundary crack are seen in Figures 5.118 and 5.119, respectively. The interfaces in the Table 5.44 were numbered according to the following representation:

- Interface 1: interface between natural aggregate and new mortar
- Interface 2: interface between adhered mortar and original coarse aggregate
- Interface 3: interface between adhered mortar and new mortar
- Interface 4: interface between original coarse aggregate and new mortar

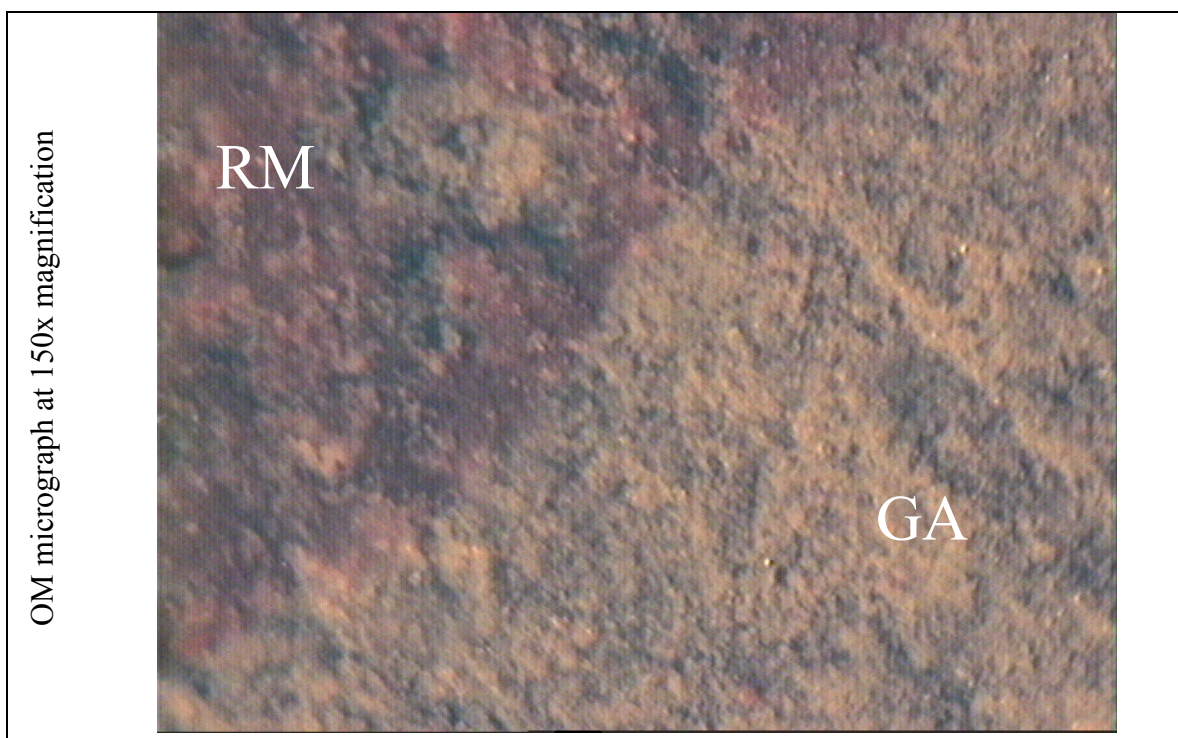


Figure 5.117. Sectional view of interface #1 (interface between natural (gray) aggregate (GA) and new (red) mortar (RM) of mix 6)

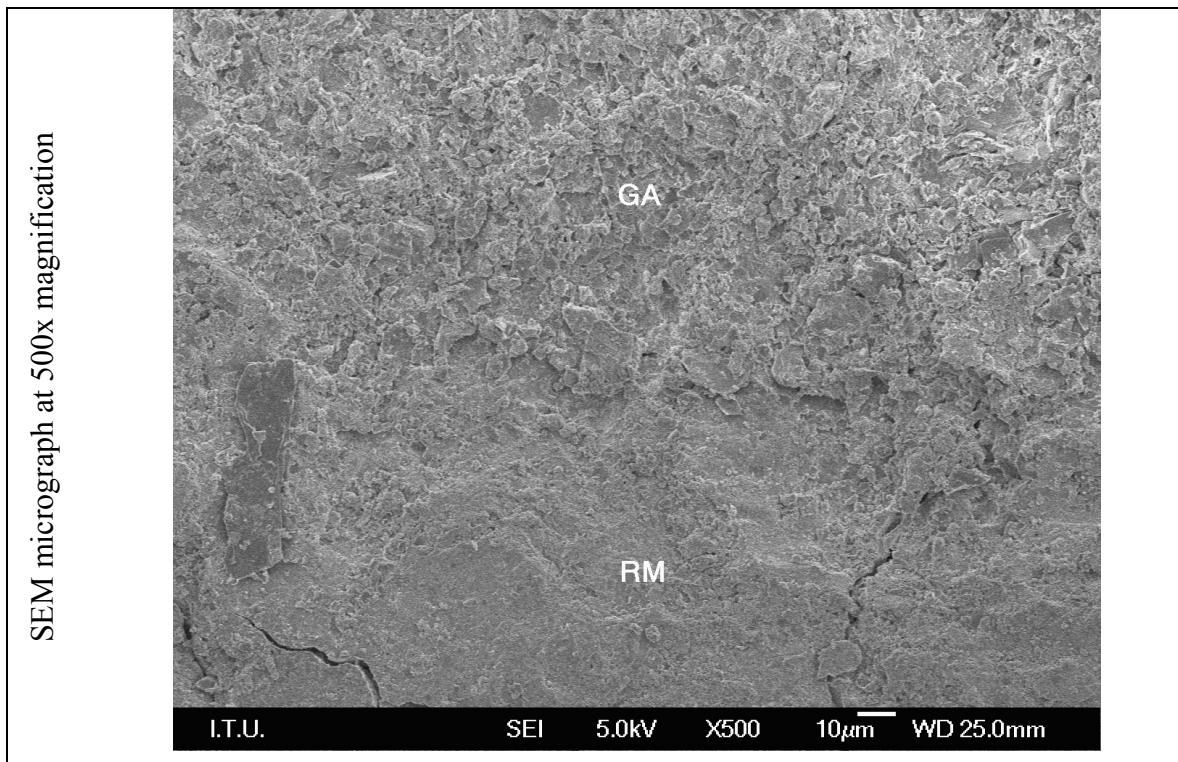


Figure 5.118. Sectional view of interface #1 (interface between natural (gray) aggregate (GA) and new (red) mortar (RM) of mix 6)

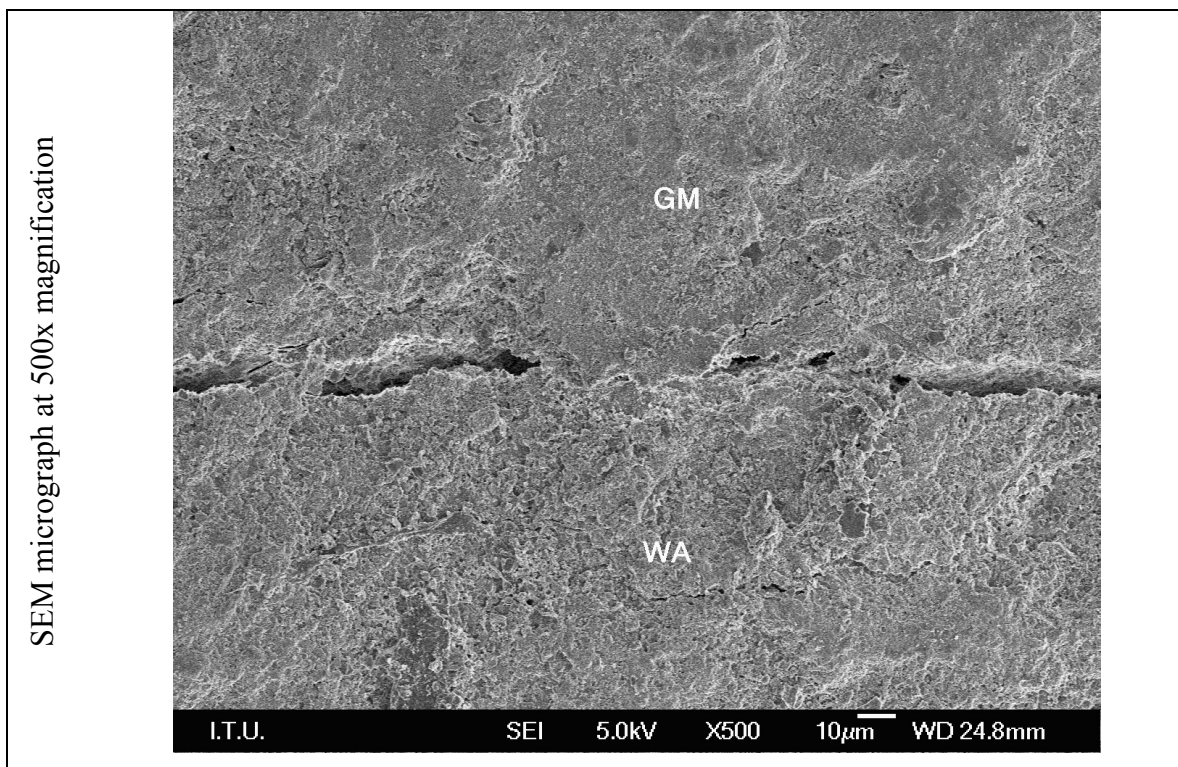


Figure 5.119. Sectional view of interface #2 (interface between adhered (gray) mortar (GM) and original coarse (white) aggregate (WA) of mix 6)

Table 5.44. Evaluation of interface micrographs

Run	w/c	RR (%)	T (°C)	Age (days)	Observation	Interface 1	Interface 2	Interface 3	Interface 4
1	0.35	0	22	90	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
2	0.35	0	105	28	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
3	0.35	0	105	28	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
4	0.35	0	175	28	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
5	0.35	0	250	28	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
6	0.35	50	22	28	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	-	+	-	+
					paste crack	+	-	-	+
7	0.35	50	105	90	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	-	-	-
					paste crack	-	-	-	-
8	0.35	100	22	90	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	-	+
					paste crack		-	-	-
9	0.35	100	105	28	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		-	-	+
					paste crack		-	-	-

Table 5.44. (cont.) Evaluation of interface micrographs

10	0.35	100	175	90	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	-	-
					paste crack		-	-	-
11	0.35	100	250	90	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	+	+
					paste crack		-	-	+
12	0.4125	25	250	90	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	+	-	-
					paste crack	-	-	-	-
13	0.475	0	175	90	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
14	0.475	0	175	90	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
15	0.475	25	22	28	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	-	+	-
					paste crack	-	-	-	-
16	0.475	50	250	28	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	+		-
					paste crack	-	+		-
17	0.475	100	105	90	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	-	+
					paste crack		-	-	-
18	0.475	100	175	28	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		-	-	-
					paste crack		-	-	-

Table 5.44. (cont.) Evaluation of interface micrographs

19	0.5375	25	175	28	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	+	-	+
					paste crack	-	-	-	-
20	0.5375	50	22	90	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	-	-	-	+
					paste crack	-	-	-	-
21	0.5375	75	250	90	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	-	+	+	-
					paste crack	-	-	+	-
22	0.60	0	22	28	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	+			
					paste crack	+			
23	0.60	0	105	90	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
24	0.60	0	105	90	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
25	0.60	0	250	90	entrained air	+	No ITZ		
					capillary pore	+			
					boundary crack	-			
					paste crack	-			
26	0.60	50	105	28	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	-	-	-
					paste crack	-	-	-	+
27	0.60	50	105	28	entrained air	+	+	+	+
					capillary pore	+	+	+	+
					boundary crack	+	-	-	-
					paste crack	-	-	-	+

Table 5.44. (cont.) Evaluation of interface micrographs

28	0.60	100	22	28	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	-	+
					paste crack		-	-	-
29	0.60	100	175	90	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	+	-
					paste crack		-	-	-
30	0.60	100	175	90	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		+	+	-
					paste crack		-	-	-
31	0.60	100	250	28	entrained air	No ITZ	+	+	+
					capillary pore		+	+	+
					boundary crack		-	-	-
					paste crack		-	-	-

As seen from Table 5.44, natural aggregate concretes has only one kind of interface (interface 1), whereas recycled aggregate concretes with 25, 50, and 75 percent replacement ratio have four kinds (interface 1-4) and 100 percent replacement ratio have three kinds (interface 2-4) of interfaces.

It was clear from evaluation table that interfaces were influenced differently with the varying factors. To identify which interface involved more cracks, the cracks with “+” were counted; a summary of this investigation is listed in Table 5.45. Based on that observation summary it could be said that paste cracks did not happen as much as boundary cracks. The paste cracks were observed 5-20 percent in the interfaces, while the percentage of boundary cracks was 26.3-60 percent, which showed that interfacial transition zones were weaker than the pastes. In general, ITZ of all kinds is the weakest link of the chain. Among the interfaces, interface between adhered mortar and new mortar (interface 3) involved the lowest paste cracks with a percentage of 5.3 despite both sides of the interface contained mortar. On the other hand, interface between original coarse aggregate and new mortar, interface 4, occupied the greatest number of paste cracks with a

percentage of 20. Boundary cracks were occurred mostly on the interfaces between adhered mortar and original coarse aggregate and lesser on the interfaces between adhered mortar and new mortar. It is seen that, the strongest interfaces were observed between adhered mortar and new mortar which may be attributed to the chemical reaction, and the weakest interfaces were interface between adhered mortar and original coarse aggregate. The high percentage of boundary cracks at interfaces between adhered mortar and original coarse aggregate may be induced with crushing process.

It was very obvious that recycled aggregate concretes had more boundary cracks than natural aggregate concretes, because the number of interfaces in recycled aggregate concretes was higher than that of natural aggregate concrete. Natural aggregate concretes suffered more from boundary cracks at w/c ratio of 0.60, while the recycled aggregate concretes experienced boundary cracks at all w/c ratios of this study.

In order to deduce the temperature effect on crack formation, occurred cracks were counted at each exposed temperature and summarized in Table 5.46. It is seen from Table 5.46 that cracks were not only induced with increasing temperatures, because at temperature of 22 °C both boundary cracks and paste cracks were formed. However, these cracks were generally formed in the recycled aggregate concretes, which showed that these cracks were most probable induced during production of the recycled aggregate. Beyond that, cracks may have formed in sample preparation, as samples were obtained from cutting of concrete specimen. On the other hand, comparison of formed boundary cracks and paste cracks at 105, 175 and 250 °C revealed that both boundary cracks and paste cracks increased in number with increasing temperature. The percentage of total cracks was 36.4, 42.1 and 63.2 percent at 105, 175 and 250 °C, respectively. Uneven volume changes of the concrete components caused cracks at higher temperatures. The lowest number of cracks which were observed at 105 °C with a percentage of 36.4 may be attributed to the autoclaving. Although temperature increases beyond 105 °C increased the formed cracks, percentage of total cracks was in the range of 36.4-63.2 which was lower than the percentage value of 65 percent of 22 °C. The temperature range between 100-300 °C is the most favorable for the formation of internal autoclaving because in this temperature range steam is liberated most intensively; additional hydration of the

unhydrated cement grains is the result of the steam effect under the condition of internal autoclaving [105].

Table 5.45. Summary of interface comparison

	Interface 1	Interface 2	Interface 3	Interface 4
number of observed interfaces	21	20	19	20
number of observed boundary cracks	8	12	5	8
number of observed paste cracks	2	1	1	4
total number of cracks	10	13	6	12
percentage of boundary crack	38.1%	60.0%	26.3%	40.0%
percentage of paste crack	9.5%	5.0%	5.3%	20.0%
percentage of total crack	47.6%	65.0%	31.6%	60.0%

Table 5.46. Effect of temperature on crack formation

	22 °C	105 °C	175 °C	250 °C
number of observed interfaces	20	22	19	19
number of observed boundary cracks	10	6	8	9
number of observed paste cracks	3	2	0	3
total number of cracks	13	8	8	12
percentage of boundary crack	50,0%	27,3%	42,1%	47,4%
percentage of paste crack	15,0%	9,1%	0,0%	15,8%
percentage of total crack	65,0%	36,4%	42,1%	63,2%

The observed cracks and crack patterns were examined by plain mathematical analysis. Examination of capillary pores needed a sophisticated analysis named image analysis. SEM micrographs were used in image analysis to measure the concrete porosity. To do that, micrographs were opened using the software Image J. Since the taken micrographs were interface images, a rectangular piece from the image was selected that included only mortar part. Then the threshold values adjusted using auto threshold option. An example of this process done with the micrograph of mix 6 showing the sectional view

of interface between natural aggregate (GA) and new mortar (RM) is illustrated in Figure 5.120. Auto thresholding was applied for each set of interface micrographs. No judgment was done in thresholding. Auto thresholding scanned all image and marked pores as black. The auto black&white threshold value was in the range of 0-121 in part c of Figure 5.120. Based on the pixel calculations it could be said that capillary pores larger than 180 nm could be detected with auto thresholding. By judging on the level of upper threshold values the minimum size of the capillary pores could be increased. Examples of auto thresholding and personal judged thresholding values are given in Figure 5.121. It is seen from the figure that changing the thresholding value changed the area fraction (porosity). Due to this reason auto thresholding was selected at every set of images. After the threshold image was obtained, then the area percentage was computed using analyze icon of the software. Porosity values were computed for both new mortar and adhered mortar. New mortar was present at interfaces 1, 3 and 4, while adhered mortar existed at interfaces 2 and 3. The results of image analysis showing the porosity computations are given in Table 5.47.

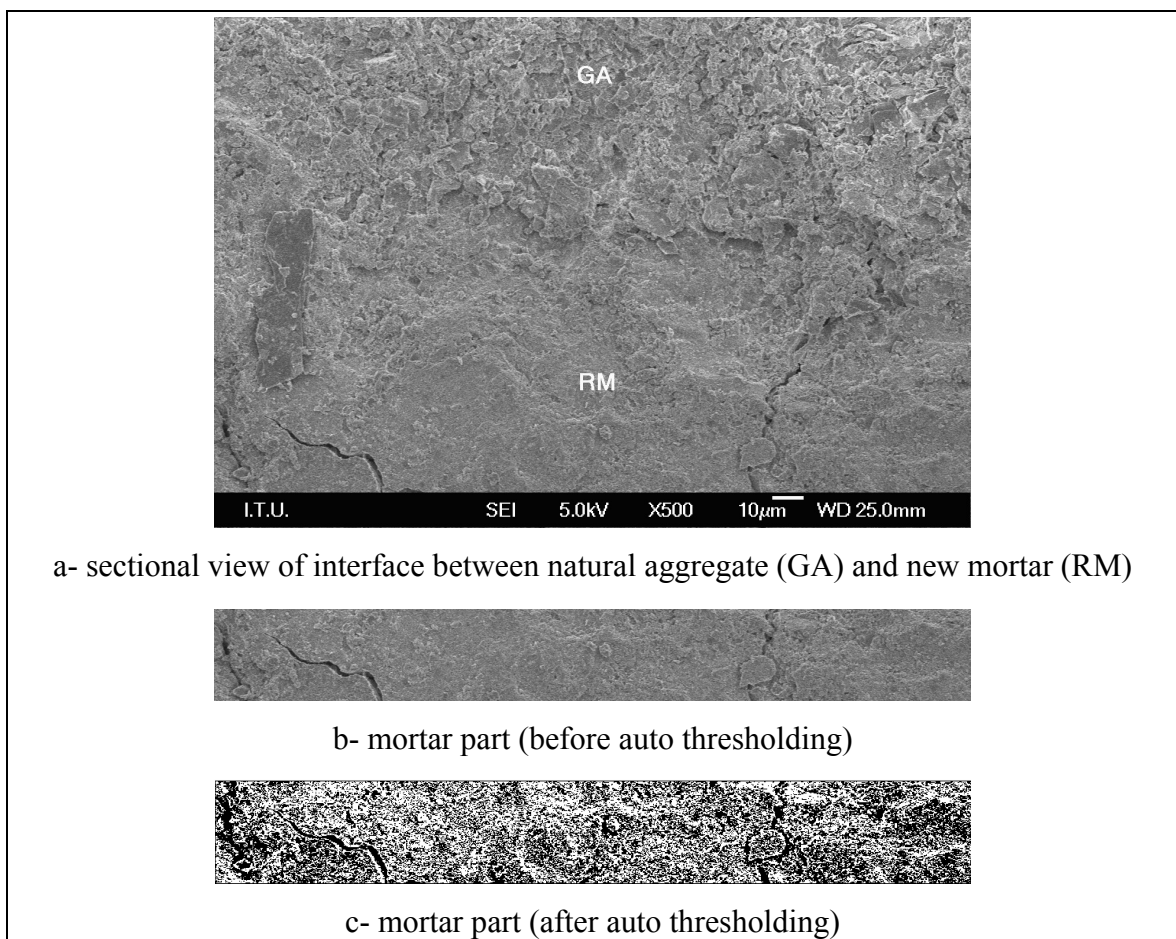


Figure 5.120. Image analysis procedure

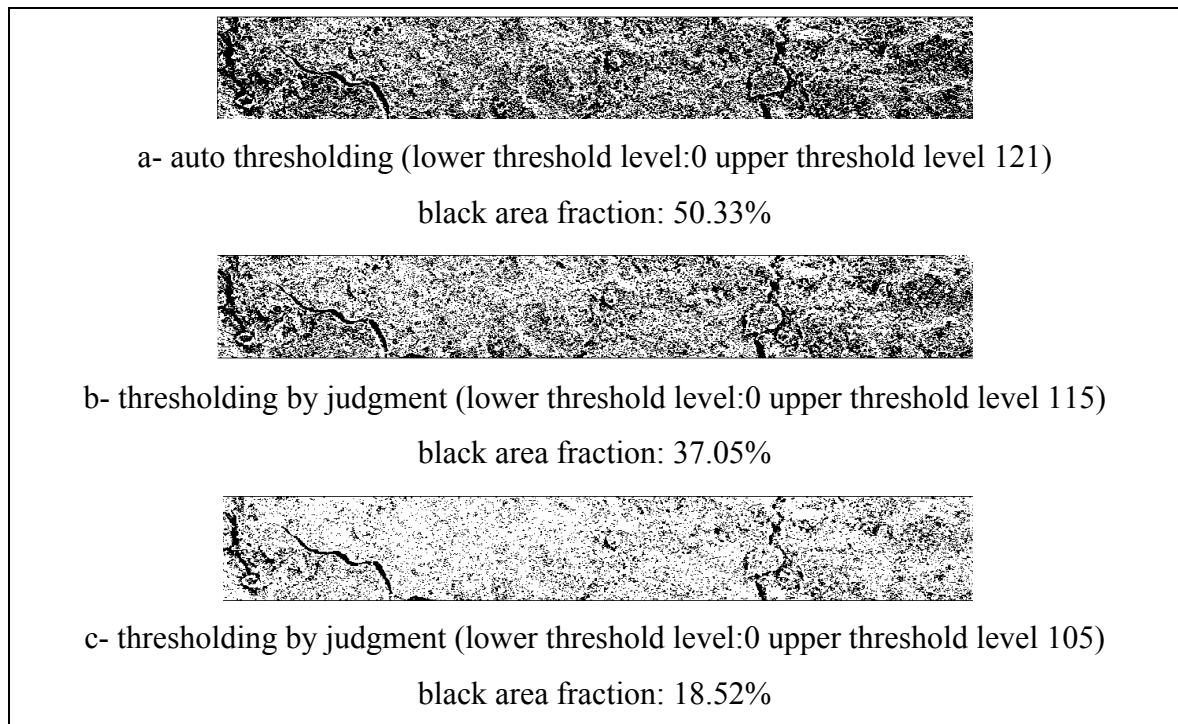


Figure 5.121. Examples of thresholding

The porosity values in Table 5.47 were the values computed at each interface, so they did not return the porosity of the mixes. Evaluation of water/cement ratio, replacement ratio, temperature factors were possible if the data were entered to the Design Expert software. In order to calculate the porosity of the mix taking into account the interface porosity values simple assumptions were made based on the Figure 5.122.

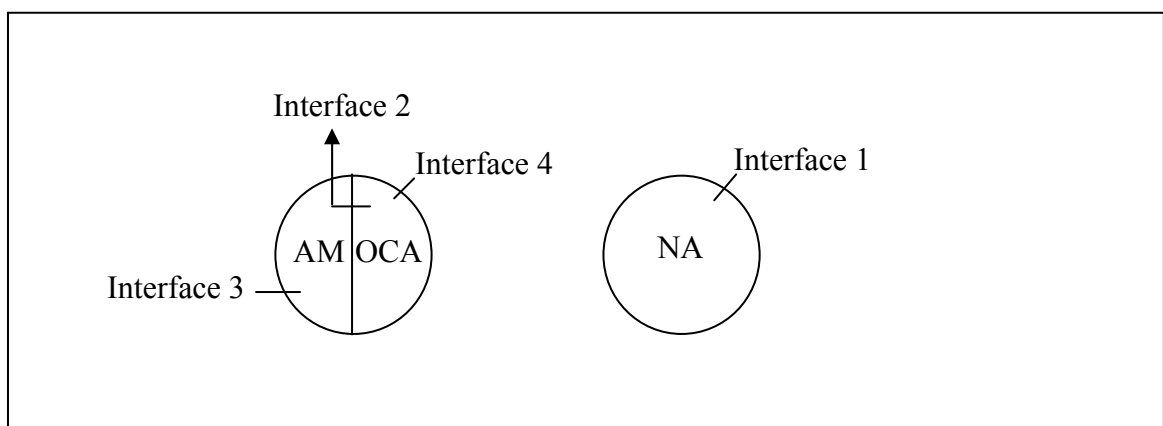


Figure 5.122. Interfaces in recycled aggregate concrete (replacement ratio <100 %)

Table 5.47. Porosity of the mortars at interfaces

Run	w/c	RR (%)	T (°C)	Age (days)	New mortar			Adhered mortar	
					Interface 1	Interface 3	Interface 4	Interface 2	Interface 3
1	0.35	0	22	90	35.66	-	-	-	-
2-3	0.35	0	105	28	57.12	-	-	-	-
4	0.35	0	175	28	61.50	-	-	-	-
5	0.35	0	250	28	51.91	-	-	-	-
6	0.35	50	22	28	50.33	56.73	56.63	58.71	57.06
7	0.35	50	105	90	57.68	53.23	53.32	59.08	56.60
8	0.35	100	22	90	-	63.00	56.12	55.70	56.66
9	0.35	100	105	28	-	56.96	57.81	55.37	56.80
10	0.35	100	175	90	-	61.61	57.70	59.56	61.34
11	0.35	100	250	90	-	54.82	60.24	57.29	57.25
12	0.4125	25	250	90	56.08	56.27	55.79	60.09	57.55
13-14	0.475	0	175	90	39.37	-	-	-	-
15	0.475	25	22	28	56.12	56.77	62.12	58.14	56.88
16	0.475	50	250	28	63.33	-	56.93	60.36	-
17	0.475	100	105	90	-	58.31	59.79	52.64	54.07
18	0.475	100	175	28	-	59.68	48.98	57.68	56.66
19	0.5375	25	175	28	52.39	56.73	57.16	54.17	53.72
20	0.5375	50	22	90	47.75	59.74	66.18	57.69	54.99
21	0.5375	75	250	90	59.65	53.78	58.66	60.03	63.20
22	0.60	0	22	28	51.23	-	-	-	-
23-24	0.60	0	105	90	49.83	-	-	-	-
25	0.60	0	250	90	49.48	-	-	-	-
26-27	0.60	50	105	28	55.79	55.04	57.70	56.15	56.06
28	0.60	100	22	28	-	57.95	59.92	59.22	61.17
29-30	0.60	100	175	90	-	54.11	57.28	53.12	56.97
31	0.60	100	250	28	-	57.11	59.15	64.22	58.52

As seen from Figure 5.122, natural aggregate (NA) and recycled aggregate (AM + OCA) were assumed circular shape. If the shape is circular then the ratio of length of the interfaces 1, 2, 3, and 4 would be 1:0.32:0.5:0.5, if the aggregate shape is square then the ratio is 1:0.25:0.50:0.50, averaging the values of circular and square shaped gave the shape factor as 1:0.28:0.50:0.50. Thus, the porosity could be calculated with these coefficients as in the Equation 5.35-5.39.

$$Porosity = P_{interface1}, \text{ natural aggregate concrete (RR= 0\%)} \quad (5.35)$$

$$Porosity = (3 * P_{interface1} + 0.28 * P_{interface2} + 0.5 * P_{interface3} + 0.5 * P_{interface4}) / 4$$

recycled aggregate concrete (RR=25%)

(5.36)

$$Porosity = (1 * P_{interface1} + 0.28 * P_{interface2} + 0.5 * P_{interface3} + 0.5 * P_{interface4}) / 2,$$

recycled aggregate concrete (RR=50%)

(5.37)

$$Porosity = (1 * P_{interface1} + 3 * (0.28 * P_{interface2} + 0.5 * P_{interface3} + 0.5 * P_{interface4})) / 4$$

recycled aggregate concrete (RR=75%)

(5.38)

$$Porosity = 0.28 * P_{interface2} + 0.5 * P_{interface3} + 0.5 * P_{interface4}$$

recycled aggregate concrete (RR=100%)

(5.39)

where,

$P_{interface1}$, $P_{interface2}$, $P_{interface4}$ = computed porosity values at interfaces 1, 2 and 4

$P_{interface3}$ = averages of interface 3 computed at adhered mortar and new mortar side

Computation of the mortar porosity values based on the assumptions yielded the values that are listed in Table 5.48. The mortar porosity values of image analysis results are demonstrated in Figures 5.123-5.130. 2FI model was used for analysis of porosity values. The standard deviation and the R-squared values of the model were 3.42 and 0.96. The R-squared value of 0.96 showed that 2FI model was quite good for the porosity prediction. The results of the ANOVA for response surface 2FI model are presented in Table 5.49. As seen from ANOVA table the model terms B, C, D, and BD are significant, because the Prob>F values are less than 0.05. This means that the replacement ratio (B), temperature (C), and age (D) have important effects on porosity. Durability tests in macro

level also have shown that the recycled aggregate concretes were more porous than natural aggregate concretes; and tested concretes were affected significantly with increased temperatures.

Studies done with SEM micrographs to evaluate the porosity with image analysis were neither practical nor economical, since these tests need more time, technology and money. Besides, according to ANOVA results, it seems that there was no effect of w/c ratio on mortar porosity, which was contrary to the fact that the porosity is affected by w/c ratio very much. Image analysis depended on many things. For instance, the smallest pore size measurable was a function of the pixel size of the image, which in turn is a function of the resolution and magnification used to capture the micrograph. Secondly, using the two dimensional micrograph did not allow the differentiation between accessible pores and inaccessible pores. Thirdly, when the test region selected from mortar part of the micrograph was changed to a different location, the measured property, in our case porosity, might be different from the previous one, because the porosity determined from each section represented the porosity of that small region. Despite these mentioned complications, SEM micrographs were helpful to observe the cracks and crack patterns and to measure the porosity. To conclude, microstructural studies showed that the differences inside the mixes, which finally affected the mechanical and durability properties.

Table 5.48. Porosity of the mortars of mixes 1-31

Run	w/c	RR (%)	T (°C)	Age (days)	Porosity
1	0.35	0	22	90	35.66
2-3	0.35	0	105	28	57.12
4	0.35	0	175	28	61.50
5	0.35	0	250	28	51.91
6	0.35	50	22	28	61.77
7	0.35	50	105	90	64.17
8	0.35	100	22	90	73.57
9	0.35	100	105	28	72.85
10	0.35	100	175	90	76.26
11	0.35	100	250	90	74.18
12	0.4125	25	250	90	60.35
13-14	0.475	0	175	90	39.37
15	0.475	25	22	28	61.03
16	0.475	50	250	28	69.44
17	0.475	100	105	90	72.73
18	0.475	100	175	28	69.73
19	0.5375	25	175	28	57.13
20	0.5375	50	22	90	62.84
21	0.5375	75	250	90	71.45
22	0.60	0	22	28	51.23
23-24	0.60	0	105	90	49.83
25	0.60	0	250	90	49.48
26-27	0.60	50	105	28	64.07
28	0.60	100	22	28	76.32
29-30	0.60	100	175	90	71.28
31	0.60	100	250	28	76.46

Table 5.49. Results of ANOVA for mortar porosity response surface 2FI model

Source	Sum of squares	Degrees of freedom	Mean square	F value	Prob > F
A	1.11	1	1.11	0.09	0.7637
B	3093.73	1	3093.73	264.17	<0.0001
C	123.29	3	41.10	3.51	0.0493
D	102.21	1	102.21	8.73	0.0120
AB	1.08	1	1.08	0.09	0.7668
AC	98.44	3	32.81	2.80	0.0852
AD	6.96	1	6.96	0.59	0.4556
BC	60.46	3	20.15	1.72	0.2157
BD	172.78	1	172.78	14.75	0.0023
CD	28.42	3	9.47	0.81	0.5129

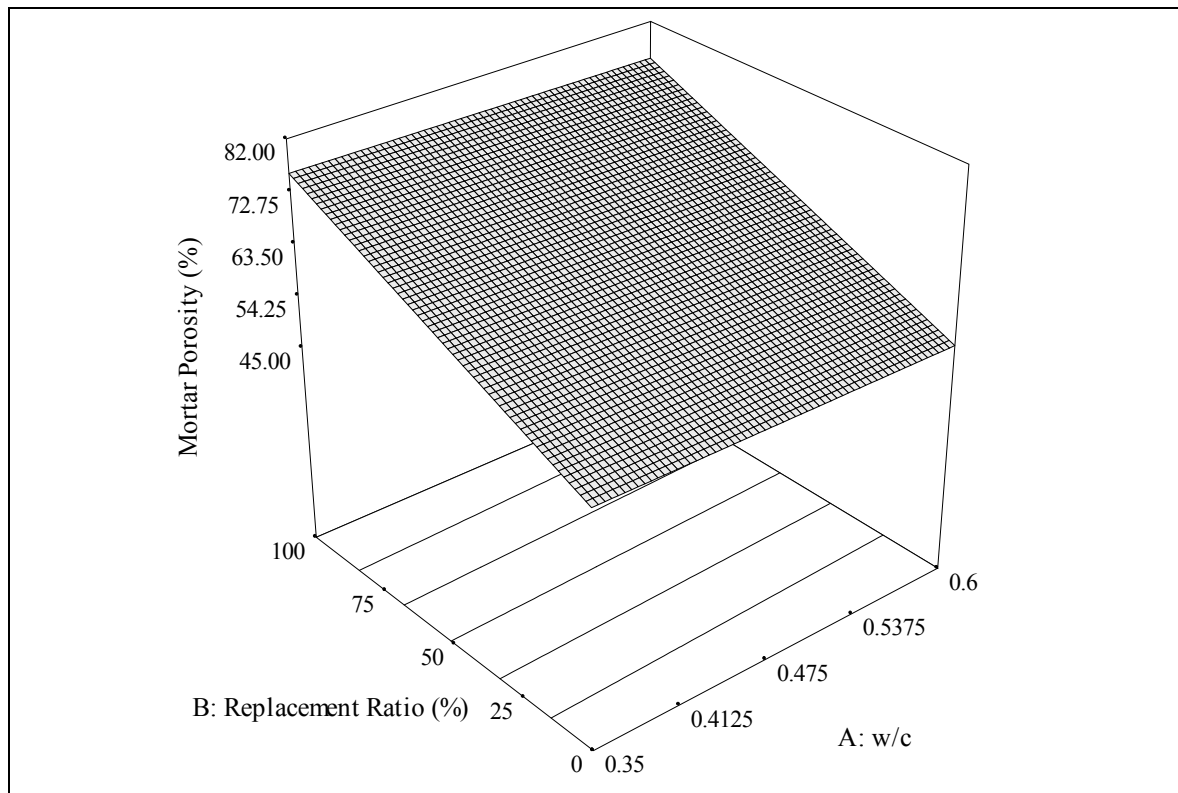


Figure 5.123. 3D surface view of mortar porosity (age: 28 days, temperature: 22 °C)

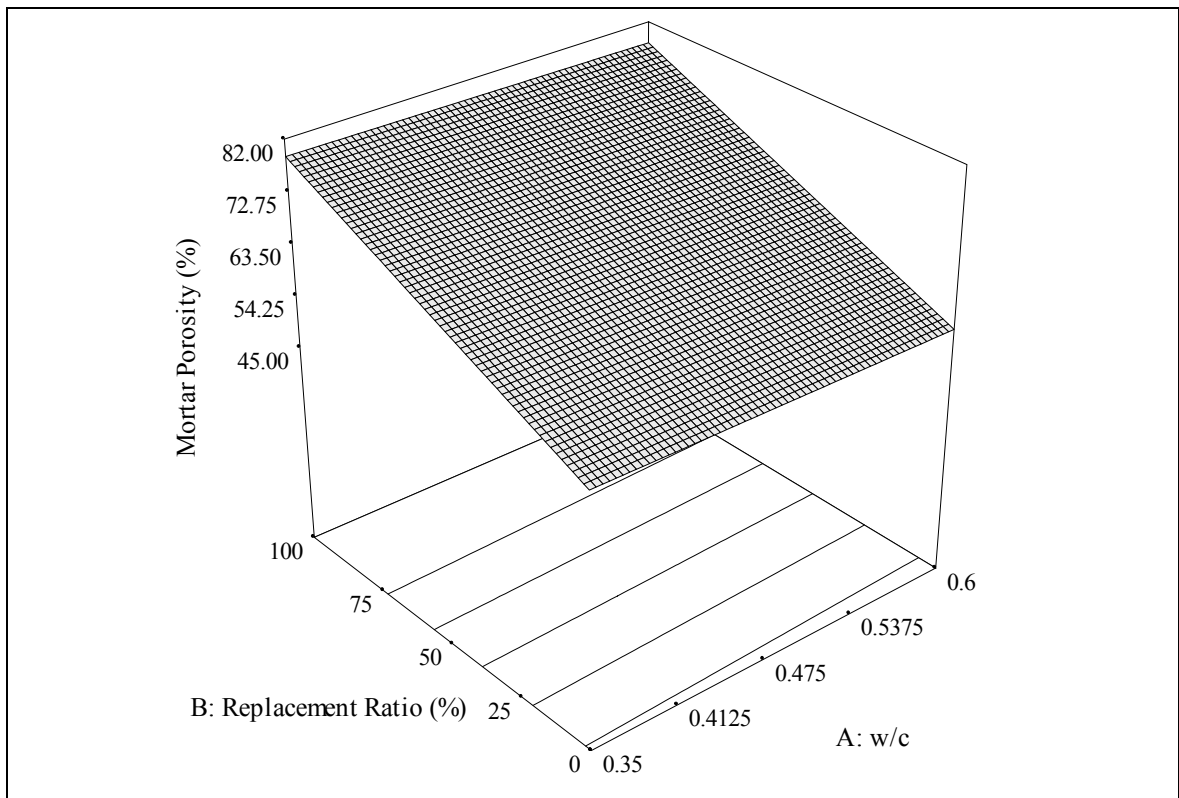


Figure 5.124. 3D surface view of mortar porosity (age: 28 days, temperature: 105 °C)

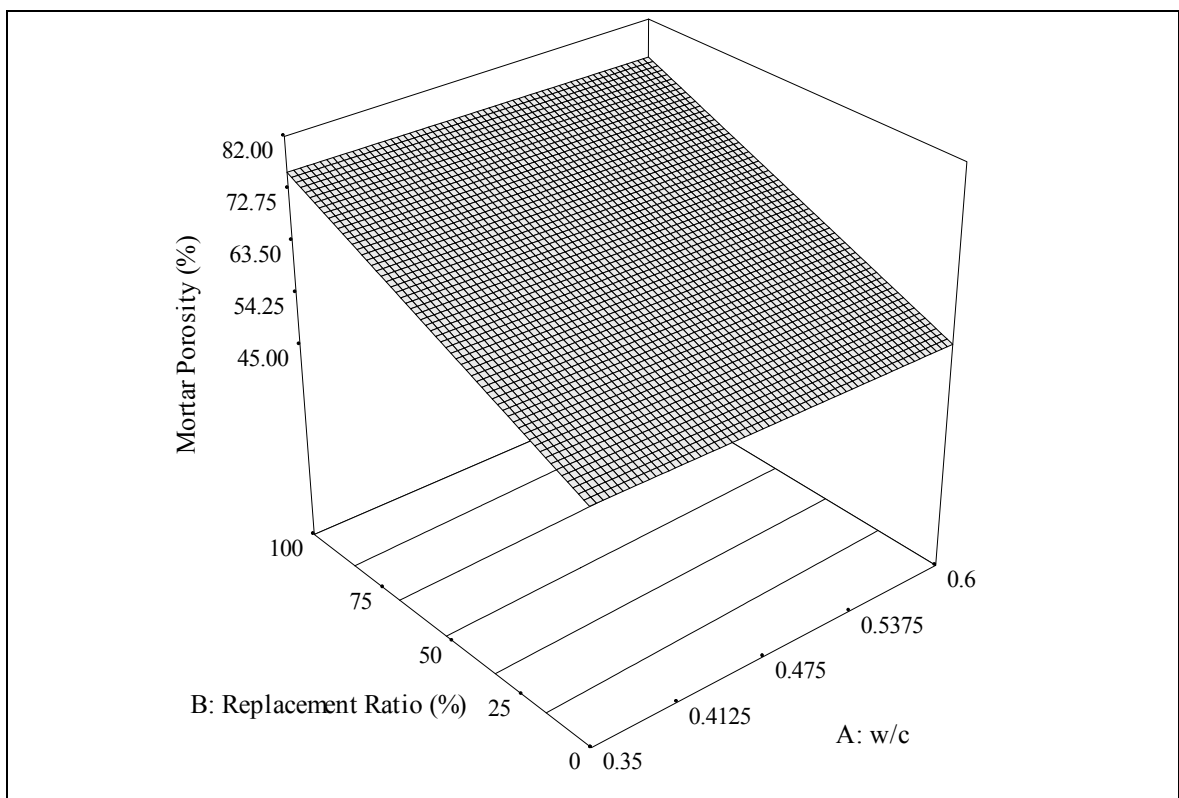


Figure 5.125. 3D surface view of mortar porosity (age: 28 days, temperature: 175 °C)

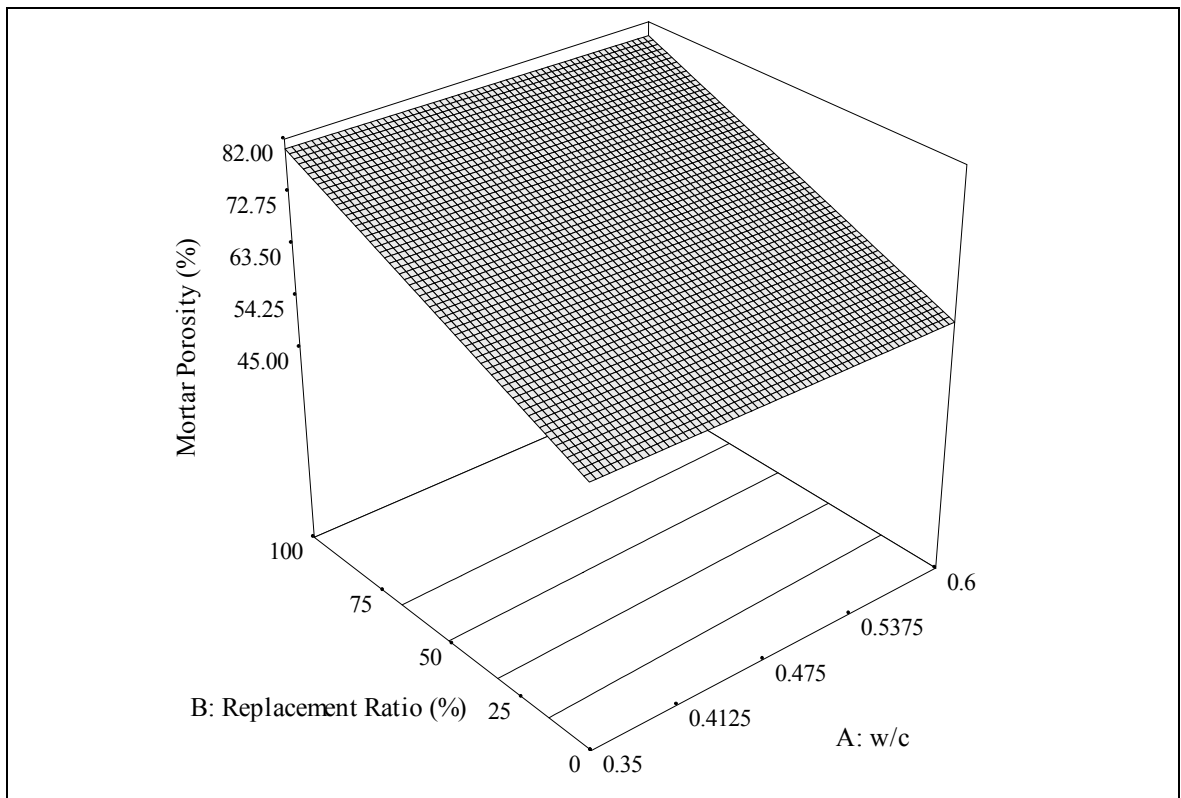


Figure 5.126. 3D surface view of mortar porosity (age: 28 days, temperature: 250 °C)

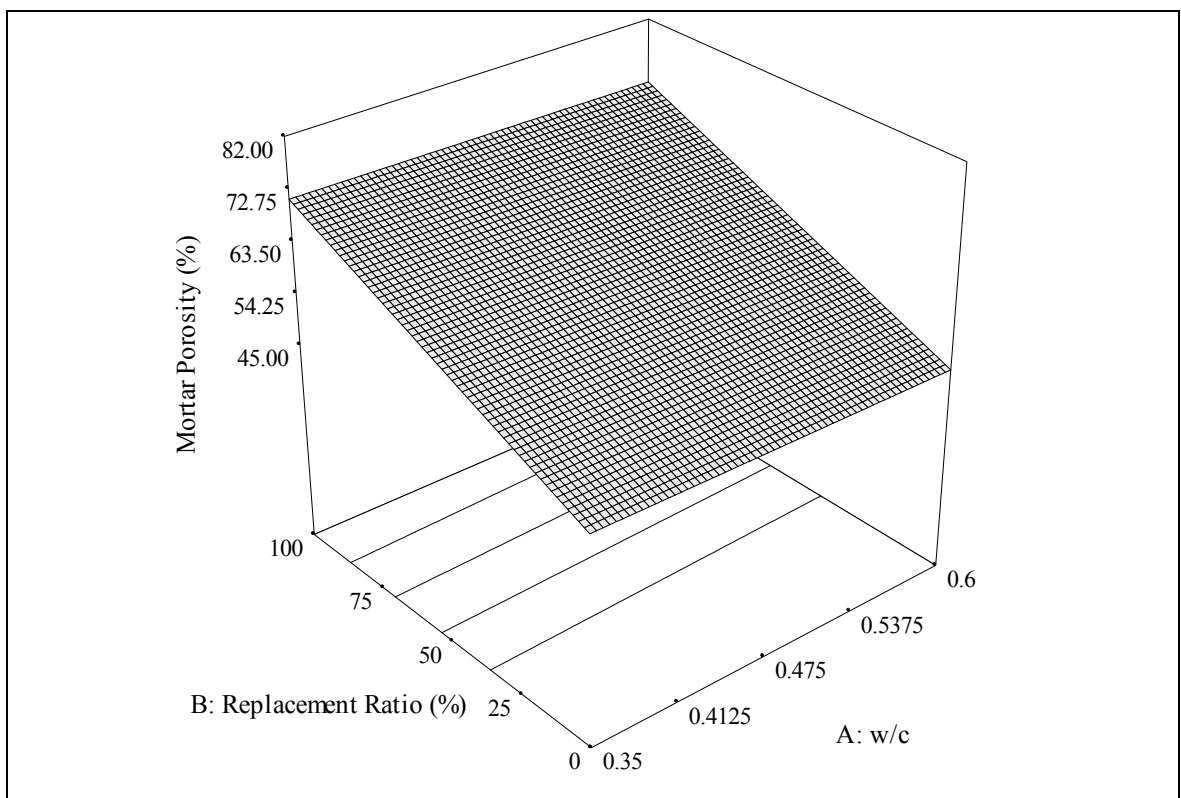


Figure 5.127. 3D surface view of mortar porosity (age: 90 days, temperature: 22 °C)

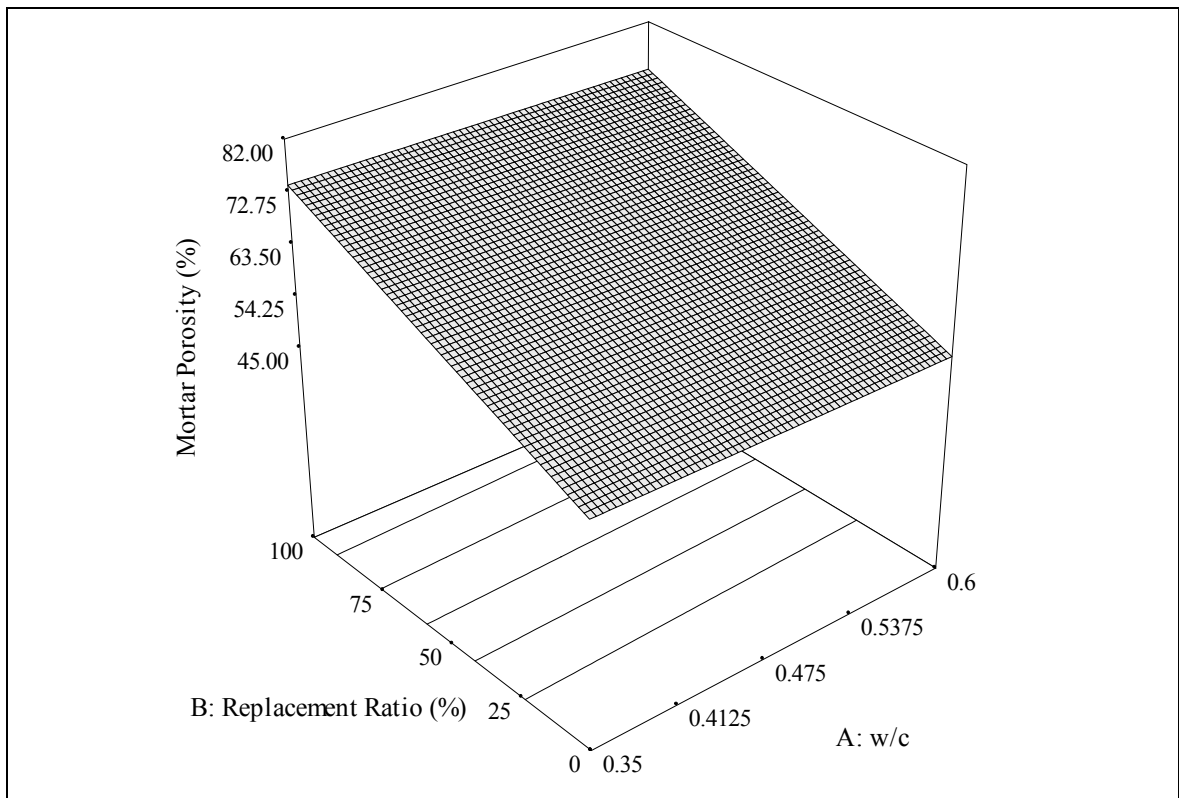


Figure 5.128. 3D surface view of mortar porosity (age: 90 days, temperature: 105 °C)

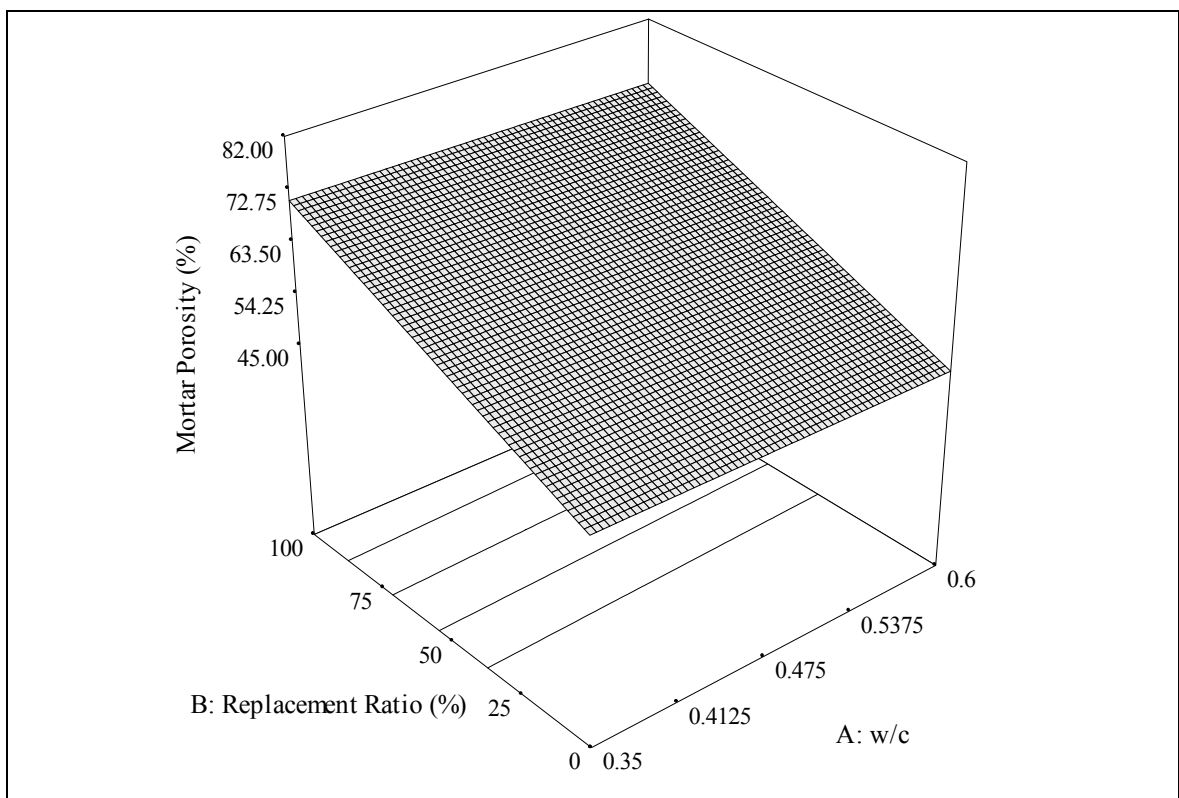


Figure 5.129. 3D surface view of mortar porosity (age: 90 days, temperature: 175 °C)

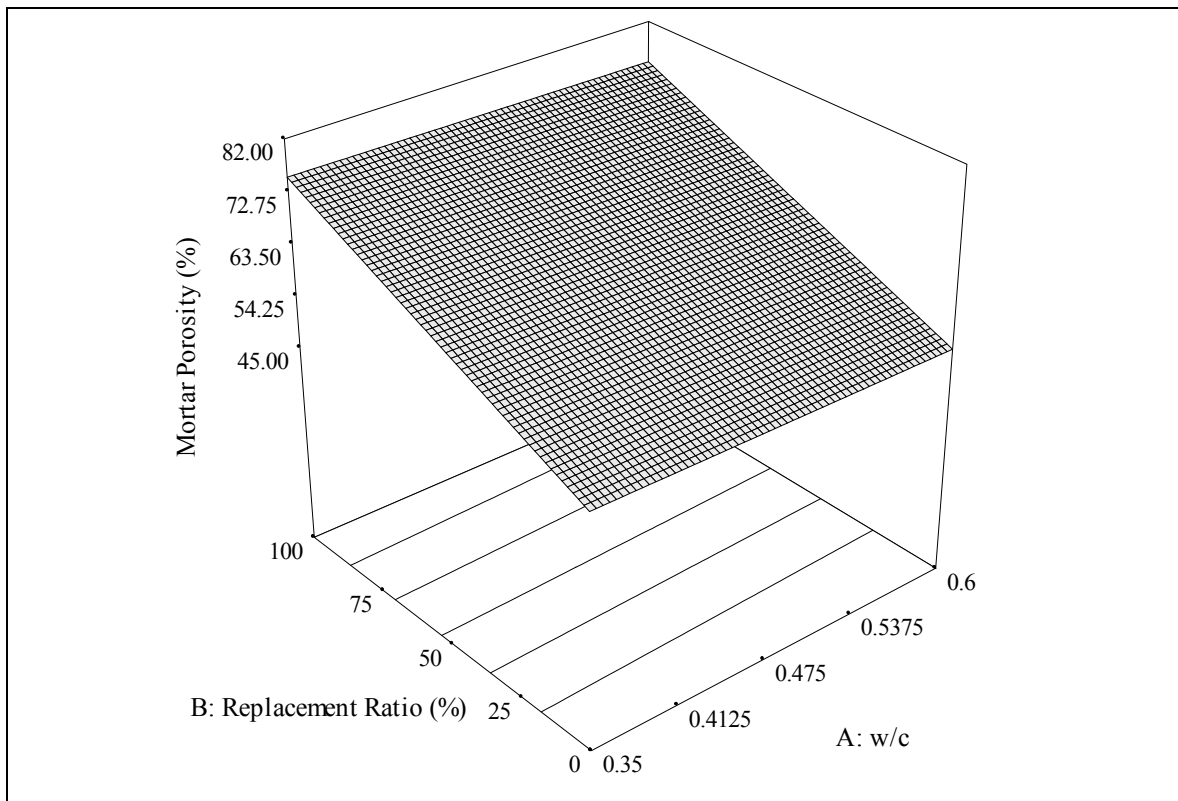


Figure 5.130. 3D surface view of mortar porosity (age: 90 days, temperature: 250 °C)

5.9. Evaluation of the Objective of the Study

The objective of this doctoral study was to analyze the suitability of recycled aggregate concrete at elevated temperatures. For this purpose natural and recycled aggregate concretes were tested for mechanical and durability properties. Measurements, observations and analysis were also performed in micro level to better understand the changes in microstructure with varying conditions. Compressive strength, static modulus of elasticity, splitting tensile strength, water absorption, chloride permeability, sorptivity values in macro level and crack observations in micro level and porosity values obtained from both MIP and image analysis were all depended on the water/cement ratio, replacement ratio, exposed temperature degree and age factors. The amount of gain or loss in the measured test or analysis was depending on the factors of water/cement ratio, replacement ratio, exposed temperature degree, and age. Test results confirmed that including recycled aggregates into the concrete affected the properties of concrete, but the degree of effect was not same for the evaluated parts. So in order to decide whether the

recycled aggregate concrete at elevated temperatures is suitable or not the properties of the natural aggregate concretes were compared with recycled aggregate concrete produced with 10 percent replacement ratio as suggested by RILEM [17]. The values in Table 5.50 showing the comparison of natural and recycled aggregate concretes found by the aid of the “point prediction” module in the Design Expert software.

The values shown in Table 5.50 give confidence to use recycled aggregates in concrete with a replacement ratio of 10 percent up to 250 °C exposure condition. Among the mechanical properties the maximum decrease was observed in splitting tensile strength with 4.5 percent (at $w/c=0.60$, $T=175\text{ }^{\circ}\text{C}$). This decrease was insignificant. Among the durability properties the greatest percentage difference was observed in rapid chloride permeability test (at $w/c=0.35$, $T=175\text{ }^{\circ}\text{C}$). The values of RCPT charge in coulombs were 4466 and 5213 for natural and recycled aggregate concrete with 10 percent replacement ratio, respectively. Although the differences seemed high, according to ASTM evaluation both concretes were high category permeability rating.

Hence it can be concluded that concretes that will be exposed to elevated temperatures up to 250 °C could be produced safely with 10 percent recycled aggregate replacement by following the same precautions taken when natural aggregate concretes used.

Table 5.50. Comparison of natural aggregate concrete (NAC) and recycled aggregate concrete (RAC) properties at varying situations (Replacement ratio = 10%)

Measured property / conditions		22 °C		105 °C		175 °C		250 °C	
		NAC	RAC	NAC	RAC	NAC	RAC	NAC	RAC
w/c= 0.35 & age= 28days	Compressive Strength (MPa)	38.22	38.22	33.02	33.35	43.08	42.88	52.21	51.31
	Static Modulus of Elasticity (GPa)	17.74	18.18	20.77	20.81	21.30	21.24	23.54	23.10
	Splitting Tensile Strength (MPa)	3.90	3.81	3.95	3.89	4.09	3.93	5.06	4.86
	RCPT Charge (Coulombs)	4917	5262	2694	2881	4466	5213	7798	8395
	Water Absorption (%)	2.78	3.21	3.56	3.91	4.36	4.77	5.57	6.05
	Log ₁₀ (Sorptivity. Si (mm/sqrt(sec)))	-3.55	-3.53	-3.18	-3.16	-2.60	-2.57	-2.43	-2.41
	Mean Distribution Diameter (nm)	20.98	20.97	13.39	13.38	14.70	14.69	23.22	23.21
	Mortar Porosity (%)	51.38	53.85	54.32	56.79	51.14	53.61	55.64	58.11
w/c= 0.60 & age= 28days	Compressive Strength (MPa)	23.23	23.56	20.37	21.02	23.40	23.54	31.11	30.55
	Static Modulus of Elasticity (GPa)	12.01	12.46	17.42	17.47	11.56	11.51	12.95	12.53
	Splitting Tensile Strength (MPa)	2.86	2.82	2.63	2.61	2.39	2.28	3.69	3.52
	RCPT Charge (Coulombs)	10430	10772	10368	10550	14878	15621	16276	16868
	Water Absorption (%)	7.19	7.48	8.12	8.32	8.50	8.77	8.33	8.67
	Log ₁₀ (Sorptivity. Si (mm/sqrt(sec)))	-2.90	-2.88	-2.53	-2.50	-1.94	-1.92	-1.78	-1.75
	Mean Distribution Diameter (nm)	19.22	19.21	11.63	11.62	12.94	12.93	21.46	21.45
	Mortar Porosity (%)	50.51	52.98	53.44	55.92	50.27	52.74	54.76	57.24

6. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORKS

6.1. Conclusions

In accordance with the experimental works carried out in this thesis, the following conclusions are drawn:

- Due to the adhered mortar, the specific gravity of recycled aggregates was lower than that of natural aggregates, which source concrete was made out of.
- The water absorption capacity of recycled aggregates was higher than that of limestone aggregates because of the adhered mortar. This absorption capacity of the recycled aggregates must be taken into account during concrete production to prevent slump loss.
- It was possible to replace natural coarse aggregates by recycled coarse aggregates in concrete without any problem in concrete preparation. Standard mix design method was used in recycled aggregate concretes without any modification. The only difference in production of concrete with recycled aggregates was the immersing process of the recycled aggregates before mixing. The natural coarse aggregate could be totally replaced by recycled coarse aggregates.
- The use of recycled coarse aggregates at saturated surface dry state in concrete prevented slump loss, thus the recycled aggregate concrete was achieved with the same workability as natural aggregate concrete.
- The replacement of recycled aggregate into concrete decreased the fresh unit weight of concrete as compared with natural aggregate concrete. The lower density was the result of lower density of the recycled aggregates due to adhered mortar.
- The density of the concretes decreased with an increase in the water/cement ratio, as expected.

- Quality of the source concrete, from which the recycled aggregates were gained, did not change significantly the recycled aggregate concrete strength. It is possible to prepare recycled aggregate concretes having 32 MPa compressive strength from the recycled aggregates made from 24 MPa strength source concrete.
- Compressive strength, static modulus of elasticity and splitting tensile strength were affected mostly from the water/cement ratio of the mixes. Among the factors w/c ratio, replacement ratio, exposure temperature, and age the most significant factor was w/c ratio. Second factor affecting the mechanical properties was the temperature increases. Each temperature range indicated a different pattern of strength increase or decrease. Lowering the w/c ratio was highly effective on the strength development. Mechanical properties increased as the age of the concrete increased, as expected.
- Recycled aggregate content affected the mechanical properties of concrete. The degree of effect depends on the exposure temperature. The effect of replacement ratio factor on compressive strength and static modulus of elasticity was lower than the water/cement ratio and temperature factors. Recycled aggregate replacement showed its significant influence on splitting tensile strength. However, the reduction could be compensated by the use of a lower water/cement ratio.
- The replacement of natural coarse aggregates with recycled coarse aggregates was found to be viable to the increase of the static modulus of elasticity. However, depending on the exposed temperature range the recycled aggregate concretes lost their static modulus of elasticity in a great percentage. Recycled aggregate concretes were affected more than natural aggregate concretes under elevated temperatures. Reductions of the static modulus of elasticity in the recycled aggregate concretes were not considered to be a problem. Lowering the water/cement ratio and replacement ratio to a certain degree were highly beneficial to the increase in static modulus of elasticity value of recycled aggregate concretes.
- Observed cracks in the microstructural study were the evidence for splitting tensile strength loss. The tensile strength compared to the compressive strength was more sensitive to cracks which were caused by thermal incompatibility observed under high exposure temperatures. Incorporation of recycled aggregates generally lowered the

splitting tensile strength of concretes because of the cracks occurred during crushing process of source concretes. Lowering the water/cement ratio and replacement ratio would balance the negative effect of recycled aggregates.

- Recycled aggregate content affected the durability properties more than mechanical properties. The use of recycled aggregate with high absorption capacity in the recycled aggregate concrete resulted in a reduction in chloride ion penetration resistance, and increase in water absorption capacity and sorptivity.
- Higher water absorption values were recorded on concrete specimens produced with higher water/cement ratio and having higher recycled aggregate content. Early ages and elevated temperatures were the other factors causing the high absorption.
- The water absorption capacity of the recycled aggregate concrete was found to be always higher than that of natural aggregate concrete because of the presence of adhered mortar in recycled aggregate. Secondly more interfacial transition zones were involved in the system when recycled aggregates were used.
- Cracks either induced by thermal incompatibility or crushing process increased the water absorption capacity of the concretes and reduced the chloride ion penetration resistance.
- The resistance to chloride ion penetration decreased as the recycled aggregate content, water/cement ratio, and exposure temperature increased. Lowering the w/c ratio and replacement ratio solved the chloride ion penetration problem to an extent.
- The adhered mortar had an important role in determining the mechanical and durability properties. The adhered mortar content of the recycled aggregate was determined by heating method, in other words thermal shock method. A simple relationship was also developed to determine the adhered mortar content based on the density of the recycled aggregate, adhered mortar, and original coarse aggregate.
- Generally, the proposed relationships between the concrete properties were not suitable to predict one property from another.
- The mercury intrusion porosimetry (MIP) test results showed that the porosity depended on the factors of temperature, water/cement ratio and replacement ratio. The analysis of MIP revealed that the differences in pore structure caused different mechanical and durability properties.

- The microstructural studies on interfaces gave very important results in the explanation of the temperature factor and replacement ratio factor. Especially the cracks observed on interfaces between adhered mortar and original coarse aggregate were the reasons of the weak bonds affecting the mechanical properties.
- The microstructural observations showed that the interfacial transition zones were weaker than the pastes. It was found that the weakest link of the chain was the interfacial transition zone, and the weakest component was the recycled aggregate having three interfaces.
- According to the image analysis results, porosity was mostly affected by replacement ratio, temperature and age factors. The porosity increased as the recycled aggregate content increased.
- The recycled aggregate concrete with 10 percent recycled aggregate content assured almost the same mechanical and durability properties of the natural aggregate concrete at all conditions. On the other hand, the numerical optimization technique showed that it is possible to produce recycled aggregate concrete with 30 percent recycled aggregate content, which will satisfy maximum mechanical properties and minimum permeability properties.
- Since there were no principal technical problems with the use of recycled aggregate in concrete, using recycled aggregates in concrete production will provide strong environmental benefits.

6.2. Recommendations for Future Works

The study carried out in this experimental work dealt only with the recycled aggregates gained from source concretes produced at laboratory or ready mix concrete plant. It is needed to investigate other recycled aggregates gained from real demolished structures. Shape and texture, grading, absorption, mineralogy and coatings, strength and stiffness, specific gravity, soundness and toughness are the main characteristics of aggregate that affect the performance of fresh and hardened concrete. Since there is not any standard for recycled aggregates

limiting these values, preliminary tests should be done before using the recycled aggregates in structural concrete.

Recycled coarse aggregate fraction was used in this study. The fine portion of the recycled aggregates was not used. Research is needed to determine the application areas of these unused recycled fine aggregates.

The effect of elevated temperatures over 250 °C was not investigated. Due to the fire risk, recycled aggregate concretes need further investigation at high temperatures. The heating rate, duration of maximum exposure temperature, cooling rate, and specimen size will affect the results. More experimental work is needed to verify the structural behaviour of recycled aggregate concrete considering the concrete cover under high temperatures.

A better understanding of the actual reasons of cracks formed at the interfaces is desirable. The study of shrinkage of recycled aggregate concretes will be helpful to understand the underlying causes of cracks observed at the interfaces.

When preparing the small samples for MIP tests by grinder machine it was experienced that reducing the size of the small concrete piece was easier in concretes with lower strength and reducing the size needed more effort if the pieces were obtained from high strength concrete. This experience brings into mind that whether a semi destructive test machine could be manufactured to test the compressive strength. A preliminary design was drawn in Figure 6.1. The compressive strength and static modulus of elasticity will be a function of diameter (D) and the thickness of the saw, penetrated depth (H) and penetration time at each applied force. Further discussions are needed to construct the machine and derive the equations.

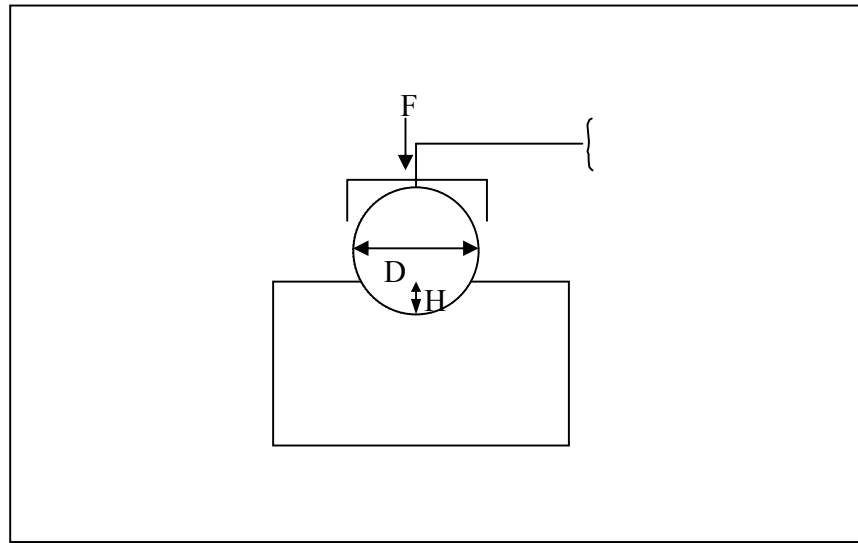


Figure 6.1. In situ compressive test machine

APPENDIX A: RECYCLED AGGREGATE PRODUCTION

Figure A.1. Recycled aggregate production stages

APPENDIX B: DETERMINATION OF ADHERED MORTAR CONTENT

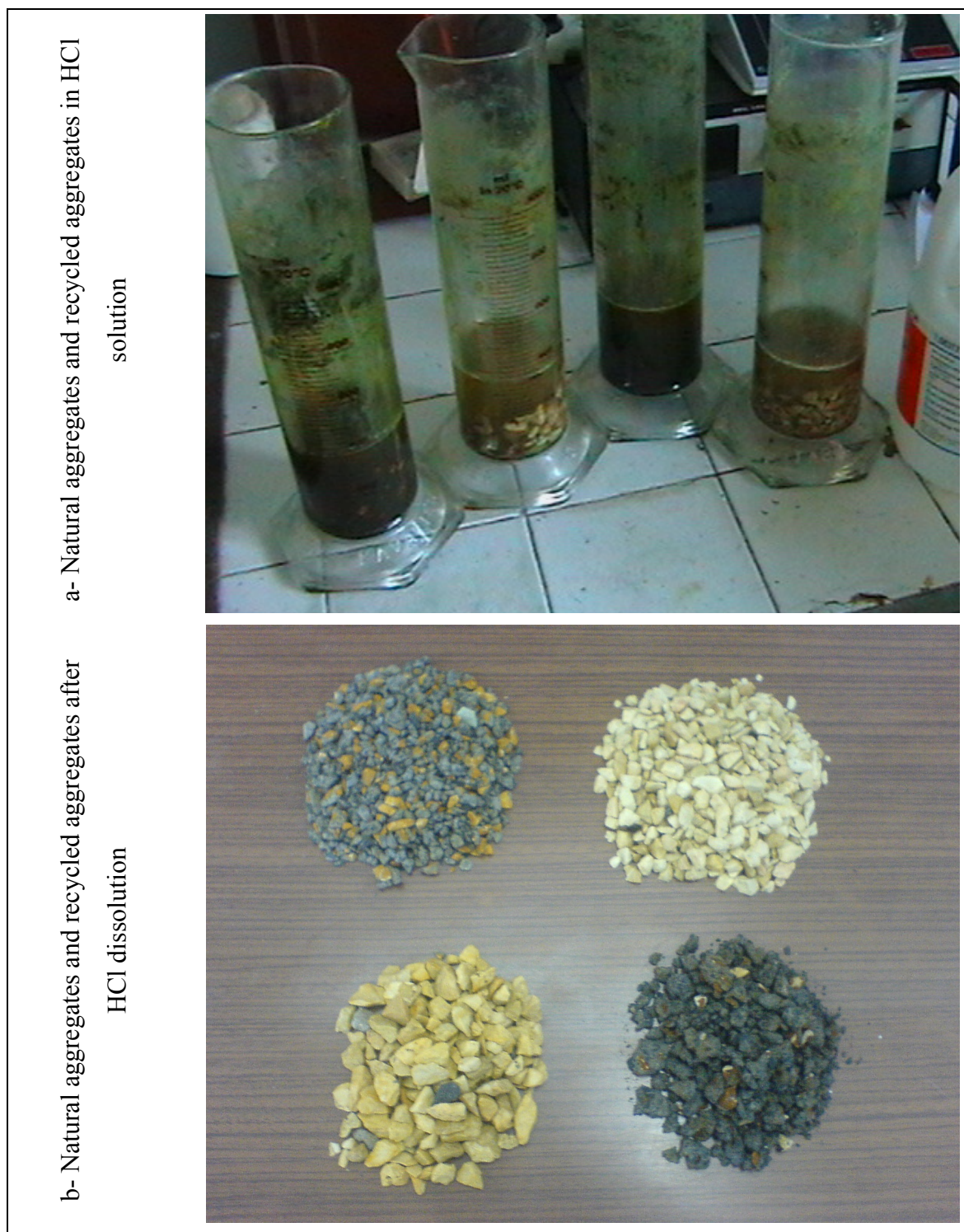


Figure B.1. Hydrochloric dissolution method

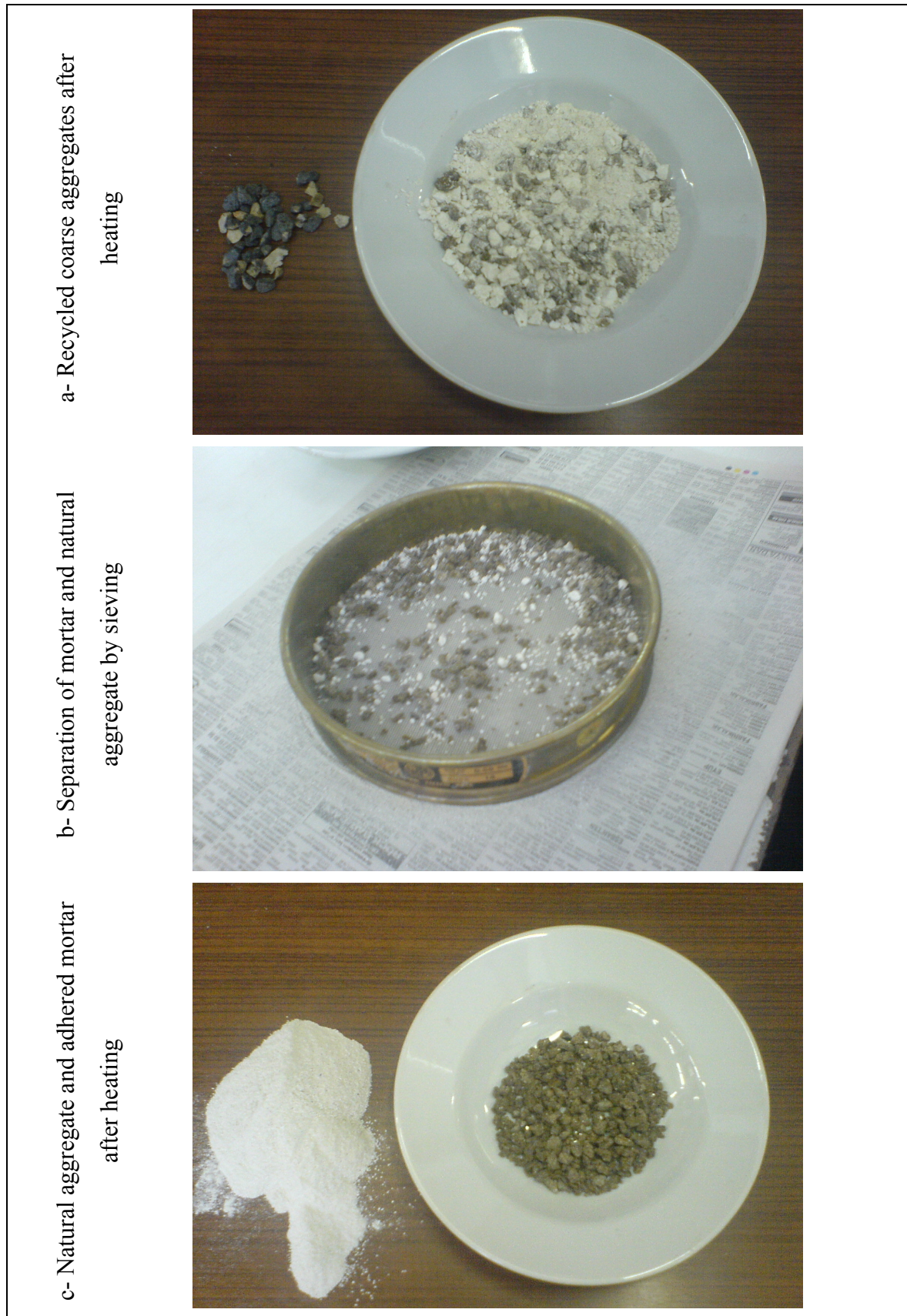


Figure B.2. Heating method

APPENDIX C: D OPTIMAL ALGORITHM

The algorithm used to find an approximately D optimal design is as follows:

1. Select a non-singular initial design of p points:

- Pick the candidate point with the largest Euclidean (the shortest straight line distance between two points in a design space) norm as the first design point. This first point will be one of the perimeter design points, i.e., one of the points furthest from the center of the design space.
- Pick each subsequent design point to give the maximum information about the estimable linear combinations of model coefficients and to increase the rank of the design matrix.

2. Select the remaining model points ($\#$ model points - p). Select the candidate point that adds the maximum amount of information about the model coefficients, i.e., the D optimal candidate point.

3. Perform exchange steps:

- A 1-point exchange step consists of adding to the current design the point in the candidate list that increases the determinant of the information matrix the most, and then deleting from the augmented design, the point that decreases the determinant of the information matrix the least. A 2-point exchange step adds two points in sequence, and then deletes two points. Each step is taken to increase the determinant of the information matrix the most, or decrease it the least. An n -point exchange step adds and deletes n points.
- Perform 1-point exchange steps until there is no improvement in the design. Then perform 2-point exchange steps, and so on until 5-point exchange steps show no improvement. If at any point, there is improvement, start over with 1-point exchanges.

APPENDIX D: PRODUCTION OF CONCRETES

Figure D.1. Production of concretes

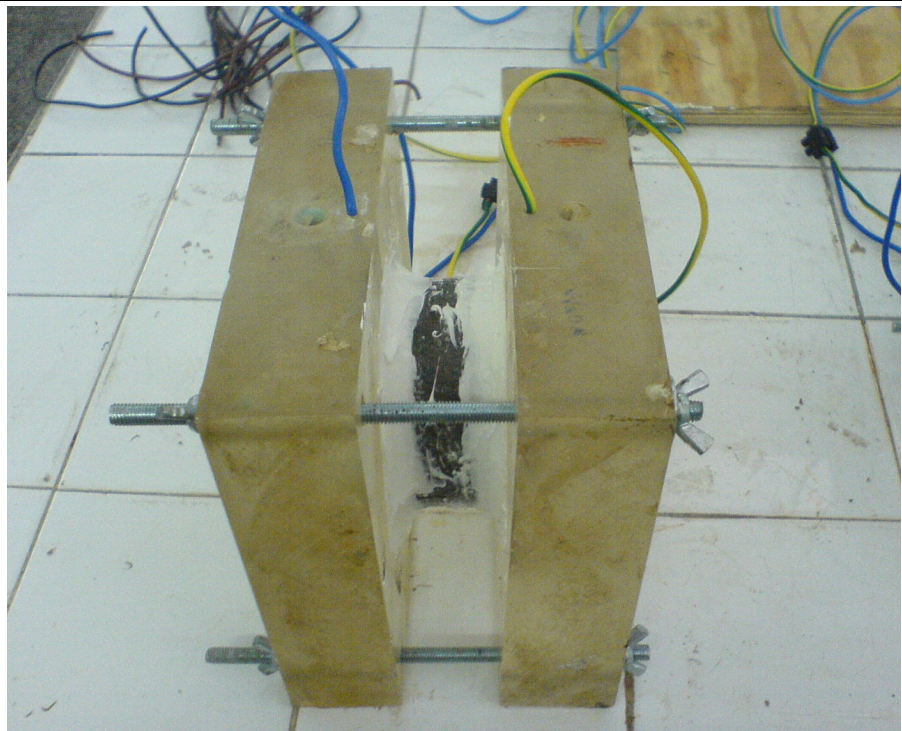
APPENDIX E: TESTS FOR DETERMINATION OF MECHANICAL PROPERTIES OF HARDENED CONCRETE



Figure E.1. Tests for determination of mechanical properties of hardened concrete

APPENDIX F: TESTS FOR DETERMINATION OF DURABILITY PERFORMANCE OF HARDENED CONCRETE

a- Clamped test cell



b- Rapid chloride permeability test setup

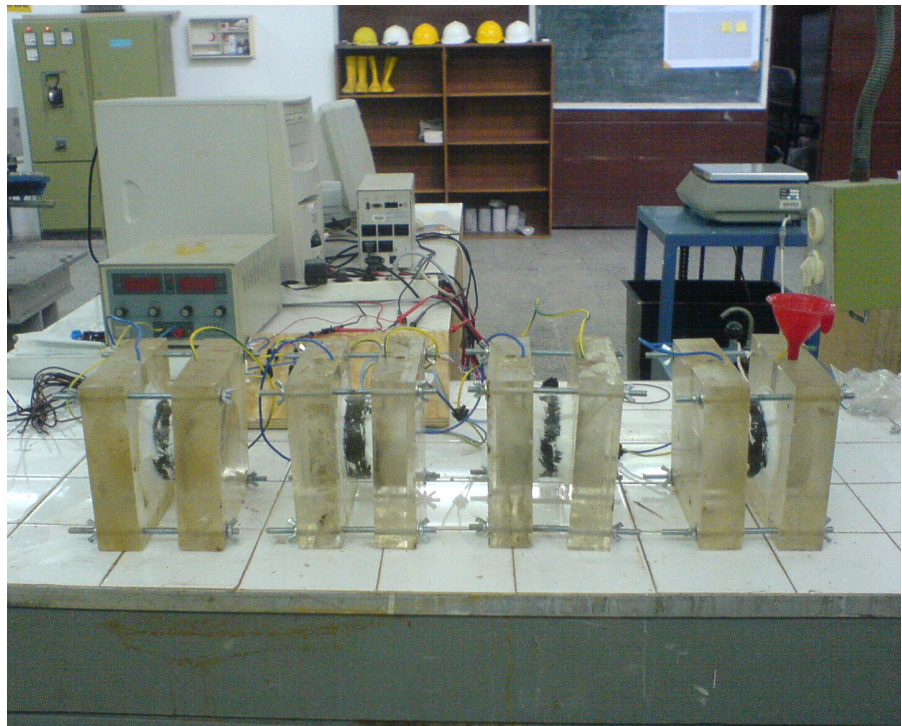


Figure F.1. Test for determination of chloride permeability of hardened concrete

APPENDIX G: COMPRESSIVE STRESS-STRAIN CURVES

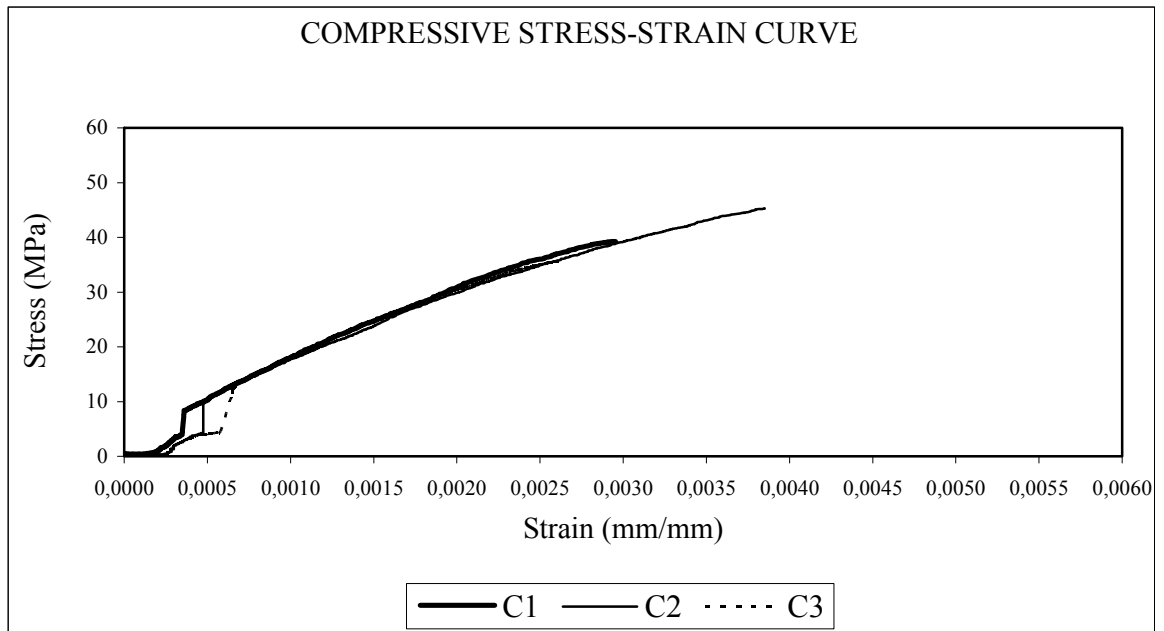


Figure G.1. Compressive stress-strain curve of concrete specimens of mix 1
(w/c: 0.35, RR: 0%, T:22 °C, Age: 90 days)

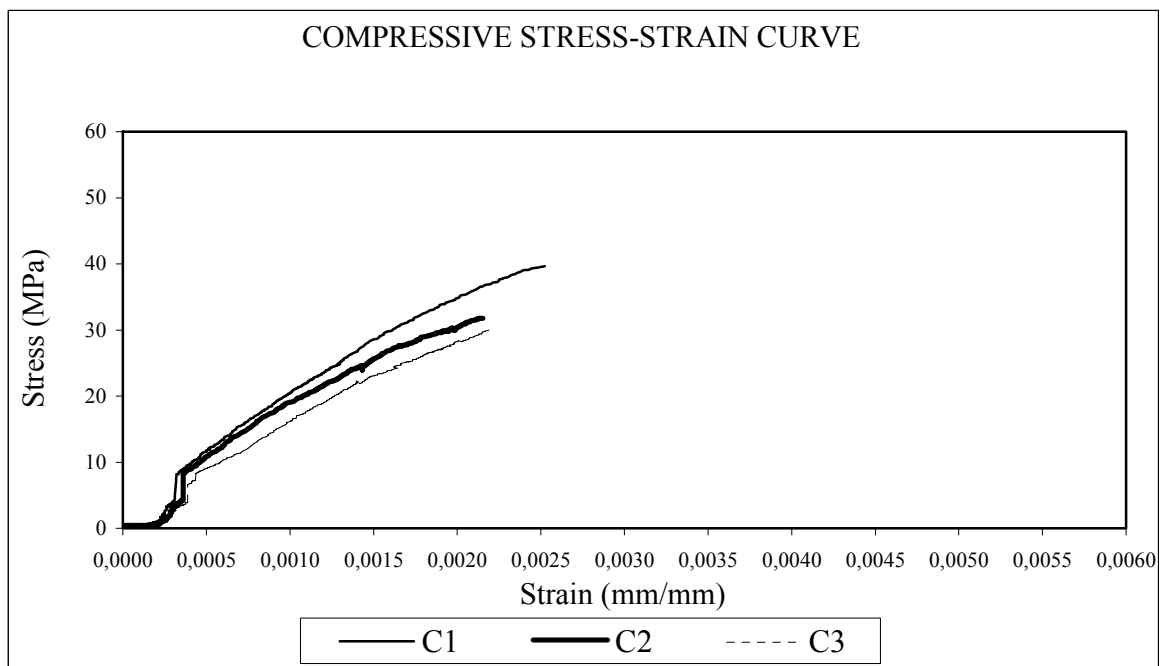


Figure G.2. Compressive stress-strain curve of concrete specimens of mix 2
(w/c: 0.35, RR: 0%, T:105 °C, Age: 28 days)

REFERENCES

1. Çelik, B., *Effect Of Binding Material Composition on Bond Strength of Concrete Subjected to Different Exposure Conditions*, M. S. Thesis, Boğaziçi University, 2001.
2. Güneyisi, E., M. Gesoğlu and T. Özturan, “Properties of Rubberized Concretes Containing Silica Fume”, *Cement and Concrete Research*, Vol. 34, No. 12, pp. 2309-2317, December 2004.
3. Papakonstantinou, C. G. and M. J. Tobolski, “Use of Waste Tire Steel Beads in Portland Cement Concrete”, *Cement and Concrete Research*, Article in Press, 2006.
4. Olivares, F. H., Barluenga, G., M. Bollati and B. Witoszek, “Static and Dynamic Behaviour of Recycled Tyre Rubber-Filled Concrete”, *Cement and Concrete Research*, Vol. 32, No. 10, pp. 1587-1596, October 2002.
5. Shayan, A. and A. Xu, “Value-Added Utilization of Waste Glass in Concrete”, *Cement and Concrete Research*, Vol. 34, No. 1, pp. 81-89, January 2004.
6. Park, S. B., B. C. Lee and J. H. Kim, “Studies on Mechanical Properties of Concrete Containing Waste Glass Aggregate”, *Cement and Concrete Research*, Vol. 34, No. 12, pp. 2181-2189, December 2004.
7. Soroushian, P., J. Plasencia and S. Ravanbakhsh, “Assessment of Reinforcing Effects of Recycled Plastic and Paper in Concrete”, *ACI Materials Journal*, Vol. 100, No. 3, pp. 203-207, May-June 2003.

8. Silva, D. A., Betioli, A. M., Gleize, P. J. P., Roman, H. R., L. A. Gomez and J. L. D. Ribeiro, "Degradation of Recycled PET Fibers in Portland Cement-Based Materials", *Cement and Concrete Research*, Vol. 35, No. 9, pp. 1741-1746, September 2005.
9. Palos, A., D'Souza, N. A., C. T. Snively and R. F. Reidy III, "Modification of Cement Mortar with Recycled ABS", *Cement and Concrete Research*, Vol. 31, No. 7, pp. 1003-1007, July 2001.
10. Henrichsen, A., "Use of Recycled Aggregates in Europe", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 1-8.
11. Katz, A., "Properties of Concrete Made with Recycled Aggregate from Partially Hydrated Old Concrete", *Cement and Concrete Research*, Vol. 33, No. 5, pp. 703-711, May 2003.
12. Lin, Y. H., Tyan, Y. Y., T. P. Chang and C. Y. Chang, "An Assessment of Optimal Mixture for Concrete Made with Recycled Concrete Aggregates", *Cement and Concrete Research*, Vol. 34, No. 8, pp. 1373-1380, August 2004.
13. Ajdukiewicz, A. and A. Kliszczewicz, "Influence of Recycled Aggregates on Mechanical Properties of HS/HPC", *Cement and Concrete Composites*, Vol. 24, No. 2, pp. 269-279, April 2002.
14. Lam, L., C. S. Poon and S. C. Kou, "Use of Recycled Aggregates in Molded Concrete Bricks and Blocks", *Construction and Building Materials*, Vol. 16, No. 5, pp. 281-289, July 2002.
15. Nixon, P. J., "Recycled Concrete as an Aggregate for Concrete – A Review", *Materials and Structures (37-Demolition and Re-use of Concrete Committee)*, Vol. 11, No. 65, pp. 371-378, September 1978.

16. Hansen, T. C., "Recycled Aggregates and Recycled Aggregate Concrete Second State-of-the Art Report Developments 1945-1985", *Materials and Structures (RILEM Technical Committee 37-Demolition and Re-use of Concrete)*, Vol. 19, No. 111, pp. 201-246, May 1986.
17. RILEM 121 DRG, "Specifications for Concrete with Recycled Aggregates", *Materials and Structures*, Vol. 27, No. 9, pp. 557-559, November 1994.
18. Roos, F., *Ein Beitrag zur Bemessung von Beton mit Zuschlag aus Rezyklierter Gesteinskörnung nach DIN 1045-1*, <http://www.mb.bv.tum.de/index-d/forschung/promo/Diss%20Roos.PDF>, 2002.
19. Chan, C., *Recycled Aggregate in Shotcrete and Concrete*, <http://www.civil.ubc.ca/home/mat/cesar.html>, 1998.
20. Limbachiya, M. C., T. Leelawat and R. K. Dhir, "Use of Recycled Concrete Aggregate in High-Strength Concrete", *Materials and Structures*, Vol. 33, No. 9, pp. 574-580, November 2000.
21. Buyle-Bodin, F. and R. Hadjieva-Zaharieava, "Influence of Industrially Produced Recycled Aggregates on Flow Properties of Concrete", *Materials and Structures*, Vol. 35, No. 8, pp. 504-509, September 2002.
22. Kobayashi, S. and H. Kawano, "Properties and Usage of Recycled Aggregate Concrete", *Proceedings of the Second International Symposium held by RILEM on Demolition and Reuse of Concrete Masonry*, Tokyo-Japan, 7-11 November 1988, Vol. 2, pp. 547-556, 1988, Chapman and Hall.
23. Nagataki, S., Gokce, A., T. Saeki and M. Hisada, "Assessment of Recycling Process Induced Damage Sensitivity of Recycled Concrete Aggregates", *Cement and Concrete Research*, Vol. 34, No. 6, pp. 965-971, June 2004.

24. Ravindrarajah, R. S., M. Stewart and D. Greco, "Variability of Recycled Concrete Aggregate and Its Effects on Concrete Properties: A Case Study in Australia", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 15-31.
25. Gokce, A., *Investigation of the Parameters Controlling Frost Resistance of Recycled Aggregate Concrete*, Doctoral Thesis, Niigata University, 2001.
26. Topçu, İ. B. and S. Şengel, "Properties of Concretes Produced With Waste Concrete Aggregate", *Cement and Concrete Research*, Vol. 34, No. 8, pp. 1307-1312, August 2004.
27. Schultmann, F., *Deconstruction in Germany*, <http://web.dcp.ufl.edu/ckibert/DeconstructionBook/CountryReports/2.Germany2003.doc>.
28. Poon, C. S., S. C. Kou and L. Lam, "Influence of Recycled Aggregate on Slump and Bleeding of Fresh Concrete", *Materials and Structures*, DOI 10.1617/s11527-006-9192-y, RILEM, 2006.
29. Nealen, A. and S. Schenk, *The Influence of Recycled Aggregate Core Moisture on Freshly Mixed and Hardened Concrete Properties*, <http://www.b-i-m.de/public/tudmassiv/dacon13nealenschenk.htm>.
30. Sagoe-Crentsil, K. K., T. Brown and A. H. Taylor, "Performance of Concrete Made with Commercially Produced Coarse Recycled Concrete Aggregate", *Cement and Concrete Research*, Vol. 31, No. 5, pp. 707-712, May 2001.
31. Xiao, J., J. Li and C. Zhang, "Mechanical Properties of Recycled Aggregate Concrete Under Uniaxial Loading", *Cement and Concrete Research*, Vol. 35, No. 6, pp. 1187-1194, June 2005.

32. Poon, C. S., Shui, Z. H., Lam, L., H. Fok and S. C. Kou, "Influence of Moisture States of Natural and Recycled Aggregates on the Slump and Compressive Strength of Concrete", *Cement and Concrete Research*, Vol. 34, No. 1, pp. 31-36, January 2004.
33. Khalaf, F. M. and DeVenny A. S., "Recycling of Demolished Masonry Rubble as Coarse Aggregate in Concrete: Review", *Journal of Materials in Civil Engineering*, Vol. 16, No. 4, pp. 331-340, July/August 2004.
34. Tamura, H., Nishio, A., J. Ohashi and K. Imamoto, "High Quality Recycled Aggregate Concrete (HiRAC) Processed by Decompression and Rapid Release", *Fifth International Conference on Recent Advances in Concrete Technology (ACI Special Publication)*, Vol. 200, pp. 491-502, June 2001.
35. Xiao, J., J. Li and C. Zhang, "On Relationships Between the Mechanical Properties of Recycled Aggregate Concrete: An Overview", *Materials and Structures*, Vol. 39, No. 6, pp. 655-664, July 2006.
36. Etxeberria, M., A. R. Mari and E. Vazquez, "Recycled Aggregate Concrete as Structural Material", *Materials and Structures*, Vol. 40, No. 5, pp. 529-541, June 2007.
37. Chen, H. J., T. Yen and K. H. Chen, "Use of Building Rubbles as Recycled Aggregates", *Cement and Concrete Research*, Vol. 33, No. 1, pp. 125-132, January 2003.
38. Kheder, G. F. and S. A. Al-Windawi, "Variation in Mechanical Properties of Natural and Recycled Aggregate Concrete as Related to the Strength of their Binding Mortar", *Materials and Structures*, Vol. 38, No. 7, pp. 701-709, August 2005.

39. Shima, H., Tateyashiki, H., Nakato, T., Okamoto, M., Y. Koga and K. Hashimoto, "New Technology for Recovering High Quality Aggregate from Demolished Concrete", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 33-44.
40. Yamato, T., M. Soeda and Y. Emoto, "Physical Properties of Recycled Aggregate and Utilization as Coarse Aggregate", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 59-68.
41. Matsushita, H., H. Tsuruta and T. Chikada, "Experimental Studies on Compressive Strength and Drying Shrinkage of Recycled Aggregate Concrete", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 69-78.
42. Ayano, T. and K. Sakata, "Improvement of Concrete with Recycled Aggregate", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 79-86.
43. Fukute, T., Hamada, H., Yamaji, T., M. Ito and K. Hayakawa, "A Study on the Applicability of Recycled Aggregate Concrete under Marine Environments", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 115-130.
44. Tamura, H., Nishio, A., M. Arai and K. Imamoto, "Utilization of HiRAC Using Two Innovative Technologies", *Proceedings of International Seminar on Recycled Concrete*, Niigata-Japan, 29 September 2000, pp. 141-154.
45. Topçu, İ. B., "Physical and Mechanical Properties of Concretes Produced with Waste Concrete", *Cement and Concrete Research*, Vol. 27, No. 12, pp. 1817-1823, December 1997.

46. Gomez-Soberon, J. M. V., "Shrinkage of Concrete with Replacement of Aggregate with Recycled Concrete Aggregate", *Special Publication of the ACI Fifth International Conference on Innovation in Design with Emphasis on Seismic, Wind, and Environmental Loading, Quality Control, and Innovation in Materials/ Hot-Weather Concreting*, Cancun-Mexico, 26 September 2002, Vol. 209, pp. 475-496.
47. Gomez-Soberon, J. M. V., "Creep of Concrete with Substitution of Normal Aggregate by Recycled Concrete Aggregate", *Special Publication of the ACI Fifth International Conference on Innovation in Design with Emphasis on Seismic, Wind, and Environmental Loading, Quality Control, and Innovation in Materials/ Hot-Weather Concreting*, Cancun-Mexico, 26 September 2002, Vol. 209, pp. 461-474.
48. Buttler, A. M. and E. F. Machado, "Properties of Concrete with Recycled Concrete Coarse Aggregates", *ACI Symposium Publication: The Fourth International ACI/CANMET Conference*, Brazil, 1 September 2005, Vol. 229, pp. 497-510.
49. Gokce, A., Nagataki, S., T. Saeki and M. Hisada, "Freezing and Thawing Resistance of Air-entrained concrete Incorporating Recycled Coarse Aggregate: The Role of Air Content in Demolished Concrete", *Cement and Concrete Research*, Vol. 34, No. 5, pp. 799-806, May 2004.
50. Neville, A., "Consideration of Durability of Concrete Structures: Past, Present, and Future", *Materials and Structures*, Vol. 34, No. 2, pp. 114-118, March 2001.
51. Bimel, C., "Choosing Concrete Proportions for Durable Floors", *Concrete International*, Vol. 28, No. 3, pp. 60-62, March 2006.
52. Mather, B., "Concrete Durability", *Cement and Concrete Composites*, Vol. 26, No. 1, pp. 3-4, January 2004.

53. Sarja, A., “Durability Design of Concrete Structures – Committee Report 130-CSL”, *Materials and Structures*, Vol. 33, No. 1, pp. 14-20, January 2000.
54. Stanish, K. D., R. D. Hooton and M. D. A. Thomas, *Testing the Chloride Penetration Resistance of Concrete: A Literature Review*, <http://www.tfhrc.gov/hnr20/pubs/chlconcrete.pdf>.
55. Zaharieva, R., F. Buyle-Bodin and E. Wirqun, “Frost Resistance of Recycled Aggregate Concrete”, *Cement and Concrete Composites*, Vol. 34, No. 10, pp. 1927-1932, October 2004.
56. Rao, A., K. N. Jha and S. Misra, “Use of Aggregates from Recycled Construction and Demolition Waste in Concrete”, *Resources Conservation & Recycling*, Vol. 50, No. 1, pp. 71-81, March 2007.
57. Zaharieva, R., Buyle-Bodin, F., F. Skoczylas and E. Wirquin, “Assessment of the Surface Permeation Properties of Recycled Aggregate Concrete”, *Cement and Concrete Composites*, Vol. 25, No. 2, pp. 223-232, February 2003.
58. Olorunsogo, F. T. and N. Padayachee, “Performance of Recycled Aggregate Concrete Monitored by Durability Indexes”, *Cement and Concrete Research*, Vol. 32, No. 2, pp. 179-185, February 2002.
59. Levy, S. M. and P. Helene, “Durability of Recycled Aggregates Concrete: A Safe Way to Sustainable Development”, *Cement and Concrete Research*, Vol. 34, No. 11, pp. 1975-1980, November 2004.
60. Poon, C. S. and D Chan, “Effects of Contaminants on the Properties of Concrete Paving Blocks Prepared with Recycled Concrete Aggregates”, *Construction and Building Materials*, Vol. 21, No. 1, pp. 164-175, January 2007.

61. Otsuki, N., S. Miyazato and W. Yodsudjai, "Influence of Recycled Aggregate on Interfacial Transition Zone, Strength, Chloride Penetration and Carbonation of Concrete", *Journal of Materials in Civil Engineering*, Vol. 15, No. 5, pp. 443-451, September / October 2003.
62. Tamura, H., M. Arai and K. Imamoto, "Innovative Methods for Water Content Control of Recycled Aggregate Concrete: Rapid Absorption Test for Aggregate and Continuous Monitoring of Freshly-Mixed Concrete at Site", *ACI Special Publication: The Fifth CANMET/ACI International Conference on Recent Advances in Concrete Technology*, 1 June 2001, Vol. 200, pp. 769-780.
63. Soberon, G., Manuel, J., F. L. Agullo and V. R. Enric, "Repercussions on Concrete Permeability Due to Recycled Concrete Aggregate", *ACI Special Publication: The Third CANMET/ACI International Symposium on Sustainable Development of Cement and Concrete*, 1 August 2001, Vol. 202, pp. 181-195.
64. Abou-Zeid, M. N., Shenouda, M., S. McCabe and F. El-Tawil, "Reincarnation of Concrete", *Concrete International*, Vol. 27, No. 2, pp. 53-59, February 2005.
65. Eguchi, K., Teranishi, K., Nakagome, A., Kishimoto, H., K. Shinozaki and M. Narikawa, "Application of Recycled Coarse Aggregate by Mixture to Concrete Construction", *Construction and Building Materials*, Vol. 21, No. 7, pp. 1542-1551, June 2007.
66. Lee, S. T., Moon, H. Y., Swamy, R. N., S. S. Kim and J. P. Kim, "Sulfate Attack of Mortars Containing Recycled Fine Aggregates", *ACI Materials Journal*, Vol. 102, No. 4, pp. 224-230, July-August 2005.
67. Scott IV, H. C. and D. L. Gress, "Mitigating Alkali Silica Reaction in Recycled Concrete", *ACI Special Publication: 2003 ACI Spring Convention in Vancouver, Canada on Recycling Concrete and other Materials for Sustainable Development*, 1 March 2004, Vol. 219, pp. 61-76.

68. Salem, R. M., E. G. Burdette and N. M. Jackson, "Resistance to Freezing and Thawing of Recycled Aggregate Concrete", *ACI Materials Journal*, Vol. 100, No. 3, pp. 216-221, May-June 2003.
69. Salem, R. M. and E. G. Burdette, "Role of Chemical and Mineral Admixtures on the Physical Properties and Frost Resistance of Recycled Aggregate Concrete", *ACI Materials Journal*, Vol. 95, No. 5, pp. 558-563, September 1998.
70. Tu, T. Y., Y. Y. Chen and C. L. Hwang, "Properties of HPC with Recycled Aggregates", *Cement and Concrete Research*, Vol. 36, No. 5, pp. 943-950, May 2006.
71. Calder, A., "The Use of Recycled Concrete Aggregate in Structural Concrete", *Concrete*, Vol. 39, No. 5, pp. 62-63, May 2005.
72. Ghailan, A. H., "Modified Concrete by Using a Waste Material as a Coarse Aggregate", *Proceedings of the Congress: Construction Research Congress 2005: Broadening Perspectives*, 5-7 April 2005, San Diego, California
73. Shayan, A. and A. Xu, "Performance and Properties of Structural Concrete Made with Recycled Concrete Aggregate", *ACI Materials Journal*, Vol. 100, No. 5, pp. 371-380, September-October 2003.
74. Meinhold, U., G. Mellmann and M. Maultzsch, "Performance of High-Grade Concrete with Full Substitution of Aggregates by Recycled Concrete", *ACI Special Publication: The Third CANMET/ACI International Symposium on Sustainable Development of Cement and Concrete*, 1 August 2001, Vol. 202, pp. 85-96.

75. Corinaldesi, V., G. Moriconi and F. Tittarelli, “Sustainable and Durable Reinforced Concrete Construction”, *ACI Special Publication: The Fifth International Conference on Innovation in Design with Emphasis on Seismic, Wind, and Environmental Loading, Quality Control, and Innovation in Materials/ Hot-Weather Concreting*, Cancun-Mexico, 26 September 2002, Vol. 209, pp. 169-186.
76. Culfik, M. S. and T. Özturan, “Effect of Elevated Temperatures on the Residual Mechanical Properties of High-Performance Mortar”, *Cement and Concrete Research*, Vol. 32, No. 5, pp. 809-816, May 2002.
77. Fu, Y. F., Wong, Y. L., C. A. Tang and C. S. Poon, “Thermal Induced Stress and Associated Cracking in Cement-Based Composite at Elevated Temperatures – Part II: Thermal Cracking Around Multiple Inclusions”, *Cement and Concrete Composites*, Vol. 26, No. 2, pp. 113-126, February 2004.
78. Fu, Y. F., Wong, Y. L., C. A. Tang and C. S. Poon, “Thermal Induced Stress and Associated Cracking in Cement-Based Composite at Elevated Temperatures – Part I: Thermal Cracking Around Single Inclusion”, *Cement and Concrete Composites*, Vol. 26, No. 2, pp. 99-111, February 2004.
79. Yüzer, N., F. Aköz and L. D. Öztürk, “Compressive Strength – Color Change Relation in Mortars at High Temperature”, *Cement and Concrete Research*, Vol. 34, No. 10, pp. 1803-1807, October 2004.
80. Arioz, O., “Effects of Elevated Temperatures on Properties of Concrete”, *Fire Safety Journal*, doi: 10.1016/j.firesaf.2007.01.03, 2007.
81. Handoo, S. K., S. Agarwal and S. K. Agarwal, “Physicochemical, Mineralogical, and Morphological Characteristics of Concrete Exposed to Elevated Temperatures”, *Cement and Concrete Research*, Vol. 32, No. 7, pp. 1009-1018, July 2002.

82. Zhang, B., Bicanic, N., C. J. Pearce and D. V. Phillips, "Relationship Between Brittleness and Moisture Loss of Concrete Exposed to High temperatures", *Cement and Concrete Research*, Vol. 32, No. 3, pp. 363-371, March 2002.
83. Sakr, K. and E. El-Hakim, "Effect of High Temperature or Fire on Heavy Weight Concrete Properties", *Cement and Concrete Research*, Vol. 35, No. 3, pp. 590-596, March 2005.
84. Hossain, K. M. A., "Macro- and Microstructural Investigations on Strength and Durability of Pumice Concrete at High Temperatures", *Journal of Materials in Civil Engineering*, Vol. 18, No. 4, pp. 527-536, July / August 2006.
85. Noumowe, A., "Mechanical Properties and Microstructure of High Strength Concrete Containing Polypropylene Fibres Exposed to Temperatures up to 200 °C", *Cement and Concrete Research*, Vol. 35, No. 11, pp. 2192-2198, November 2005.
86. Savva, A., P. Manita and K. K. Sideris, "Influence of Elevated Temperatures on the Mechanical Properties of Blended Cement Concretes Prepared with Limestone and Siliceous Aggregates", *Cement and Concrete Composites*, Vol. 27, No. 2, pp. 239-248, February 2005.
87. Lion, M., Skoczylas, F., Z. Lafhaj and M. Sersar, "Experimental Study on a Mortar. Temperature Effects on Porosity and Permeability. Residual Properties or Direct Measurements under Temperature", *Cement and Concrete Research*, Vol. 35, No. 10, pp. 1937-1942, October 2005.
88. Terro, M. J., "Properties of Concrete Made with Recycled Crushed Glass at Elevated Temperatures", *Building and Environment*, Vol. 41, No. 5, pp. 633-639, May 2006.

89. Chang, Y. F., Chen, Y. H., M. S. Sheu and G. C. Yao, "Residual Stress – Strain Relationship for Concrete after Exposure to High Temperatures", *Cement and Concrete Research*, Vol. 36, No. 10, pp. 1999-2005, October 2006.
90. Khoury, G. A., "Effect of Fire on Concrete and Concrete Structures", *Progress in Structural Engineering and Materials*, Vol. 2, No. 4, pp. 429-447, October / December 2000.
91. Lau, A. and M. Anson, "Effect of High Temperatures on High Performance Steel Fibre Reinforced Concrete", *Cement and Concrete Research*, doi:10.1016/j.cemconres.2006.03.24, 2006.
92. Chan, Y. N., G. F. Peng and M. Anson, "Residual Strength and Pore Structure of High – Strength Concrete and Normal Strength Concrete after Exposure to High Temperatures", *Cement and Concrete Composites*, Vol. 21, No. 1, pp. 23-27, February 1999.
93. Xu, Y., Wong, Y. L., C. S. Poon and M. Anson, "Impact of High Temperature on PFA Concrete", *Cement and Concrete Research*, Vol. 31, No. 7, pp. 1065-1073, July 2001.
94. Wang, X. S., B. S. Wu and Q. Y. Wang, "Online SEM Investigation of Microcrack Characteristics of Concretes at Various Temperatures", *Cement and Concrete Research*, Vol. 35, No. 7, pp. 1385-1390, July 2005.
95. Naus, D. J., *Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures – A Review of Pertinent Factors*, <http://www.nrc.gov/reading-rm/doc-collections/nuregs/contract/cr6927/cr-6927.pdf>
96. Poon, C. S., Azhar, S., M. Anson and Y. L. Wong, "Comparison of the Strength and Durability Performance of Normal- and High – Strength Pozzolanic Concretes at Elevated Temperatures", *Cement and Concrete Research*, Vol. 31, No. 9, pp. 1291-1300, September 2001.

97. Gawin, D., F. Pesavento and B. A. Schrefler, "Modelling of Deformations of High Strength Concrete at Elevated Temperatures", *Materials and Structures*, Vol. 37, No. 4, pp. 218-236, May 2004.
98. Nielsen, C. V. and N. Bicanic, "Residual Fracture Energy of High – Performance and Normal Concrete Subject to High Temperatures", *Materials and Structures*, Vol. 36, No. 8, pp. 515-521, October 2003.
99. Peng, G. F. and Z. S. Huang, "Change in Microstructure of Hardened Cement Paste Subjected to Elevated Temperatures", *Construction and Building Materials*, doi:10.1016/j.conbuildmat.2006.11.002, 2006.
100. Peng, G. F., Yang, W. W., Zhao, J., Liu, Y. F., S. H. Bian and L. H. Zhao, "Explosive Spalling and Residual Mechanical Properties of Fiber – Toughened High – Performance Concrete Subjected to High Temperatures", *Cement and Concrete Research*, Vol. 36, No. 4, pp. 723-727, April 2006.
101. Phan, L. T. and N. J. Carino, "Code Provisions for High Strength Concrete Strength – Temperature Relationship at elevated Temperatures", *Materials and Structures*, Vol. 36, No. 2, pp. 91-98, March 2003.
102. Poon, C. S., Azhar, A., M. Anson and Y. L. Wong, "Performance of Metakaolin Concrete at Elevated Temperatures", *Cement and Concrete Composites*, Vol. 25, No. 1, pp. 83-89, January 2003.
103. Chung, J. H. and G. R. Consolazio, "Numerical Modeling of Transport Phenomena in Reinforced Concrete Exposed to Elevated Temperatures", *Cement and Concrete Research*, Vol. 35, No. 3, pp. 597-608, March 2005.
104. Kalifa, P., F. D. Menneveau and D. Quenard, "Spalling and Pore Pressure in HPC at High Temperatures", *Cement and Concrete Research*, Vol. 30, No. 12, pp. 1915-1927, December 2000.

105. Cülfik, M. S., *Deterioration of Bond Between Cement Paste and Aggregate at High Temperatures*, Doctoral Thesis, Boğaziçi University, 2001.
106. Poon, C. S., Z. H. Shui and L. Lam, “Compressive Behavior of Fiber Reinforced High – Performance Concrete Subjected to Elevated Temperatures”, *Cement and Concrete Research*, Vol. 34, No. 12, pp. 2215-2222, December 2004.
107. Chan, Y. N., X. Luo and W. Sun, “Compressive Strength and Pore Structure of High – Performance Concrete after Exposure to High Temperature up to 800 °C”, *Cement and Concrete Research*, Vol. 30, No. 2, pp. 247-251, February 2000.
108. Phan, L. T. and N. J. Carino, “Review of Mechanical Properties of HSC at Elevated Temperature”, *Cement and Concrete Research*, Vol. 10, No. 1, pp. 58-64, February 1998.
109. Balendran, R. V., Rana, T. M., T. Maqsood and W. C. Tang, “Strength and Durability Performance of HPC Incorporating Pozzolans at Elevated Temperatures”, *Structural Survey*, Vol. 20, No. 4, pp. 123-128, October 2002.
110. Youssef, M. A. and M. Moftah, “General Stress – Strain Relationship for Concrete at Elevated Temperatures”, *Engineering Structures*, doi:10.1016/j.engstruct.2007.01.002, 2007.
111. Palardy, D., M. Onofrei and G. Ballivy, “Microstructural Changes Due to Elevated Temperature in Cement Based Grouts”, *Advanced Cement Based Materials*, Vol. 8, No. 3, pp. 132-138, October 1998.
112. Skvara, F. and V. Sevcik, “Influence of High Temperature on Gypsum – Free Portland Cement Materials”, *Cement and Concrete Research*, Vol. 29, No. 5, pp. 713-717, May 1999.

113. Cheng, F. P., V. K. R. Kodur and T. C. Wang, "Stress – Strain Curves for High Strength Concrete at Elevated Temperatures", *Journal of Materials in Civil Engineering*, Vol. 16, No. 1, pp. 84-90, January / February 2004.
114. Elbadry, M. M., H. Abdalla and A. Ghali, "Effects of Temperature on the Behaviour of Fiber Reinforced Polymer Reinforced Concrete Members: Experimental Studies", *Canadian Journal of Civil Engineering*, Vol. 27, No. 5, pp. 993-1004, October 2000.
115. Kumar, R. and B. Bhattacharjee, "Study on Some Factors Affecting the Results in the Use of MIP Method in Concrete Research", *Cement and Concrete Research*, Vol. 33, No. 3, pp. 417-424, March 2003.
116. Güneyisi, E., *Mechanical and Durability Performance of Plain and Blended Cement Concrete Exposed to Chlorides and Different Curing Regimes*, Doctoral Thesis, Boğaziçi University, 2004.
117. Moon, H. Y., H. S. Kim and D. S. Choi, "Relationship Between Average Pore Diameter and Chloride Diffusivity in Various Concretes", *Construction and Building Materials*, Vol. 20, No. 9, pp. 725-732, November 2006.
118. Kumar, R. and B. Bhattacharjee, "Assessment of Permeation Quality of Concrete through Mercury Intrusion Porosimetry", *Cement and Concrete Research*, Vol. 34, No. 2, pp. 321-328, February 2004.
119. Diamond, S., "Mercury Porosimetry an Inappropriate Method for the Measurement of Pore Size Distributions in Cement – Based Materials", *Cement and Concrete Research*, Vol. 30, No. 10, pp. 1517-1525, October 2000.
120. Yang, C. C., "On the Relationship Between Pore Structure and Chloride Diffusivity from Accelerated Chloride Migration Test in Cement – Based Materials", *Cement and Concrete Research*, Vol. 36, No. 7, pp. 1304-1311, July 2006.

121. Pradhan, B., M. Nagesh and B. Bhattacharjee, "Prediction of the Hydraulic Diffusivity from Pore Size Distribution of Concrete", *Cement and Concrete Research*, Vol. 35, No. 9, pp. 1724-1733, September 2005.
122. Yang, C. C. and C. T. Chiang, "On the Relationship Between Pore Structure and Charge Passed from RCPT in Mineral – Free Cement – Based Materials", *Materials Chemistry and Physics*, Vol. 93, No. 1, pp. 202-207, September 2005.
123. Cekic, A., *Nondestructive Testing and Image Analysis of Geomembranes*, Doctoral Thesis, Wayne State University, 2005.
124. Park, T., "Application of Construction and Building Debris as Base and Subbase Materials in Rigid Pavement", *Journal of Transportation Engineering*, Vol. 129, No. 5, pp. 558-563, September / October 2003.
125. Khatib, J. M., "Properties of Concrete Incorporating Fine Recycled Aggregate", *Cement and Concrete Research*, Vol. 35, No. 4, pp. 763-769, April 2005.
126. Correia, J. R., J. D. Brito and A. S. Pereira, "Effects on Concrete Durability of Using Recycled Ceramic Aggregates", *Materials and Structures*, Vol. 39, No. 2, pp. 151-158, March 2006.
127. Bayramov, F., C. Taşdemir and M. A. Taşdemir, "Optimisation of Steel Fibre Reinforced Concretes by Means of Statistical Response Surface Method", *Cement and Concrete Composites*, Vol. 26, No. 6, pp. 665-675, August 2004.
128. Baştopçu, M. E., *Effect of Binder Composition on Durability and Mechanical Properties of High Performance Self – Compacting Concrete*, Doctoral Thesis, Boğaziçi University, 2006.