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Table of Contents

The Effect of Sulfate in Sand on Some Mechanical Properties of Nano Meta kaolin Self Compacting Concrete

Dr. Ghalib Mohsin Habeeb & Zainab Hashim Abbas

Abstract— **The aim of this study is to investigate the effect of internal sulfates on some mechanical properties of self-compacting concrete containing Nona Metakaolin. Tests were conducted on mixes with varied Nona Metakaolin ranged between (1-3%) by cement weight .Two types of sand were used one with artificial SO_3 content ranged between (0.35-1%) by weight of sand, other sand is with natural SO_3 of 1%. Nano Metakaolin and SO_3 contents were the only variables among the mixtures. Results indicated that there were harmful effect of sulfates on all the mechanical properties of the self-compacting concrete. Reduction in compressive, tensile ,flexural strength and modulus of elasticity were about 13.15,4.21,12.42 MPa, and 12.63 GPa respectively at 90 days age for the artificial sand. While the reduction for natural SO_3 sand were more than of the artificial sand by about(15-34%).Mixes containing Nano Metakaolin showed less reduction in its mechanical properties by about 10%compaired with reference mixes.**

Keywords: concrete, meta kaolin, sulfate and mechanical properties.

I. INTRODUCTION

1-Exepermental Procedure

Concrete testing is performed in order to investigate the effect of internal sulfates in fine aggregate on compressive strength, splitting tensile strength, flexural strength, modulus of elasticity, ultrasonic pulse velocity and length change tests of self-compacting concrete (SCC) with adding Nano metakaolin. In this study, the percentages of sulfate in concrete ingredients are obtained by adding natural gypsum to the fine aggregate as a partial replacement by weight of fine aggregate (sand with artificial SO_3), and also used sand not confirmed to ***(IQS No.45/ 1984)*** specification (naturally) without adding gypsum. All specimens were cured in water until the time of testing and the curing time are (28, 60 and 90 days). Percentages of sulfates in fine aggregate (by weight of fine aggregate) which were used in this study are (0.35%, 0.5%, and 1.0%) for sand of artificial SO_3 and 1% for natural SO_3 sand(not confirmed to specification).

1.1Materials

Cement

Ordinary Portland cement (OPC) according to (ASTM C150-Type I) was used in this research, its chemical and physical properties are listed in Table (1)and (2).

Table (1): Chemical composition of cement.*

Compound composition	*Chemical composition*	*Percentage by weight*	*Limits of IQS 5:1984*
Lime	CaO	62.23	_
Silica	SiO_2	19.66	_
Alumina	Al_2O_3	4.66	_
Iron oxide	Fe_2O_3	3.44	_
Magnesia	MgO	2.83	<5
Sulfate	SO_3	2.61	<2.8
Free lime	Free CaO	1.23	
Loss on ignition	L.O.I	3.95	<4
Insoluble residue	I.R	1.27	<1.5
Lime saturation factor	L.S.F	0.94	0.66-1.02
Main compounds (Bogue's equation) percentage by weight of cement			
Tricalcium silicate (C_3S)		55.23	
Dicalcium silicate (C_2S)		14.70	
Tricalcium aluminate (C_3A)		6.53	
Tetraclcium aluminoferrite (C_4AF)		10.46	

*Chemical tests were conducted in Construction Materials Laboratory of Babylon University.

Table (2) Physical properties of cement.*

Physical properties	*Test results*	*Limits of Iraqi Specification NO.5 /1984)*
Setting time (Vicat's Method) Initial, (min) Final, (hr:min)	 140 4:15	 ≥ 45 min ≤ 600 min
Fineness (Blaine Method) ,m^2/kg	351	≥ 230 m^2/kg
Compressive strength, MPa 3 days 7 days	 21.5 31	 ≥ 15, MPa ≥ 23, MPa

*Physical tests were conducted by the constructional materials laboratory in University of Babylon.

Fine Aggregate

Two types of fine aggregate were used throughout this work. Both are brought from AL-Eakhadir region. Results indicate that the grading and sulfate content are conformed to the requirements of ***(IQS No.45/ 1984),*** except sulfate content of one of them which is not conformed to ***(IQS No.45/ 1984)***.Table (3) and (4) show there grading and some physical properties.

Table (3): Grading of fine aggregate.

No.	*Sieve size(mm)*	*Passing % Sand 1*	*Passing% Sand 2*	*Limits of Iraqi specification No. 45/1984 for zone 2*
1	9.5	100	100	100
1	4.75	94.4	91.8	90-100
2	2.38	79.4	75.8	75-100
3	1.18	64.6	55	55-90
4	0.59	46.8	35.2	35-59
5	0.30	24.4	10.8	8-30
6	0.15	3.2	2	0-10

Table (4): Physical and chemical properties of fine aggregate*

No.	*Properties*	*Test results Sand 2*	*Test result Sand 1*	*Limits of Iraqi specification No.45/1984*
1	Specific ravity	2.6	2.6	_
2	SO_3 content	1%	0.35%	$\leq$ 0.5%
3	Fineness Modulus	3.294	2.872	
4	Absorption	1.9%	2.1%	

* Chemical tests were conducted in Construction Materials Laboratory of Babylon University.

Coarse Aggregate

Rounded gravel of 14 mm maximum size from Al-Nebai quarry was used. The coarse aggregate was washed, and then stored in air to dry. The grading and the physical properties of this aggregate are shown in Table (5) and(6) Which is conformed to the Iraqi specification (***IQS No.45/ 1984)***.

Table (5): Grading of coarse aggregate.

No.	*Sieve size (mm)*	*Passing %*	*Limits of Iraqi specification No. 45/1984 (5-14) mm*
1	20	100	100
2	14	100	90-100
3	10	56.8	50-85
4	5	0.4	0-10

Table (6): Physical and chemical properties of coarse aggregate. *

No.	*Properties*	*Test results*	*Limits of Iraqi specification No.45/1984*
1	Specific gravity	2.65	_
2	SO_3 content	0.05%	$\leq$ 0.1%
3	Absorption	0.5%	_

* Chemical tests were conducted in Construction Materials Laboratory of Babylon.

Water & Super plasticizer

Tap water is used throughout this work for both mixing and curing of concrete. A chemical admixture based on modified polycarboxylic ether, which is known commercially (Glenium 51) was used in producing SCC as a super plasticizer admixture.

Natural Gypsum

The natural gypsum was bringing from Kufa cement factory. It was grinned by the hammer and passed through the same sieve set of fine aggregate used in the mix of internal sulfate attack to avoid the effect of large surface area of particles and used as a partial replacement by weight of fine aggregate with limited percentage. Table (7) shows its chemical properties.

Table (7): Chemical properties of Gypsum.*

Compound composition	*Percent*
SiO_2	3.32
R_2O_3= (Fe_2O_3 + Al_2O_3)	1.42
CaO	23.81
MgO	1.27
SO_3	34.7
L.O.I	20.9

*Chemical tests were conducted in Construction Materials Laboratory of Babylon University.

Nano Metakaolin

Nano metakaolin was imported from English Indian Clays Limited (EICL). The Nano meta kaolin used in this work conforms to the requirements of ***(ASTM C618, 2002)*** Class N pozzolan. The percentages of Nano metakaolin that used in this study as partial replacement with cement are (1%, 2%, and 3%) by weight of cement, Which were found to gave good fresh concrete properties and less cost***(Alsallami 2013)***.Table (8) shows its chemical and physical properties.

Table (8): Physical properties of Nano metakaolin.*

Appearance	*Off-white*
Lime reactivity (Chapelle test) mg/gm	740 – 1,000
Specific gravity	2.6
Blaine value (cm^2/g)	22,000 – 25,000
Loss on ignition	0.5 – 1.5%
pH (10% solids)	4.0 – 5.0
Bulk density, kg/lt	0.4 – 0.5
SO_3 %	0.05

*Given by manufacture

2-Methods

2-1 Introduction

Iraq and Middle East countries suffer from sand contaminated with a large quantity of sulfate which leads to exclude a large amount of these materials from using it in concrete productions. Most of the aggregates used in Iraq for concrete are sea deposits which have become contaminated with Sulfates due to the tropical dry climate. These Sulfates are the source of the problem of internal Sulfates attack in concrete ***(Al-Harbi et al., 1993) AL-Robayi, 2005*** studied the resistance of high performance concrete (HPC) to external and internal sulfate attack. He found that high performance concrete exposed to $MgSO_4$ and Na_2SO_4 solutions (external sulfate attack) showed lower strength at all ages with respect to the mix that curried in water. ***Hussain, 2008*** studied the effect of sulfate in fine aggregates on some mechanical properties of SCC, he found that there was an optimum gypsum content at which the mechanical properties are maximum.

Mahmoud, 2012 studied the effect of sulfate on the properties of self-compacting concrete reinforced by steel fiber. He found that the optimum SO_3 content, at which a higher mechanical properties and little tendency to the expanding were obtained, was at SO_3 equal to 5 (% by weight of cement), note that sulfate ratios in concrete which used in his study were (3.9, 5, 6, 7, and 8%) by weight of cement.

Nano metakaolin participates in the pozzolanic reactions, resulting in the consumption of $Ca(OH)_2$ and formation of an additional C-S-H. and improve the structure of the aggregates contact zone, resulting in a better bond between aggregates and cement paste. Nano metakaolin can fill the voids in cement, which makes the microstructure of matrix denser due to its ultra-fine particles. It reacts with $Ca(OH)_2$ released during hydration process produces excess calcium silicate hydrate thus improving of mechanical properties against internal sulfates attack.

There were little research available on the use of Nano metakaolin to improve the durability of SCC against internal sulfates attack ,so the main objective of the research is to investigate the effect of internal sulfates in fine aggregate on compressive strength, splitting tensile strength, flexural strength, modulus of elasticity, ultrasonic pulse velocity and length change tests of SCC, with adding Nano metakaolin.

3-Concrete Mixes

In order to achieve the scopes of this study, the total of 16 SCC mixtures were made, all based on the same control mixture. The concrete was designed according to the (***EFNARC, 2005)*** method. Table (9)provides the proportions of the reference mixture, from which all other mixtures were developed. The mix proportions, w/p ratio and SP kept constant for all mixes, the only variations in the mixture were the Nano meta kaolin contents and SO_3 contents in fine aggregate in order to investigate only the effect of sulfates on SCC with different Nano meta kaolin contents on its properties in fresh and hardened state and compare its behavior with the behavior of plain SCC. Curing time was for three ages (28, 60, 90) days. Table (10) shows concrete mix designations.

Table (9) Proportions of reference mixture.

Material	*Amount*
Cement (kg/m^3)	500
Water (L/m^3)	185
Sand (kg/m^3)	882
Gravel (kg/m^3)	780
SP (Liter/100kg cement)	0.8
w/c or w/p	0.37

Table (10) Concrete mix designations.

Mix No.	*Mix symbol*	*Type of fine aggregate*	*So_3%by weight of sand*	*Nano-metakaolin (% by weight of cement)*
1	Z1	Sand 1*	0.35%	Mix without (NMK)
2	Z2	Sand 1	0.35%	Mix with 1% of (NMK)
3	Z3	Sand 1	0.35%	Mix with 2% of (NMK)
4	Z4	Sand 1	0.35%	Mix with 3% of (NMK)
5	Z5	Sand 1	0.5%	Mix without (NMK)
6	Z6	Sand 1	0.5%	Mix with 1% of (NMK)
7	Z7	Sand 1	0. 5%	Mix with 2% of (NMK)
8	Z8	Sand 1	0.5%	Mix with 3% of (NMK)
9	Z9	Sand 1	1%	Mix without (NMK)
10	Z10	Sand 1	1%	Mix with 1% of (NMK)
11	Z11	Sand 1	1%	Mix with 2% of (NMK)
12	Z12	Sand 1	1%	Mix with 3% of (NMK)
13	Z13	Sand 2**	1%	Mix without (NMK)
14	Z14	Sand 2	1%	Mix with 1% of (NMK)
15	Z15	Sand 2	1%	Mix with 2% of (NMK)
16	Z16	Sand 2	1%	Mix with 3% of (NMK)

* sand with artificial SO_3.
** sand with natural SO_3.

4-Fresh Concrete Tests

The fresh properties of SCC were tested by the procedures of ***(European Guidelines for self-compacting concrete)***. In this work three tests were used slump flow test, L-box test and V-funnel test. These mixes were carried out to ensure that the mixes satisfy the requirements of SCC. All mixes were satisfied to the requirements of ***(EFNARC, 2005)***.

5-Hardened Concrete Properties

The mechanical properties studied were compressive strength, splitting tensile strength, flexural strength and static modulus of elasticity. Furthermore, the non-destructive test methods, ultrasonic pulse velocity test and length change test were used. The compressive strength test was performed in accordance with ***(BS. 1881: Part 116: 1989)***. The splitting tensile strength test was carried out according to ***(ASTM C496-2004)***. The test procedure given in ***(ASTM C78-84, 2003)*** was used to determine the flexural strength. The static modulus of elasticity test was performed in accordance with ***(ASTM C469-2002)***. The ultrasonic velocity test was performed according with ***(ASTM C597, 2002)*** and length change test was performed according with ***(ASTM C 157/ C157M, 2004)*** and ***(ASTM C490, 2004)***.Plate (1) and (2) shows experimental works.

Plate (1) Curing of specimens.

Plate (2) Compressive strength test.

6-Result of Tests of Hardened Concrete Properties

Fig (1) shows the results of compressive strength at 28,60 and 90 days age gained from cubes. The results indicated that the optimum SO_3 content for mixes is about 0.5 (% by weight of fine aggregate), beyond this optimum value the compressive strength decreased with the increase of sulfates content for all mixes, at all ages of test. The results indicated also that all specimens exhibited a continuous increase in compressive strength with progress in age. The deviation from the average value of the compressive strength were 0.5 MPa Also, it was clear from the results that the compressive strength increased with adding Nano meta kaolin at all mixes and at all ages. It has been suggested that the surface of pozzolan can adsorb many Ca^{+2} ions and that lowering of the concentration of the calcium ions accelerates the rate of dissolution of C_3S that increases the rate of hydration ***(Zelic et al., 2000)***. The strength enhancement of Nano meta kaolin can be attributed to reduction in the content of $Ca(OH)_2$ which has not any cementing property and thus reduce the sulfate formation and production of hydrated calcium silicate (C-S-H) that plays a vital role in mechanical properties of concrete. It was clear from the results that mixes with sand of natural SO_3 gave compressive strength less than that with sand of artificial SO_3 and the decreasing percent decreases with adding Nano meta kaolin and with age.

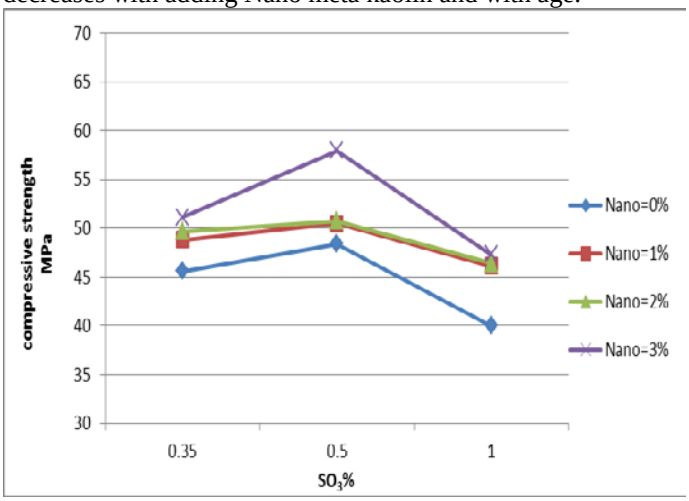


Figure (1-a) Effect of increasing SO_3 content on compressive strength at 28 days (sand 1).

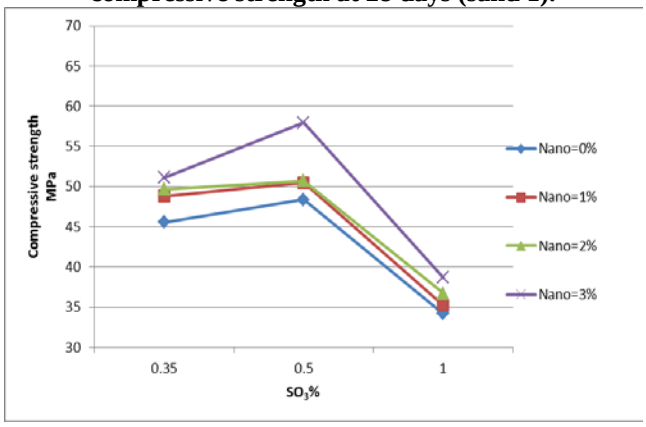


Figure (1-b) Effect of increasing SO_3 content on compressive strength at 28 days (sand 2).

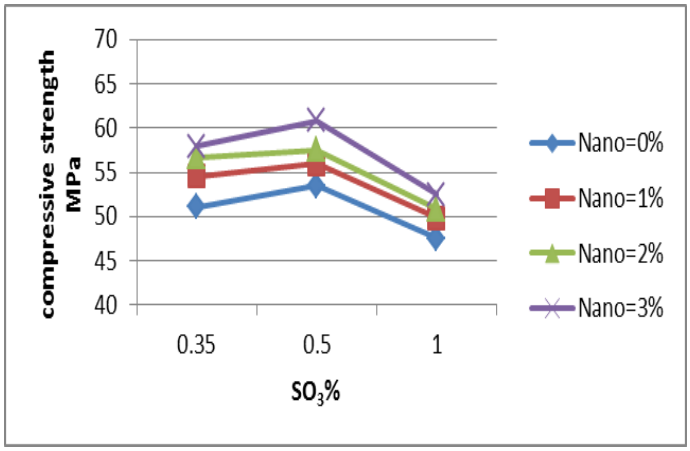


Figure (1-c) Effect of increasing SO_3 content on compressive strength at 60 days (sand 1).

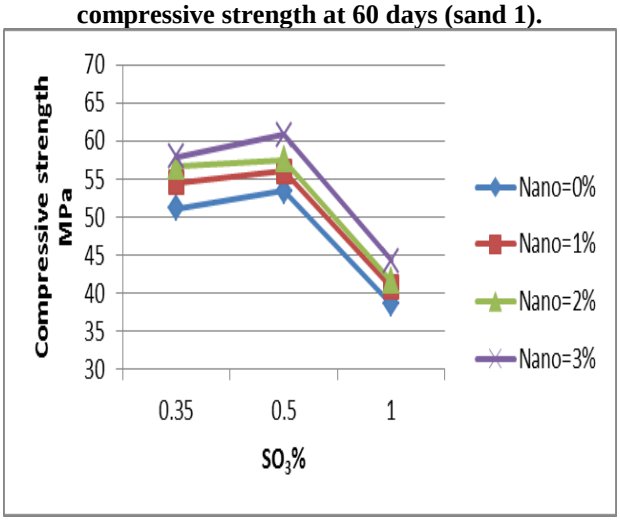


Figure (1-d) Effect of increasing SO_3 content on compressive strength at 60 days (sand 2).

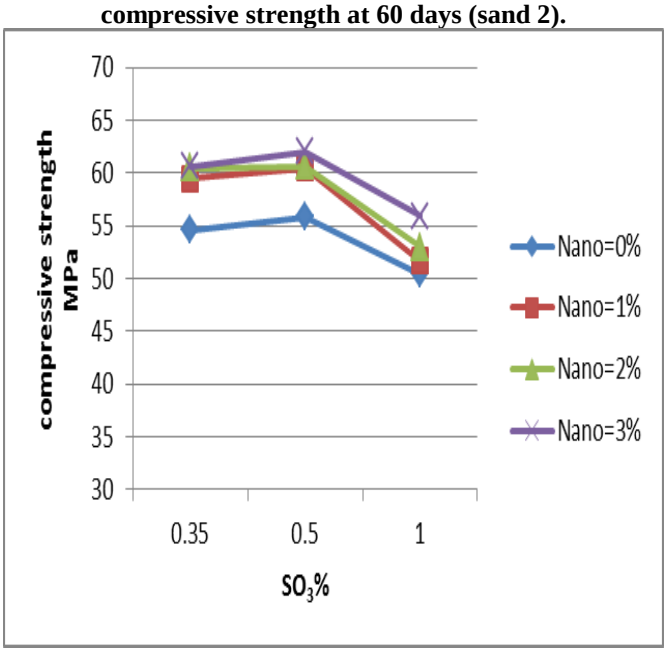


Figure (1-e) Effect of increasing SO_3 content on compressive strength at 90 days (sand 1).

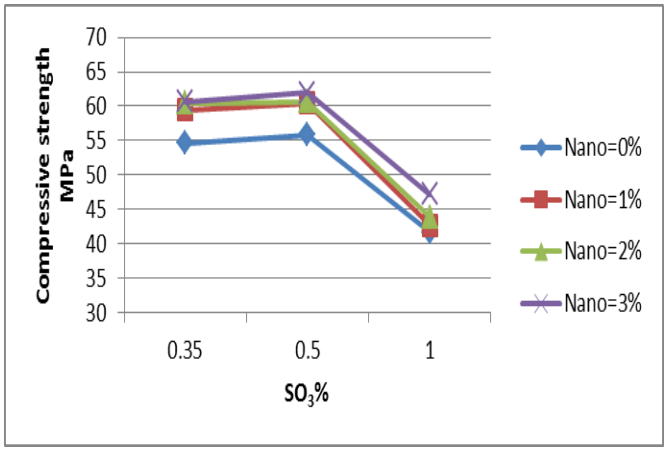


Figure (1-f) Effect of increasing SO_3 content on compressive strength at 90 days (sand 2).

The results of splitting tensile strength at 28,60 and 90 days age gained from cylinders are shown in Fig.(2). It is clear that the optimum SO_3 content for mixes was about (0.5%) (% by weight of fine aggregate), beyond this optimum value the splitting tensile strength decreased with the increase of sulfates content for all mixes, at all ages of test. The results indicated also that all specimens exhibited a continuous increase in splitting tensile strength with progress in age. Also, it was clear from the results that the splitting tensile strength increased with adding Nano meta kaolin at all mixes and at all ages. The NMK enhanced the tensile strength of hardened cement mortar by two mechanisms. The first of which is the packing effect of NMK as filler into interstitial spaces inside the skeleton of hardened microstructure of cement mortar and thus increasing its density as well as the strength. The second was the pozzolanic effect ***(Morsy at el., 2008)***. It was clear from the results that mixes with sand of natural SO_3 gave splitting tensile strength less than that with sand of artificial SO_3 and the decreasing percent decreased with adding Nano meta kaolin and with age.

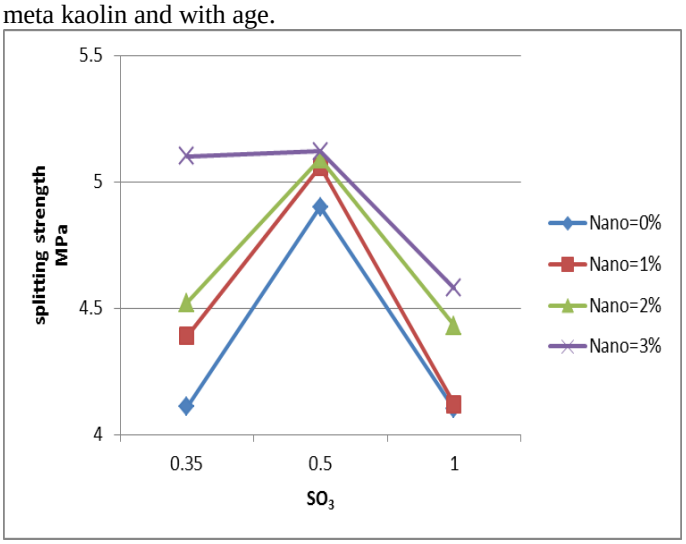


Figure (2-a) Effect of increasing SO_3 content on splitting strength at 28 days (sand 1).

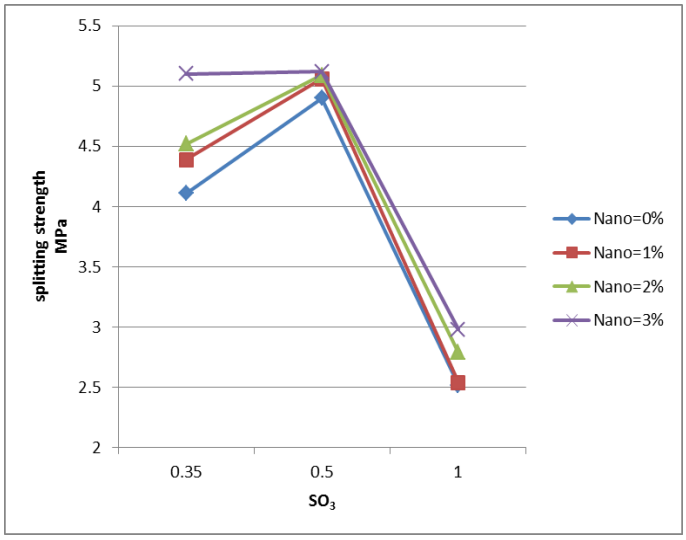


Figure (2-b) Effect of increasing SO_3 content on splitting strength at 28 days (sand 2).

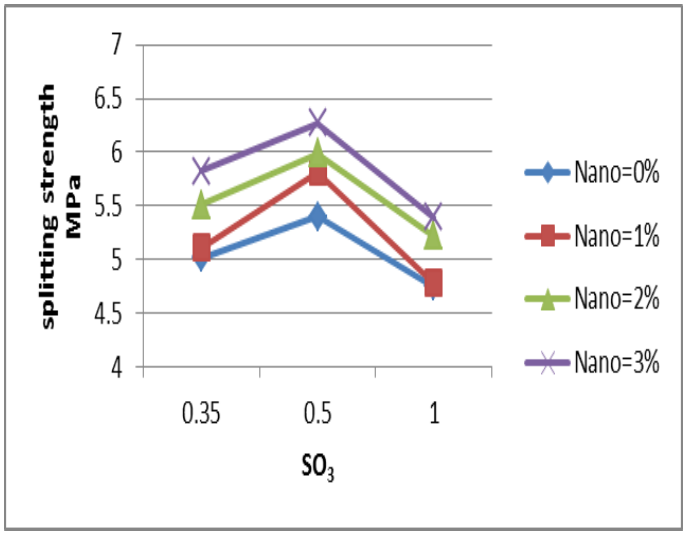


Figure (2-c) Effect of increasing SO_3 content on splitting strength at 60 days (sand 1).

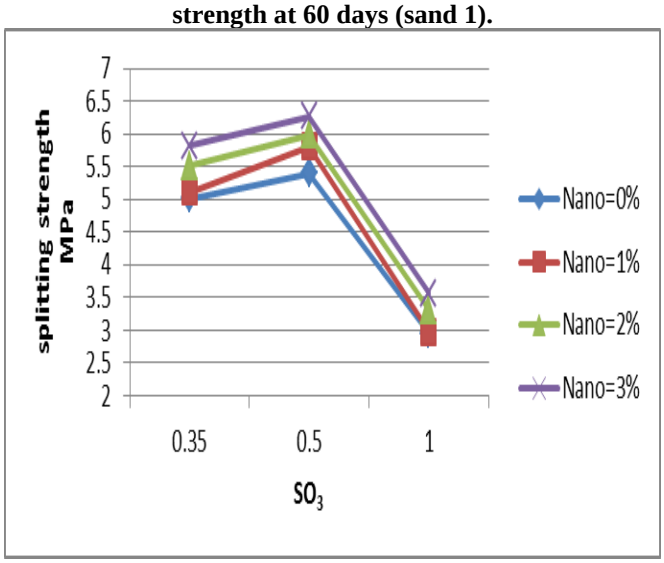


Figure (2-d) Effect of increasing SO_3 content on splitting strength at 60 days (sand 2).

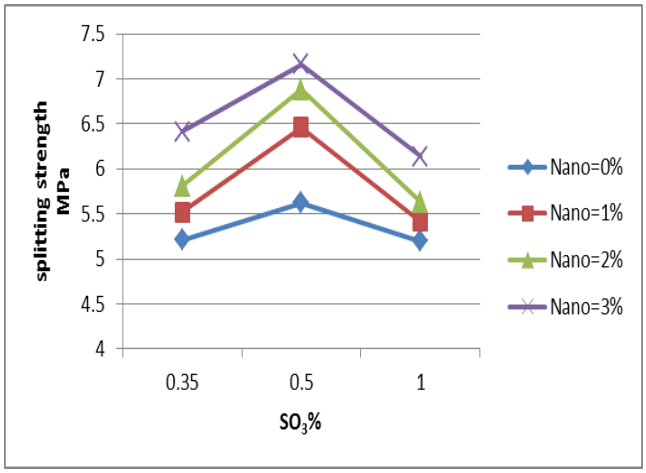


Figure (2-e) Effect of increasing SO_3 content on splitting strength at 90 days (sand 1).

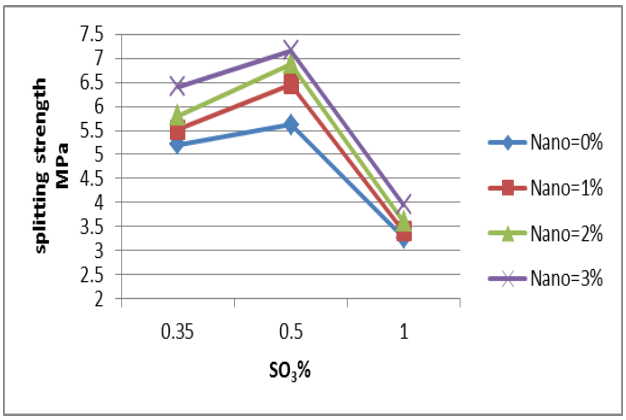


Figure (2-f) Effect of increasing SO_3 content on splitting strength at 90 days (sand 2).

Similar behavior as in compressive and tensile strength were observed for flexural strength i.e. there was optimum SO_3 of 0.5which gave optimum value of flexural strength. Adding NMK enhanced the resistance of SCC mixes against sulfates attack. Natural SO_3 sand gave less resistance than artificial SO_3 sand by about 15%. Fig. (3) shows the results of flexural strength at 28, 60 and 90 days age. It was clear from the results that mixes of sand with natural SO_3 gave flexural strength less than that with sand artificial SO_3 and the decreasing percent decreases with adding Nano meta kaolin and with age.

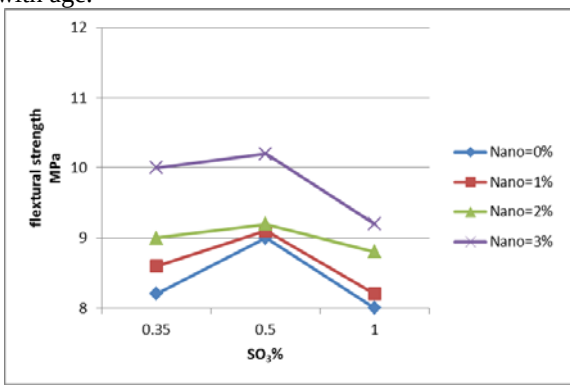


Figure (3-a) Effect increasing SO_3 content on flexural strength at 28 days (sand 1).

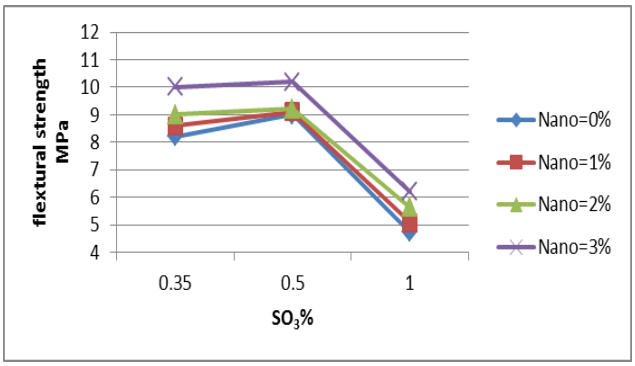


Figure (3-b) Effect increasing SO_3 content on flexural strength at 28 days (sand 2).

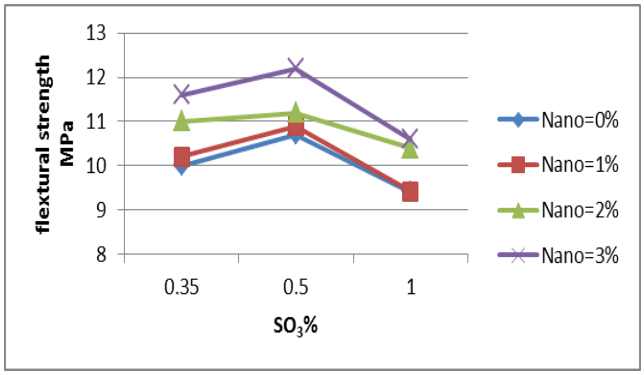


Figure (3-c) Effect increasing SO_3 content on flexural strength at 60 days (sand 1).

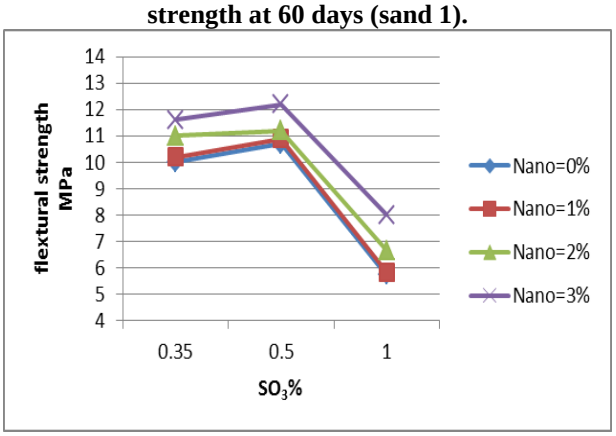


Figure (3-d) Effect increasing SO_3 content on flexural strength at 60 days (sand 2).

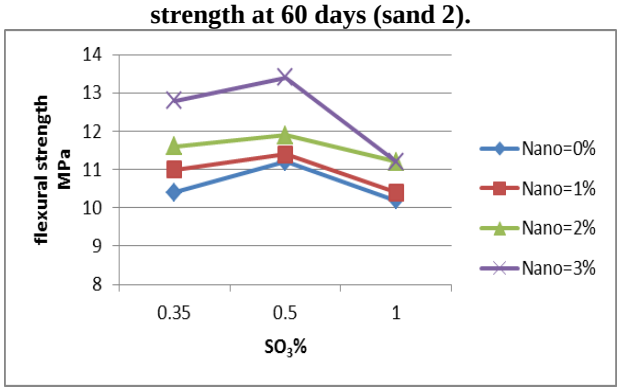


Figure (3-e) Effect of increasing SO_3 on flexural strength at 90 days (sand 1).

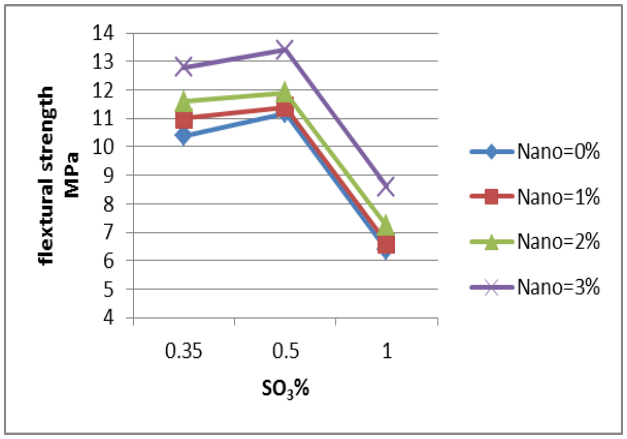


Figure (3-f) Effect of increasing SO_3 on flexural strength at 90 days (sand 2).

It can be observed from the results that SCC tends to have lower modulus of elasticity as shown in Fig.(4), the static modulus of elasticity of SCC specimens ranged between (22.35 – 41.11) GPa. This behavior was due to the high content of paste and less content of aggregate at SCC ***(EFNARC, 2005)***. The results indicated that the optimum SO_3 content for mixes was about 0.5 which is similar as in compressive and tensile strength beyond this optimum value the modulus of elasticity decreased with the increase of sulfates content for all mixes, at all ages of test. The results indicated also that all specimens exhibited a continuous increase in modulus of elasticity with progress in age. Also, it was clear from the results that the modulus of elasticity increased with adding Nano metakaolin at all mixes and at all ages. The results indicated that mixes with natural SO_3 sand gave modulus of elasticity less than that with sand of artificial SO_3.

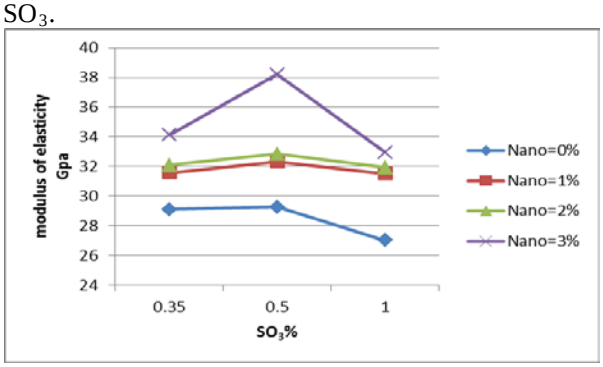


Figure (4-a) Effect increasing SO_3 content on modulus of elasticity at 28 days (sand 1).

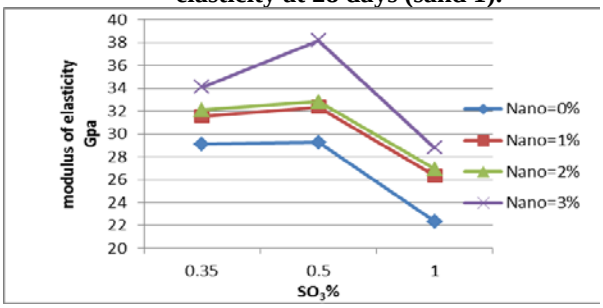


Figure (4-b) Effect increasing SO_3 content on modulus of elasticity at 28 days (sand 2).

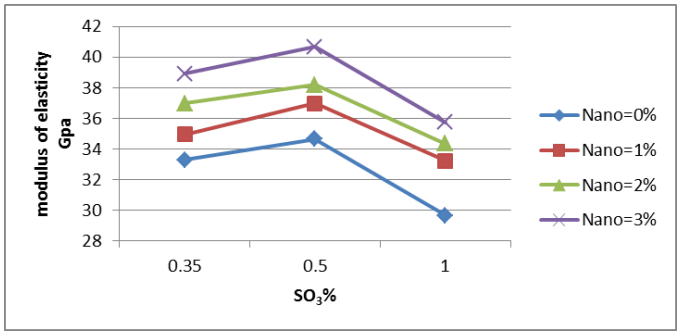


Figure (4-c) Effect increasing SO_3 content on modulus of elasticity at 60 days (sand 1).

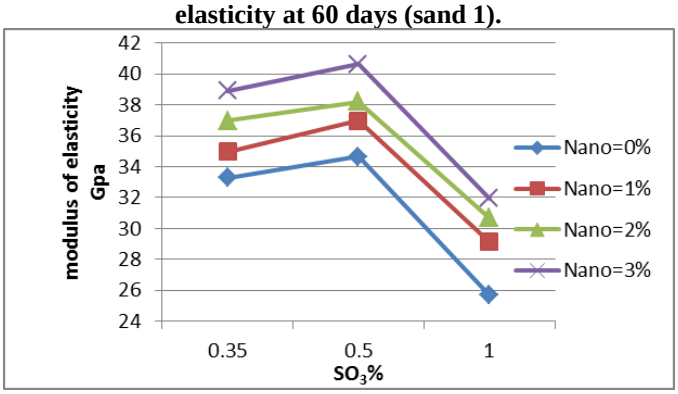


Figure (4-d) Effect increasing SO_3 content on modulus of elasticity at 60 days (sand 2).

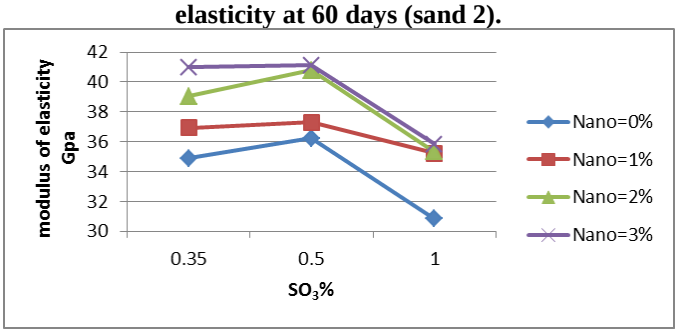


Figure (4-e) Effect increasing SO_3 content on modulus of elasticity at 90 days (sand 1).

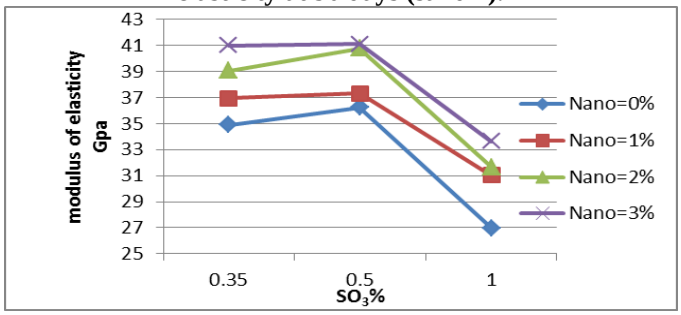


Figure (4-f) Effect increasing SO_3 content on modulus of elasticity at 90 days (sand 2).

7-Non Destructive Test

7-1 Length change test

The results of length change at 28, 60 and 90 days as shown in Fig.(5) indicated that expansion was increased with increasing sulfate content at all mixes (with and without Nano meta kaolin) and at all ages**.** This was in agreement with ***(Allahverdi et al., 2011)*** who found that linear expansion was increased with time in all mixtures. Sulfate attack is the destructive process acting on concrete due to the formation of expansive reaction products within concrete exposed to sulfate sources. Ettringite formation, expansive product, is associated with expansion. Continuous ettringite formation after concrete hardens is harmful to the concrete structure ***(Al-Rawi, 1985)***. The expansion decreased with increasing Nano meta kaolin is directly related to the pozzolanic reaction between Nano meta kaolin and calcium hydroxide released during the hydration of cement which consumes part of calcium hydroxide. Thus, the quantity of expansive gypsum formed by the reaction of calcium hydroxide with sulfate ions will be less. It was clear from the results that mixes with sand natural SO_3 gave expansion more than mixes with the sand artificial $SO_3$and this increasing in expansion percent decrease with adding Nano meta kaolin and with age.

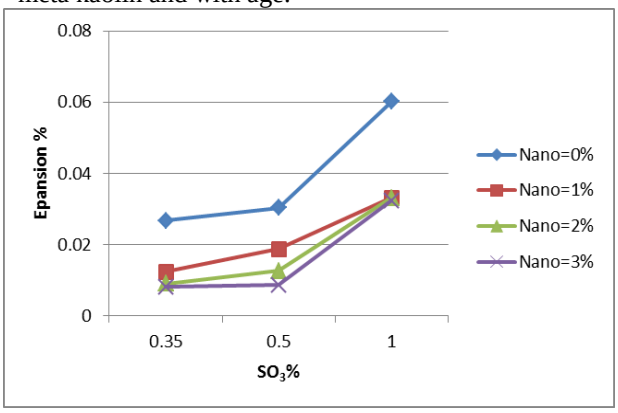


Figure (5-a) Effect increasing SO3 content on expansion at 28 days (sand 1).

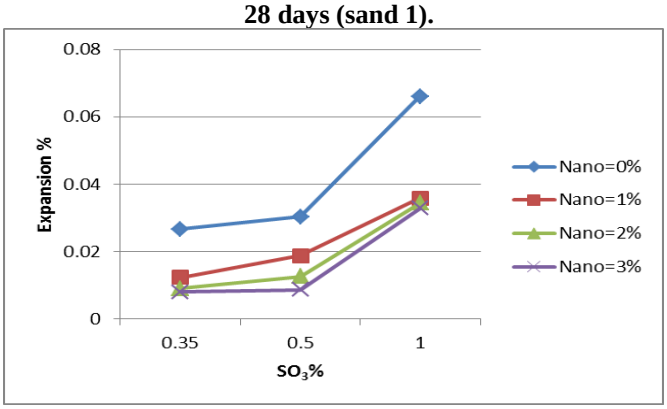


Figure (5-b) Effect increasing SO_3 content on expansion at 28 days (sand 2).

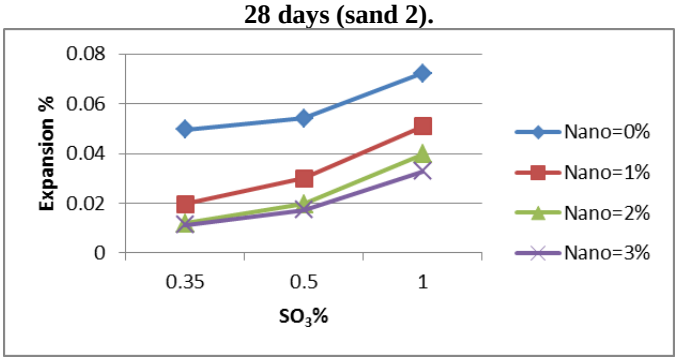


Figure (5-c) Effect increasing SO_3 content on expansion at 60 days (sand 1).

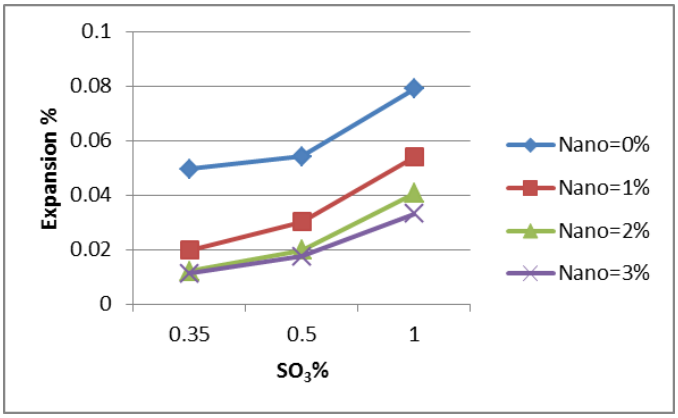


Figure (5-d) Effect increasing SO_3 content on expansion at 60 days (sand 2).

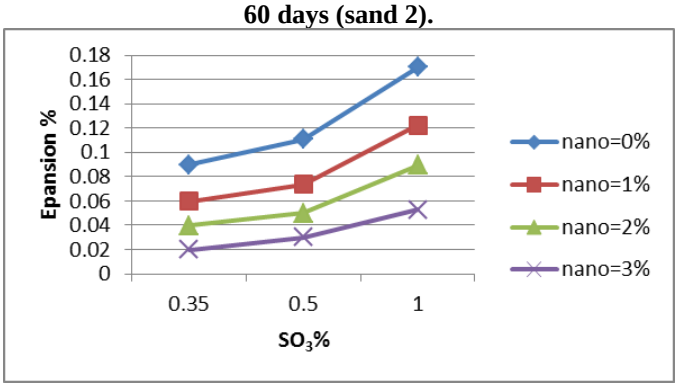


Figure (5-e) Effect increasing SO_3 content on expansion at 90 days (sand 1).

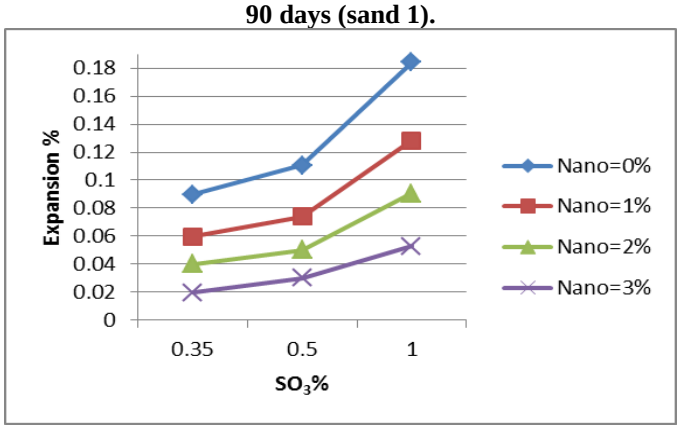


Figure (5-f) Effect increasing SO_3 content on expansion at 90 days (sand 2).

7-2 Ultrasonic Velocity Test(UPV)

Fig.(6) shows the results of ultrasonic velocity at 28, 60 and 90 days gained from cubes 150 mm. It was clear from the results that the ultrasonic velocity increased with adding Nano metakaolin at all mixes and at all ages. This behavior may be due that the concrete with Nano metakaolin was constructed by high density and filled the pores of the cement matrix with pozzolanic activity; this makes the pulse velocity increases for self- compacting concrete. It was clear from the results that mixes with sand natural SO_3 gave ultrasonic velocity less than that with sand artificial SO_3 and the decreasing percent decreased with adding Nano metakaolin and with age.

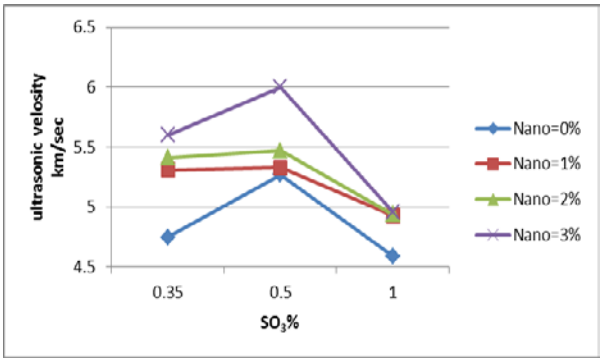


Figure (6-a) Effect increasing SO_3 content on ultrasonic velocity at 28 days (sand 1).

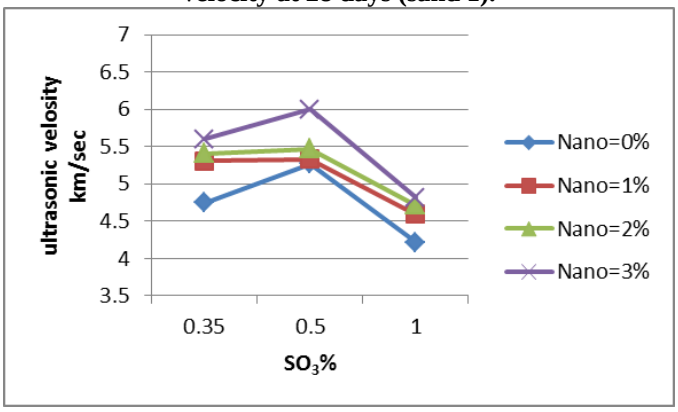


Figure (6-b) Effect increasing SO_3 content on ultrasonic velocity at 28 days (sand 2).

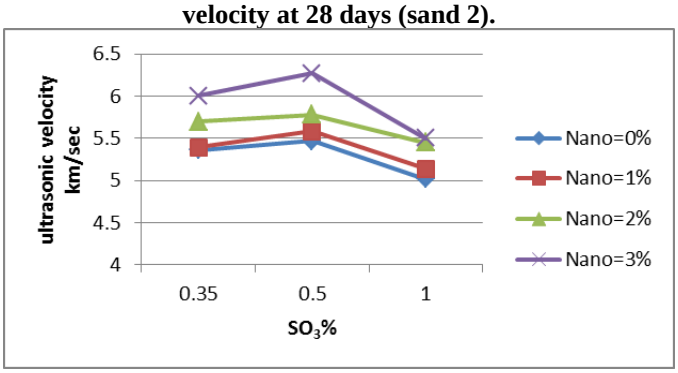


Figure (6-c) Effect increasing SO_3 content on ultrasonic velocity at 60 days (sand 1).

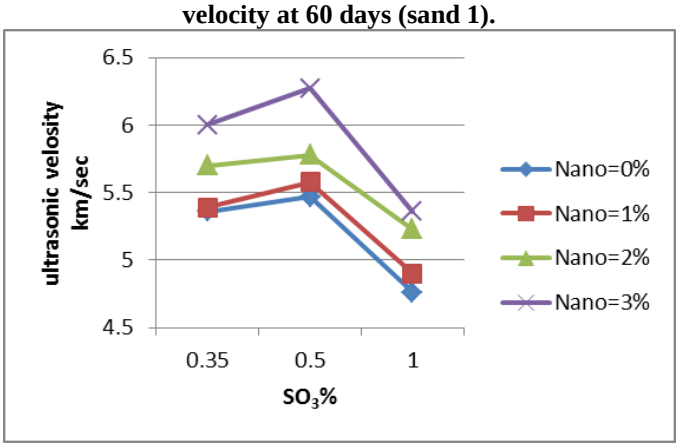


Figure (6-e) Effect increasing SO_3 content on ultrasonic velocity at 60 days (sand 2).

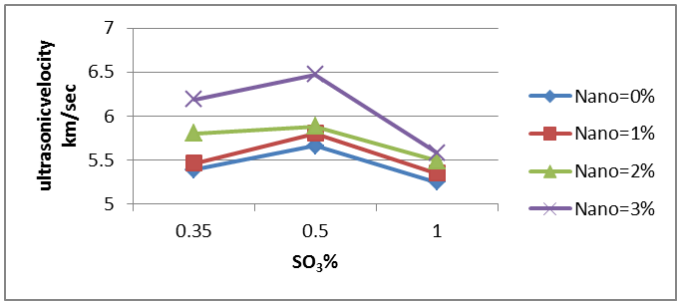


Figure (6-e) Effect increasing SO_3 content on ultrasonic velocity at 90 days (sand 1).

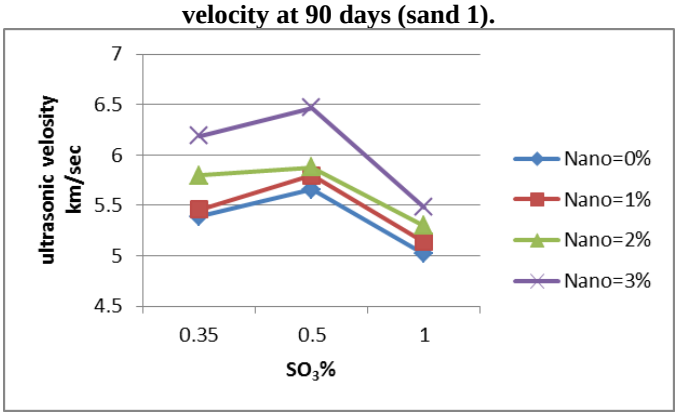


Figure (6-e) Effect increasing SO_3 content on ultrasonic velocity at 90 days (sand 2).

8-Conclusions

1. The optimum SO_3 content, at which a higher mechanical properties and little tendency to the expansion were equal to 0.5 (% by weight of fine aggregate).
2. Expansion was increasing with the increasing sulfate content for all mixes, at all ages and there was not optimum SO_3 content but the expansion in mixes at 0.5% SO_3 content was close to reference mixes (mixes with 0.35% SO_3 content).
3. Mixes with sand of natural SO_3 gave compressive strength, splitting strength, flexural strength, modulus of elasticity and ultrasonic velocity less than that with sand artificial, but Mixes with sand of natural SO_3 give expansion more than that with sand artificial.
4. The change in mechanical properties and non-distractive tests between sand of artificial SO_3 and sand of natural SO_3 reduces with adding Nano meta kaolin and with age.
5. The self-compacting concrete gives improvements in the mechanical properties and non-destructive tests with increasing in Nano meta kaolin content.

9-References

[1]Al-Harbi, Mowaffaq and others *"the Percentage of Sulfate Salts in the Concrete and Modify the Specification of the Iraqi 45"*, the Ministry of Housing and Construction, the National Center for laboratory construction, Baghdad, in November 1993 (Translated from Arabic)

[2]Allahverdi, A., Kani, E.N., and Fazlinejhad, M., "Investigating The Linear Expansion, Set, and Strength Behaviors of The Binary Mixture; Portland Cement Clinker-Gypsum" Iranian Journal of Materials Science & Engineering Vol. 8, Number 4, December 2011.

[3]Al-Rawi R.S., “Internal sulfate attack in concrete related to gypsum content of cement with pozzolan addition“, ACI-RILEM, Joint Symposium, Monterey, Mexico, pp. 543, 1985.

[4]Al-Robayi, A., "Resistance of High Performance Concrete to External and Internal Sulfate Attack", M.Sc. Thesis, University of Technology, October, pp.107(2005).

[5]Alsallami Zainab, "The Effect of Sulfate in Sand on Some Mechanical Properties of Nano Meta kaolin Self compacting concrete", M.Sc. Thesis, Babylon University, Iraq, 2013.

[6ASTM C150 "Standard Specification for Portland Cement", American Society for Testing and Materials ,(2005).

[7]ASTM C157/C157M "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete", American Society for Testing and Materials (2004).

[8]ASTM C469 "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression", American Society for Testing and Materials, (2002).

[9]ASTM C490 "Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete", American Society for Testing and Materials, (2004).

[10]ASTM C496 "Standard Test Method for Splitting Tensile Strength for Cylindrical Concrete Specimens", American Society for Testing and Materials, (2004).

[11]ASTM C597 "Standard Test Method for Pulse Velocity Through Concrete", American Society for Testing and Materials, (2002).

[12]ASTM C618 "Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete", American Society for Testing and Materials, (2002).

[13]ASTM C78-84"Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Two Points Loading)", Annual Book of ASTM Standard, Vol. 04.02, 2003.

[14]*BS 1881: Part 116, (1989), "Method for Determination of Compressive Strength of Concrete Cubes", British Standards Institution, (2003).

[15]Hussain, T.H., "Effect of sulfates in fine aggregate on some mechanical properties of self-compacting concrete", M.Sc. Thesis, University of Babylon (2008).

[16]EFNARC: The European Federation of Specialist Construction and concrete System "The European

Guidelines for Self-compacting concrete, Specifications ,production and use", may, 2005, PP.63.
[17]Iraqi Specification, No.45/1984, "Aggregate from Natural Sources for Concrete and Construction".
[18]Mahmoud, R.H., "Effect of Sulfate on the Properties of Self-Compacting Concrete Reinforced by Steel Fiber" M.Sc. Thesis, University of Babylon(2012).
[19]Morsy, M.S., Al-Sayed, S.H., and Aqel, M., "effect of Nano-clay on Mechanical properties and Microstructure of ordinary Portland cement Mortar", International Journal of Civil & Environmental Engineering IJCEE –IJENS VOL: 10N:01, Egypt(2008) .
[20] Zelic J., rusic, D., veza, D., krstulovic, R"The role of silica fume in the kinetics and mechanisms during the early.

Effect of Age on Concrete Core Strength Results

Asst. Lec. Mushtaq Sadiq Radhi, Prof. Dr. Shakir Ahmed Al-Mishhadani , and Asst. Prof. Dr. Hasan Hamodi Joni.

***Abstract*— Assessment of in-situ concrete strength by means of cores cut from hardened concrete is accepted as the most common in-situ nondestructive method, however the assessing of the concrete in the existing buildings, particularly in the troubleshooting of problems with new construction, If the strength of standard compression test specimens found to be below the specified 28 days value, frequently, cores tests are undertaken at later ages exceeding the 28 days. This study includes an attempt to find the influence of the long-term concrete age and strength level on the compressive strength development for the standard concrete core.**

This study involves laboratory investigation were number of specimens including concrete panels and cubes with specified compressive strength ranging from 25-55 MPa were prepared and tested at concrete age of 28, 60, 90, 120, 180,and 270 days by in-situ nondestructive tests (cores) and destructive tests (cubes). The test results obtained from core specimen were compared with those of standard specimens.

The test results showed that the core compressive strength increases as the age of concrete increase, but the core strength is somewhat higher than 28-day cube compressive strength even up to the age 270 days in moderate concrete, while the core compressive strength remains lower than 28-day cube compressive strength in the higher strength level even up to the age 270 days.

***Index Terms*—**Compressive strength, Concrete cores, Concrete age, Strength level.

I. Introduction

The strength of concrete is traditionally characterized by the 28 days value. However, strength of concrete is expected to increase with time at continuously diminishing rate. Knowledge of the strength-time relationship is of importance when a structure is subjected to certain type of loading at a later age. Many factors can significantly influence the compressive strength of the concrete. These include cement type, water-cement ratio, aggregate content, water curing period, and exposure conditions [1].

Asst. Lec. Mushtaq Sadiq Radhi is with College of Engineering, Civil Engineering Department, University of Kerbala, Kerbala, Iraq (phone: 009647801136645; e-mail: mushtaq79811@ yahoo.com).
Prof. Dr. Shakir Ahmed Al-Mishhadani is with the Building and Construction Engineering Department, University of Technology, Baghdad, Iraq .
Asst. Prof. Dr. Hasan Hamodi Joni is with the Building and Construction Engineering Department, University of Technology, Baghdad, Iraq .

Locally, the in-situ nondestructive tests like core compressive strength, ultrasonic pulse velocity, and rebound hardness hammer are widespread in the assessing the concrete in the existing buildings, particularly in the troubleshooting of problems with new construction, if the strength of standard compression test specimens found to be below the specified 28 days value, frequently, these nondestructive tests are undertaken at later ages exceeding the 28-days.

One of the well-known in situ nondestructive tests to estimate compressive strength of the concrete is core test. Core test is done by extracting cylindrical usually from existing concrete structure using a rotary cutting tool with diamond bits. Core testing is the most direct method to determine the compressive strength of concrete in a structure. Generally, cores are obtained either to assess whether suspect concrete in a new structure complies with strength-based acceptance criteria or to evaluate the structural capacity of an existing structure based on the actual in-place concrete strength. In either case, the process of obtaining core specimens and interpreting the strength test results is often confounded by various factors that affect either the in-place strength of the concrete or the measured strength of the test specimen [2]. The measured core compressive strength is mainly affected by the following factors [2-5] Moisture condition, Length-diameter ratio of core, Diameter of core, Direction of drilling, Effect of Age, Microcracking, and Reinforcement.

In-situ strength is the strength of the concrete as it exists in the element at the time of sampling and is the end result of the quality materials characteristics, construction techniques, workmanship and exposure. Tapkin *et al.* [6] were made a study to examine a method to evaluate concrete core strengths by using artificial neural networks. Eight different concrete mixtures were prepared by using two different aggregates of four different maximum sizes. Beam specimens were cast by prepared mixtures. Cores with different diameters and length-to-diameter ratios were drilled from beam specimens. Compressive strength tests were carried out on core specimens at different ages. The parameters influencing the strength of cores were used as input for neural network architecture and the core strengths were evaluated. The core strength test results were analyzed by means of multi-layer feed forward back propagation neural network model. The authors concluded that the average compressive strengths of concrete cores determined by the artificial neural networks and by destructive tests during the investigation were very similar to each other. It

indicates that the estimations were representative of the real results.

ACI 318 [7] and ACI 301[8] stated that concrete shall be considered adequate (as specified) when the average of three cores is equal to at least 85% of the specified strength of concrete and no single core is less than 75% of the specified strength of concrete .No allowance for age or curing history is mentioned. The value of 85% in ACI 318 is not very different from the value used in BS 6089[9], which require the "estimated in-situ cube strength" to be 83% of the specified characteristic strength of concrete, to which the partial safety factor for design strength is applied.

Bartlett and MacGregor [10] submitted a relationship between the average *in situ* strength and the specified strength is determined from the analysis of result test data. The average *in situ* strength at 28 days, as corrected for the effect of the core moisture condition and damage sustained during drilling, approximately equals the average cylinder strength. The average cylinder strength is usually greater than the specified strength, because the concrete producer desires assurance that the product will meet specifications. The relationship obtained by regression analysis between the average *in situ* strength at 28 days and the specified strength is:

$$f_{c,is} = [1.205 + 0.108 Z_n] f_c$$

where Z_n, is an indicator variable equal to 0 for elements less than 450 mm high or 1 otherwise

This study includes an attempt to find the influence of the long-term concrete age on concrete properties by using the well known locally in-situ nondestructive test method core compressive strength. Many researches have been conducted to assess or expect the mechanical properties of normal concrete using nondestructive tests. Though, locally little work has been published about the influence of the long-term concrete age and strength level on the compressive strength development for the standard concrete core.

II. Experimental Study

In order to achieve the aim of this study, three concrete mixes with 28 days specified strength ranging from 25 to 55 MPa were produced by using ordinary Portland cement manufactured by united cement company commercially known (Tasluja-Bazian) , AL-Ekhaider natural sand of 4.75mm maximum size with grading limited zone 2, Crushed gravel of 20 mm maximum size from Al-Nebai quarry and Tap water and superplasticizer admixture. Three mixes are prepared according to Building Research Establishment method, the mixes were designed to have a 28 days potential compressive strength of 25, 40, and 55 MPa. According to the design and trial mixes the cement content was 350, 400, and 450 kg / m3, while the water-cement ratio 0.5, 0.45, and 0.35 respectively .The details of the mixes used throughout the laboratory work are given in Table 1. By using these mixtures, 100 mm cube specimens were cast and water cured until being tested. An average of three specimens was considered for each age. Also 1000×1000×250 mm unreinforced concrete slabs were cast and moist cured for 7 days. 100 mm diameter cylindrical core specimens were extracted from the slabs at different ages, prepared, and tested at the same age of the cubes (The general procedures for core cutting and testing are demonstrated on the following headlines, Drilling, Trimming, Core Capping, and Compression testing..

TABLE (1) DETAILS OF THE MIXES USED THROUGH THIS INVESTIGATION.

Mix	Cement Content (kg / m^3)	Fine Aggregate (kg / m^3)	Coarse Aggregate (kg / m^3)	Water Content (kg / m^3)	Water-Cement Ratio	SP % by wt. of cement
C25	350	800	1040	192.5	0.55	-
C40	400	665	1135	180	0.45	-
C55	450	610	1140	157.5	0.35	1.5

III. Results and Discussion

The core compressive strengths developments for all mixes moist cured for 7 days schemed in Figure (1). Depending on the test results, it can be seen that the core compressive strength of all mixes increases gradually with the progress of concrete age due to the continuity of hydration process in concrete in the outdoor circumstances. The relative humidity in the climate, the rain falls, and the lubricating water during drilling cores supplied the concrete core panels with additional water for curing. The form of the core compressive strength increasing is comparable to the compressive strength growing.

Figure (2) shows the normalized values of core compressive strength, all the values were divided by the corresponding 28 days values to make the 28 days values equal to unity. The percent of increase at age of 90 days are 37%, 22%, and 14% for C25, C40, and C55 respectively. The percent of increase at age of 180 days are 46%, 31%, and 18% for C25, C40, and C55 respectively. It is clear that mixes with higher water-cement ratio and lower cement content higher in strength development above the 28 days core compressive strength.

Figure (3) shows the proportion of core compressive strength at different ages to the corresponding 28 days cube compressive strength. The proportions for C25 are 71.38 %, 88.41 %, 98.54 %, 101.15%, 103.46 %, and 107.78% at core ages of 28, 60, 90, 120, 180 and 270 days respectively. While the proportions for C40 are 68.64 %, 79.69 %, 83.8 %, 87.92%, 89.72%, and 92.54% at core ages of 28, 60, 90, 120, 180 and 270 days respectively . Also the proportions for C55 are 58.76 %, 64.07 %, 66.73 %, 67.61%, 69.38 %, and 73.81% at core ages of 28, 60, 90, 120, 180 and 270 days

respectively . Although the strength of core increases with age, but the rate of increase is very low when compared with 28 days compressive strength. This can be explained by the combined effect of various factors on core strength in unpredictable degrees, they are: -

(a) The difference in compressive strength between the cube and the cylinder, according to BS 1881 part 120 [11] the compressive strength of cylinder (standard core) is about 0.8 of the compressive strength of a cube. However there is no simple relation between the strengths of the specimens of the two shapes. Shetty[12] has confirmed that the ratio of compressive strength cylinder to cube specimens increases significantly with the increase of their strengths and become nearly 1 in high strength concrete .

(b) Destruction of drilling operation, where the drilling operation weakens the bonds between the aggregate and the surrounding hardened cement paste. Also in high strength concrete, the bonds between cement paste and aggregate are higher and more cohesive. This results more resistance during the coring operation and causes greater shearing between the coring bit and the concrete surface, which would cause greater damage to higher strength concrete as compared to low strength concrete.

(c) The difference in exposure conditions and curing between the core and the standard specimens. Panels of core were moist cured for 7 days, while standard specimens were moist cured up to the age of test. Petersons, cited by Neville [1], stated that the ratio of core strength to standard cylinder strength is always less than 1, and decreases with an increase in the concrete strength level. Approximate values of this ratio are: just under 1 when the cylinder compressive strength is 20 MPa and 0.7 when it is 60 MPa. Neville [1] identified a difficulty in separating out the effect of drilling operation and curing history effect. The difficulty is exacerbated by the fact that the exact curing history of a structure is, usually difficult to determine so that the effect of curing on the strength of cores is uncertain, and he suggested a reduction of drilling between 5-7%.

(d) The consequence of soaking the core in water causes the reduction in compressive strength. Bartlett and MacGregor [13] found that, on average, the strength of cores dried in the air for 7 days is 14% higher than the strength of cores soaked in water for, at least, 40 hours. Another research carried out by Bartlett and MacGregor [14]. They observed a more severe strength loss in 50 mm diameter cores compared with 100 mm diameter cores from the same element. In addition, they found that extending the soaking period beyond 40 hours duration could cause further reduction of the core strength.

It is agreed that as the concrete compressive strength increases, the core compressive strength is also increased but at a decreasing rate. A number of factors affect the relation between the two strengths such as diameter, length/diameter ratio and moisture condition of the core specimen, the direction of drilling, presence of reinforcement steel bars in the specimen and even the strength level of the concrete.

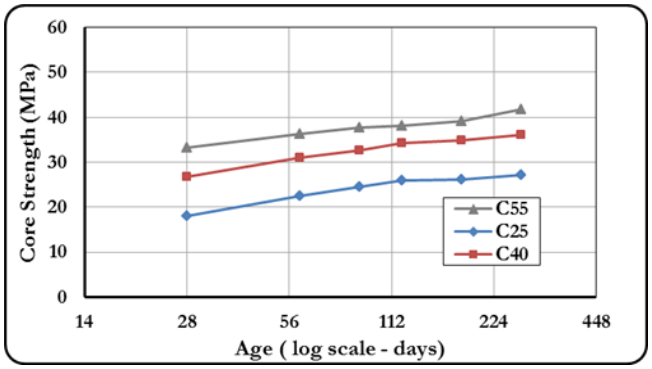


FIGURE (1) THE DEVELOPMENT OF THE CONCRETE CORE STRENGTH WITH AGE.

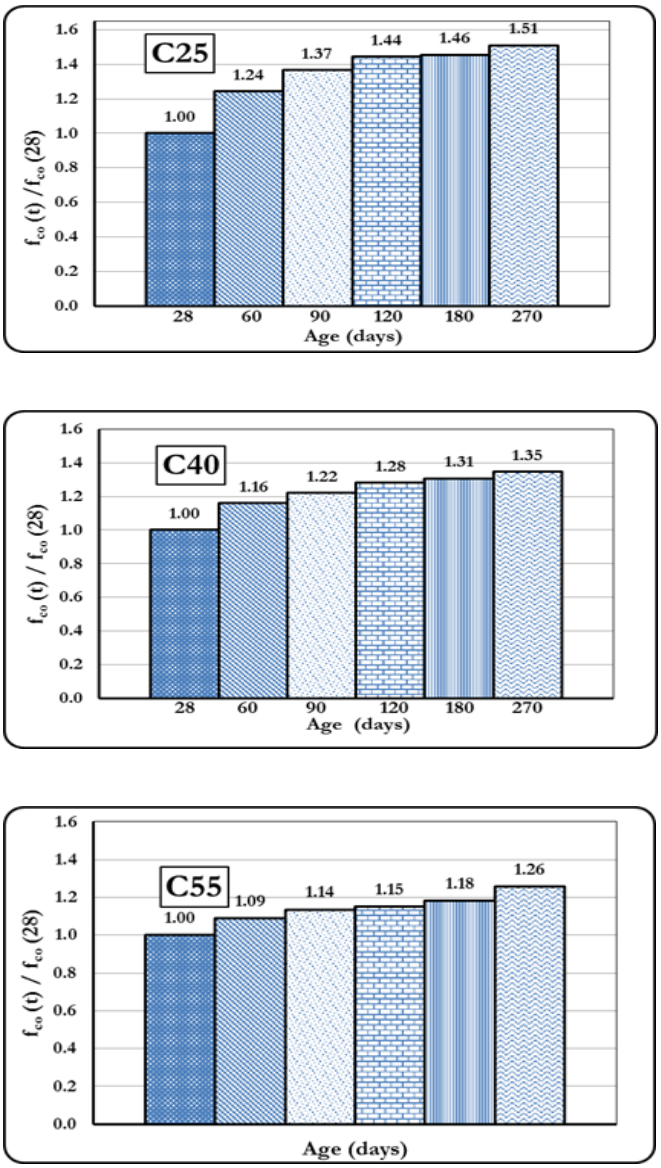
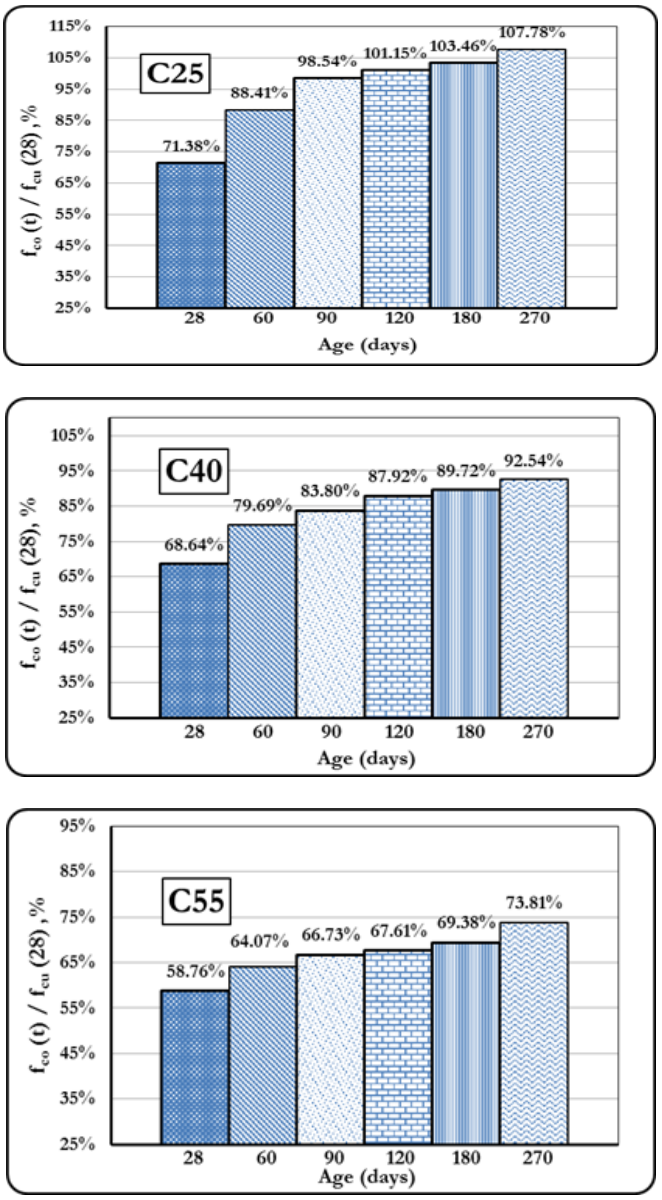


FIGURE (2) VARIATION OF NORMALIZED CONCRETE CORE STRENGTH WITH AGE FOR ALL MIXES.



FIGURE(3) CORE STRENGTH TO 28 DAYS COMPRESSIVE STRENGTH RATIO FOR ALL MIXES AT DIFFERENT AGES.

IV. CONCLUSION

Based on the results of this investigation, the conclusions that can be drawn are given below:

1. The strength of all concrete mixes, tested by destructive or nondestructive methods, increases with age in diminishing rate.
2. The water-cement ratio of concrete mix is a significant factor that affects on the rate of development of strength with age. Concrete mixes with low water-cement ratio achieved the long-term values more rapidly than concrete mixes with high water-cement ratio. The higher water-cement ratio and the higher in the development above the normalized 28-day values. The development of compressive strength for mix with w/c 0.35 (C55), and w/c 0.55(C25) at age 120 days is 95 %, and 90 % respectively, expressed as percentage of the 270 days corresponding values. The development of compressive strength for the same above mixes at age 120 days is 17 %, and 87 % respectively, when expressed as percentage of the 28 days corresponding values.
3. The core compressive strength increases as the age of concrete increase, but the core strength is somewhat higher than 28-day cube compressive strength even up to the age 270 days in moderate concrete (C25), while the core compressive strength remains lower than 28-day cube compressive strength in the higher strength level (C40 and C55) even up to the age 270 days. This means that the core strength is affected by the other factors (strength level, curing conditions, and moisture testing conditions) more than age of core. So that no age adjustment should be used in the assessment of the strength of core especially in the lack of definite moist curing.

REFERENCES

[1] Neville, A. M., *"Properties of Concrete,"* Fourth and Final Edition Standards updated to 2002, Pearson, 2005,844 p, printed in Malaysia.

[2] Neville, A. M., *"In My Judgment, Core Tests: Easy to Perform, Not Easy to Interpret,"* Concrete International, Vol.23, No.11, November 2001, pp.59-68.

[3] Bungey, J. H., Millard, S. G., Grantham, M. G., *"Testing of Concrete in Structures,"* Taylor and Francis, 4th edition, 2006, pp.35-139.

[4] ACI 214.4R-03, *"Guide for Obtaining Cores and Interpreting Compressive Strength Results,"* Reported by ACI Committee 214, ACI Manual of Concrete Practice, American Concrete Institute, 2004,16 p.

[5] ASTM C42-03, *"Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete,"*Annual Book of ASTM Standards American Society for Testing and Materials, Vol.4.2, 2006, 5 p.

[6] Tapkin, S., Ariöz,O., Tuncan,M., Tuncan,A. and Ramyar,K., *"Use of Neural Networks for The Evaluation of Concrete Core Strengths,"* 4th Faculty of Architecture and Engineering International Symposium, European University of Lefke, Turkey, 2006 , pp.195-202.

[7] ACI 318M-08, *"Building Code Requirements for Structural Concrete,"* Reported by ACI Committee 318, ACI Manual of Concrete Practice, American Concrete Institute, 2008.

[8] ACI 301M-99, *"Specifications for Structural Concrete,"* Reported by ACI Committee 301, ACI Manual of Concrete Practice, American Concrete Institute, 2004, pp.9.

[9] BS1881: part 6089:1981, *"Guide to Assessment of Concrete Strength in Existing Structures,"* British Standards Institution, 1999, 15 p.

[10] Bartlett, F. M., and MacGregor, J. G., *"Equivalent Specified Concrete Strength from Core Test Data,"* Concrete International, Vol. 17, No. 3, Mar.1995, pp. 52-58.

[11] BS1881:part120:1983, *"Method for Determination of The Compressive Strength of Concrete Cores,"* British Standards Institution, 1983, 5p.

[12] Shetty, M.S., *"Concrete Technology, Theory and Practice,"* 6th multicolour edition, 2009, pp.428.

[13] Bartlett, F. M., and MacGregor, J. G., *"Effect of Moisture Condition on Concrete Core Strengths,"* ACI Materials Journal, Vol. 91, No. 3, May-June 1994, pp. 227-236.

[14] Bartlett, F. M., and MacGregor, J. G., *"Cores from High Performance Concrete Beams,"* ACI Materials Journal, Vol. 91, No. 6, Nov.-Dec.1994, pp. 567-576.

Flexural Behavior of High Strength Hybrid Fiber Lightweight Aggregate Concrete

K. Wasan Ismail and M. Suhad Abd Al-Jabbar

Abstract— **The aim of this study is to improve the flexural behavior of high strength porcelinte lightweight aggregate concrete by incorporating hybrid fibers in different dimensions and types. High strength lightweight aggregate concrete (HSLWAC) mix with compressive strength 41 MPa at 28 days age was prepared. The fibers used include macro hooked steel fiber with aspect ratio of 100 (type S1), macro hooked steel fiber with aspect ratio 60 (type S2), micro polypropylene fiber (pp) and micro carbon fiber (CF). Seven HSLWAC mixes were prepared including, one plain concrete mix (without fibers), two mono fiber reinforced concrete mixes (1% volume fraction of steel fiber type S1 and 0.25% carbon fiber) and four double hybrid fiber reinforced concrete mixes [0.5% steel fiber type S1+ 0.5% steel fiber type S2 (HSF1), 0.75% steel fiber type S1+ 0.25% steel fiber type S2(HSF2), 0.75% steel fiber type S1+ 0.25% pp fiber (HSPPF) and 0.75% steel fiber type S1+0.25% CF (HSCF)].**

The experimental results indicate that the inclusion of fibers significantly increases the flexural strength in mono and hybrid fiber mixes. The percentage of increase in modulus of rupture for mono fiber HSLWAC containing 1% steel fiber type S1 and 0.25% carbon fiber are 29.52% and 60.64% respectively, while the percentage of increase for hybrid mixes HSF1, HSF2, HSPPF and HSCF are 36.4%, 61.56%, 34% and 65.43% respectively. The results also show that the inclusion of fibers improves the load-deflection behavior and consequently enhances the flexural toughness of HSLWAC. Specimens prepared from hybrid fiber HSLWAC containing a combination of 0.75% steel fiber type S1+ 0.25% steel fiber type S2 (mix HSF2) show the highest flexural toughness.

Keywords: **Flexural Toughness, Hybrid Fibers, Macro Fiber, Micro Fiber, Mono Fibers.**

I. Introduction

Lightweight concrete (LWC) is one of the most interesting subjects for researchers because of its advantages such as the reduction in concrete member size, reinforcement, formwork and scaffolding, foundation costs as well as the savings derived from the reduced cost of transport and erection[1]. The addition of fibers to concrete considerably improves its structural characteristics such as static flexural strength, impact strength, tensile strength, ductility and flexural toughness. The degree of improvement depends upon many factors such as type, size, aspect ratio and volume fraction of fibers [2]. In certain applications, such as long-span bridges and ocean platforms, the requirements of concrete performance are not only the advantage of lightweight, but also relatively higher toughness and higher durability, which its importance has been highlighted and emphasized recently. Some disadvantages of lightweight concrete, such as the low flexural strength, low tensile/compressive strength ratio, the low fracture toughness and high brittleness, restrict its further application and popularization in these structures [3]. The main objectives of the present work are to study the effect of incorporation of hybrid fibers on the flexural performance of high strength lightweight aggregate concrete.

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F. K. Wasan Ismail is with the University of Technology, Building and construction Engineering Department, Baghdad, Iraq.
(e-mail: wasan1959@yahoo.com).
S. M. Suhad Abd Al-Jabbar is with Baghdad Mayoralty.
(e-mail: suhad_mozan@yahoo.com).

II. Experimental Program

A. Materials

The cement used in this study was ordinary Portland cement (Type I). Test results indicate that the adopted cement satisfies the requirements of the Iraqi Specification No.5/1984. Al-Ekhaider natural sand of maximum size 4.75mm was used as fine aggregate. Its gradation lies in zone (2). The results show that sand grading; physical properties and sulfate content were within the requirements of the Iraqi Specification No.45/1984.

Local natural porcelinate stone was used as a coarse lightweight aggregate (LWA). The lumps were crushed manually to small pieces then screened on standard sieve series (12.5mm, 9.5 mm and 4.75mm), the maximum size of aggregate was 9.5mm. The aggregate prepared with grading conforms to ASTM C330-05 specifications. The aggregate was washed with water in order to remove the dust associated with crushing process of porcelinite stone and then spread inside the laboratory to have saturated surface dry (SSD) condition. Two types of concrete admixtures were used in this study:

A superplasticizer commercially known as **GLENIUM 51** was used throughout this work as a high range water reducing agent (sulphonated melamine and naphthalene formaldehyde type F). The dosage recommended by the manufacturer is 0.5-1.6 liters/100kg of cementations material. This type of admixture conforms to the ASTM C494-05 type F. Silica fume used in this investigation is commercially known as **MEYCO** from the Chemical Company **BASF**. The chemical composition and physical requirements show that the silica fume used in this investigation conforms to the chemical and physical requirements of ASTM C1240 Specifications.

Four types of fiber were used in this work including:

a) Macro hooked steel fiber with 50 mm in length and 0.5 mm in diameter (aspect ratio, l/d = 100), (type S1), with ultimate tensile strength for individual fibers of 1180 MPa and density 7800kg/m^3.

b) Macro hooked steel fibers with 30 mm in length and 0.5 mm in diameter (aspect ratio, l/d = 60), (type S2) with ultimate tensile strength for individual fibers of 1180 MPa and density 7800 kg/m^3.

c) Micro polypropylene fiber (PP) with 12mm in length, 18 micron in diameter (aspect ratio l/d= 677) and minimum tensile strength 350 MPa.

d) Micro carbon fiber (CF) (5mm in length and 7µm in diameter) with tensile strength 4300 N/mm^2.

B. Mixing of Concrete

The mixing process was performed in a pan type mixer of 0.1m^3 capacity. Initially, the silica fume and cement were mixed in dry state for about three minutes to disperse the silica fume particles throughout the cement particles, and then the sand and aggregates were added and mixed for two minutes. Seventy percent of the required amount of water was added to the mixer and mixed for about one minute, while thirty percent of the mixing water was added to the HRWRA. The solution was well stirred before used and then added gradually; the whole constituents were mixed for further two minutes. The mixer was stopped and mixing was continued manually especially for the portions not reached by the blades of the mixer. The mixer then operated for three minutes to attain reasonable fluidity. Fibers were uniformly distributed into the mix in three minutes, and then the mixing process continued for additional two minutes. In total, the mixing of one batch requires approximately 15 minutes ensuring uniform distribution of fibers.

C. Casting and Curing of Specimens

All moulds were well cleaned and their internal surfaces were lightly oiled. Then the molds were filled with the concrete mix in layers according to the specification for each specimen type. Each layer was compacted by an external table vibrator to minimize the air voids and to get well compacted concrete. The top surfaces of the moulds were leveled and the specimens were covered with nylon sheets to prevent the loss of moisture. After 24 hours, the specimens were demolded, marked and then cured by being completely immersed in water until the time of testing at age of 28 days.

D. Experimental Tests and Concrete Mixes

A number of experimental tests were carried out to in this study including, compressive strength test according to B.S.1881 part 116 (using cubes of 100 mm), ultrasonic pulse velocity according to ASTM C 597(using cubes of 100 mm), static modulus of elasticity according to ASTM C 469 (using cylinders of 150×300 mm), dry density test according to ASTM C 567 (using cylinders of 150× 300 mm), flexural tensile strength test, load- deflection relationship and flexural toughness, according to ASTM C 1609M (using 100×100×400 mm prisms).

High strength LWAC containing porcelinate coarse aggregate was prepared in previous research [4]. The mix proportion is 1: 1.35: 0.87 (cement: sand: aggregate) by weight with cement content 520 kg /m^3 and w/c of 0.29 which produce a slump of 90 mm. High range water reducing admixture (HRWRA) was used with 1 liter/100 kg of cement, 5% silica fume as a replacement by weight of cement was also used. Seven HSLWAC mixes were prepared; these mixes were classified to three groups as shown in Table (I).

TABLE I
PLAIN AND FIBROUS HSLWAC MIXES

Group	Mix Symbol	Fiber Volume Fraction %	Mix proportion by weight 1:1.35:0.87 (Cement: sand :LWA) With cement content 520 kg/m^3 w/c=0.29 HRWRA= 1L / 100kg of cement Silica fume 5% as replacement by weight of cement
1	0F	0	
2	1SF1	1% steel type S1	
	0.25CF	0.25 % Carbon Fiber (CF)	
3	HSF1	0.5% steel typeS1 + 0.5%steel typeS2	
	HSF2	0.75% steel typeS1+ 0.25%steel type S2	
	HS PPF	0.75% steel typeS1+ 0.25% PP	
	HSCF	0.75% steel typeS1 + 0.25%CF	

III. RESULTS AND DISCUSSIONS

A. Compressive Strength and Ultrasonic Pulse Velocity (UPV)

The compressive strength and UPV results of different HSLWAC studied in this investigation are shown in Table (II). Generally the results show that all hybrid fibers reinforced LWAC specimens have compressive strength higher than that reinforced with 1% volume fraction of mono steel fiber type S1. This is due to the fact that hybrid fibers with different sizes and types would offer different restraint conditions. Furthermore, this condition can be attributed to the improvement in the mechanical bond strength when the both fibers have the ability to delay the micro- crack formation and arrest their propagation afterward up to a certain extent [5, 6]. The pulse velocity increases marginally as strength of concrete increases and there is a greater interdependence between UPV and the compressive strength in LWC [7]. Generally the results show the same trend as in compressive strength. The pulse velocity of HSLWAC increases marginally in presence of fiber except for HSLWAC mix containing 1% volume fraction steel fiber (1SF). The increase in pulse velocity is because the pulse velocity in steel may be up to twice the velocity in plain concrete and the pulse travels partly in concrete and partly in steel [8]. The decrease in pulse velocity for mix containing 1% volume fraction steel fiber may be due to the presence of air voids in the matrix due to the high content of fibers. The UPV values for all HSLWAC mixes ranged between 3.30 and 4.35 km/ second. According to these values, HSLWAC prepared in

this investigation can be classified as a good quality concrete [9].

B. Static Modulus of Elasticity

The modulus of elasticity of LWC is 25-50% lower than that for normal weight concrete at the same compressive strength [10]. Table (II) and Fig.(I) show the results of static modulus of elasticity for all HSLWAC mixes. Concrete mix containing 1% steel fiber shows a decrease in modulus of elasticity of about 3% in comparison with the plain mix (without fibers). This may be due to the defects and voids which can form during placing high volume fraction of steel fibers as a result of improper consolidation. The results also illustrate that the maximum increase in modulus of elasticity is 24% for HSLWAC mix containing carbon fiber content of 0.25%. This is because the addition of small amount of carbon fiber not only increased the bearing capacity of cement composites, but significantly improved the ductility and Young's modules of mortar matrix which is related to increasing in compressive strength [11]. The modulus of elasticity for hybrid HSLWAC mix reinforced with steel and polypropylene fiber decreases by about 13.7% relative to the plain mix. This is due to the low modulus of elasticity for polypropylene fiber. Other hybrid fiber HSLWAC mixes show increase in modulus of elasticity in comparison with plain mix.

C. Modulus of Rupture (Flexural Strength)

The flexural strength test results of all HSLWAC mixes are presented in Table (II) and Fig. (II). The results show that the inclusion of fibers increases the flexural strength in mono and hybrid fiber mixes relative to plain mix. The reason for this improvement in modulus of rupture is that after matrix cracking, the fibers will carry the load that the concrete sustained until cracking by interfacial bond between the fibers and the matrix. Therefore, the fibers resist the propagation of cracks and do not fail suddenly, which causes an increase in load carrying capacity [12]. The flexural performance of fiber reinforced concrete can be improved by blending two or three different fibers together in a matrix, because these different fibers play a role at two different levels, material and structural, according to the type, length and diameter of fibers [13].

D. Load- Deflection Relationship

The flexural behavior of fiber reinforced cement composite (FRCC) is categorized into deflection hardening and softening behaviors according to the change in load carrying capacity after first cracking as shown in Fig.(III) [14]. There are several parameters described the flexural behavior of FRCC. The first cracking point of FRCC (P_{LOP}) is defined as first peak load (P_1) in ASTM C1609 [15]. The maximum equivalent bending strength point of FRCC is defined as modulus of rupture (MOR). In ASTM C 1609 it is notated as peak load (P_p) and the corresponding deflection is notated as δ_p. Load –deflection curves of all HSLWAC specimens are shown in Figures (IV) to (VI). These Figures show that plain concrete specimens (0F) have linear load-deflection relationship and fail directly at the peak load with no post-crack response. The load –deflection curves of fibrous HSLWAC specimens generally show elastic or uncracked stage at which the deflection increases almost linearly with load. Further increase in the load above the point of deviation from linearity, causes the first crack to occur. After the formation of the first crack the stresses in the matrix are progressively transferred to the fibers and the deflection slightly increases till the peak load, this is the quasi-elastic stage. Finally the third stage is the plastic stage which represents the failure of specimens due to the failure in fibers. It can be observed that for all fibrous reinforced HSLWAC specimens the peak load (P_p) or (P_{MOR}) is higher than the first peak load (P_1) or (P_{LOP}). The flexural behavior of fiber reinforced HSLWAC is classified as deflection hardening type. It can be observed that HSLWAC containing high content of macro steel fiber shows better flexural performance and load-deflection relationship is significantly enhanced. The combination of two macro steel fibers with 0.75% volume fraction of steel fiber type S1 and 0.25% volume fraction of steel fiber type S2 (mix HSF2) leads to significant improvement in load-deflection relationship as shown in Fig.(V). This is because steel fibers are the best of different fibers in offering better toughness and strength characteristics [16]. Researchers observed an amazing improvement in the bearing capacity when the fibers are hybridized in size/length. Fibers enhance a significant bearing capacity due to the combined effect of lengths and synergetic action of varying dimension. This is because the small fibers reinforce the mortar phase at the micro-crack stages and enhance the response during crack-initiation, whereas the large fibers provide the toughness at the stages of larger crack opening [17].

TABLE II
SOME PROPERTIES OF HSLWAC MIXES

Mix Symbol	Cube Compressive Strength (*fcu*) at 28 days (MPa)	Static Modulus of Elasticity (Ec) (GPa)	UPV (km/sec)	Oven Dry Density (kg/m^3)	Modulus of Rupture (fr) At 28 days (MPa)
0F	41.30	27.30	3.55	1930	3.84
1SF (1% S1)	38.95	26.60	3.30	2033	4.65
0.25CF (0.25% CF)	44.23	33.90	4.00	1944	5.76
HSF1 (0.5% S1+0.5% S2)	39.90	31.11	3.66	2078	4.97
HSF2 (0.75% S1+0.25% S2)	42.75	30.82	3.60	2070	6.14
HSPPF (0.75% S1+0.25% PP)	43.00	25.56	3.75	1998	4.86
HSCF (0.75S1+0.25CF)	45.60	32.25	4.35	2009	6.32

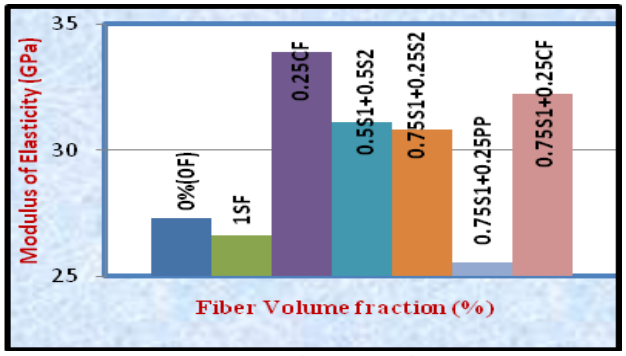

Fig. I Modulus of Elasticity of Mixes

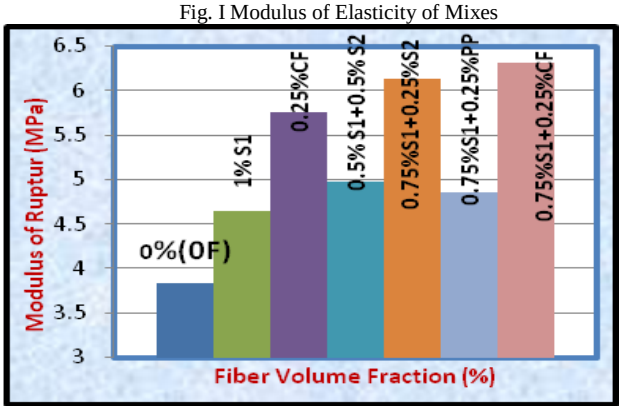

Fig. II Modulus of Rupture of Mixes

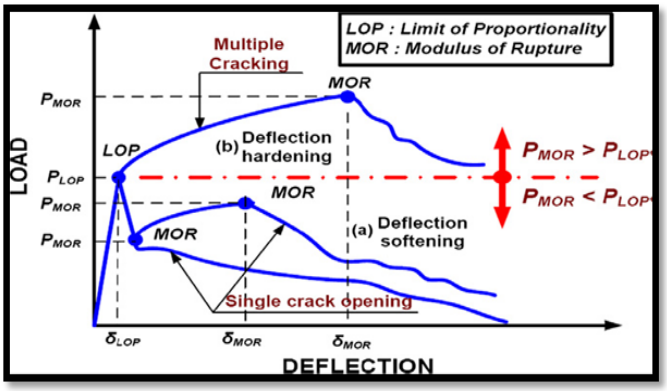

Fig. III Typical Load–Deflection Response Curves of FRCC

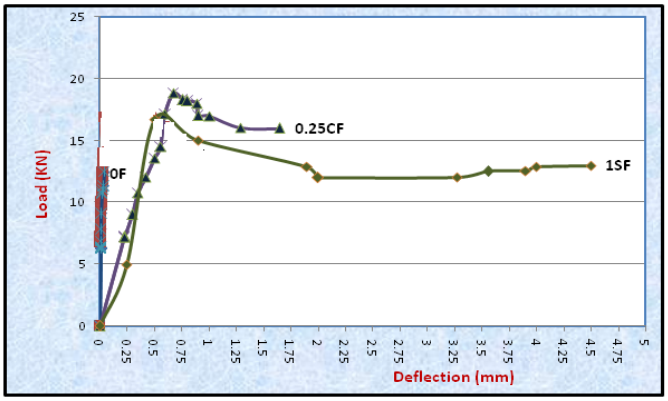

Fig. IV Load –Deflection Curves for HSLWAC Mixes with Mono Fibers

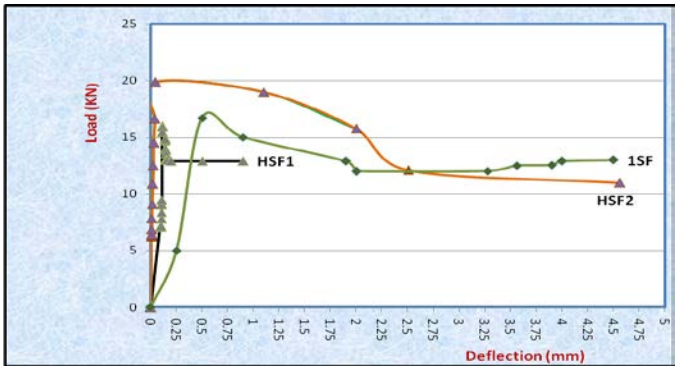

Fig.V Load –Deflection Curves of 1SF, HSF1, and HSF2 Mixes

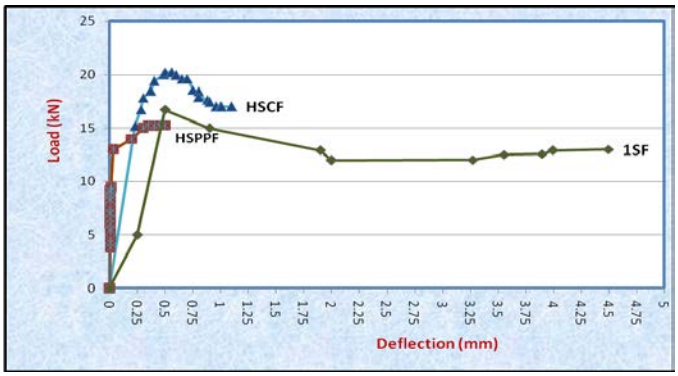

Fig. VI Load –Deflection Curves of 1SF, HSPPF, and HSCF Mixes

E. Flexural Toughness Calculated by Different Techniques

Toughness is generally defined as energy adsorption capacity of the material [18]. There are a number of different techniques for measuring toughness of fiber reinforced concrete such as, ASTM C 1609-12[15], JSEC SF-4 techniques [9] and post crack strength (PCS) method [19]. The flexural parameters such as, P_{600}, P_{150}, f_{600}, f_{150} and T_{150} for fibrous concrete obtained using ASTM C 1609 -12 are shown Table (III). Flexural toughness (T_{150}^{100}) was obtained from the area under the load–deflection curve up to L/150 of the span length (2mm deflection in this study). Only two points including L/600 and L/150 of the span length are recommended in ASTM standard C 1609-12 to analyze the flexural behavior of fiber reinforced cement composites. Additional point which is L/100 of the span length is recommended by Kim et al. [cited in 20]. This is because the two points L/600 and L/150 are not enough to describe the flexural behavior of some types of fiber reinforced cement composites. In this study, fibrous HSLWAC mix 1SF and HSF2 show high ductility and the two points L/600 and L/150 are not sufficient to describe the entire flexural behavior, so point L/100 of the span length [3mm deflection] is determined to describe the total flexural behavior of these mixes.

Generally, the results in Table (III) show that after adding fibers, the flexural toughness of HSLWAC increases greatly compared to the plain HSLWAC. Long steel fibers with high volume fractions (1%) have significant effect on flexural toughness which is 80.49 Joule. This is due to the fact that the long steel fibers delay the unstable growth of cracks which

usually occur in flexural resulting in improved post-peak load behavior. Micro mono carbon fibers have a relatively lower effect on flexural toughness of HSLWAC than that for mono steel fiber. This may be attributed to short length of this type of fiber. The improving effect on flexural toughness of polypropylene fiber is meaningfully lower in hybrid fiber HSLWAC mixes incorporating both steel and pp fibers (mix HSPPF – 0.75% S1+0.25% PP). The lower effect of pp fiber on flexural toughness is attributed to the lower tensile strength of these fibers and also the weaker bonding between pp fiber and cement matrix [6]. The use of hybrid macro steel fibers in mix HSF2 (0.75% volume fraction of macro steel fiber type S1 + 0.25% volume fraction of macro steel fiber type S2) shows the highest improvement in flexural toughness (92.42 Joule). This is attributed to the fact that steel fiber is stiff and also the values of toughness are sensitive to the volume fraction and percentage of longer fibers in concrete mix. Higher fiber volume fractions and higher percentage of longer fibers in the concrete mix leads to higher values of toughness [2].

Flexural toughness of hybrid fiber HSLWAC mixes, HSF1, HSPPF, and HSCF, containing the combination of, steel fiber type S2, polypropylene fiber, and carbon fiber as a minor fiber with volume fractions of 0.5%, 0.25%, 0.25% and steel fiber type S1 as a major fiber with volume fractions of 0.5%, 0.75%, 0.75% respectively, is lower than that for HSLWAC reinforced with 1% volume fraction of mono steel fiber type S1 (mix 1SF). Hybrid fiber HSLWAC mix reinforced with a combination of steel fiber type S1 (0.75% volume fraction) and steel fiber type S2 (0.25 volume fraction) shows a significant improvement in flexural toughness relative to LWAC mix reinforced with mono steel fiber type S1 with 1% volume fraction (mix 1SF). It can be concluded that fibrous HSLWAC specimens 1SF and HSF2 perform better than plain and other fibrous HSLWAC specimens in terms of flexural toughness, residual strength R_{150}^{D} at net deflection L/150 (2mm).

In the Japan Society of Civil Engineers (JSCE-SF4) the absolute value of the flexural toughness called "Absolute toughness value T_{JSCE}" can be calculated from the area under the load-deflection curve to a deflection of the span/150. The "flexural toughness factor (FT) is determined from the following equation:

$$\mathbf{FT = (T_{JSCE.}\, L)/\ [\delta_{150}.bh^2]} \qquad \mathbf{(1)}$$

Where, **(L)** is the span of the beam, **(b, h)** are the dimensions of beam section, and **(δ_{150})** is the deflection of L/150.

One of the concerns with this method is that, the behavior of the material immediately after the first crack, which may be important in many applications, is not indicated in the FT factor. Also the area under the curve, used to calculate FT factor combines the pre-crack and post-crack energies and it is criticized for failing to distinguish between these two areas, so fiber reinforced concrete mixes with quite different load deflection curves can yield similar toughness parameters. This test has a fixed deflection end point so it cannot easily be adapted to different serviceability condition and does not reflect the characteristics of the load-deflection curve [10]. The Post Crack Strength (PCS) approach depended on the location of first crack load in load- deflection curve which is subject to human error. The technique proposes to locate the peak load and divide the curve into two regions: the pre-peak region and post-peak region. The area under the curve is then calculated up to the peak load and termed as pre-peak energy, i.e., E_{pre}. In the post-peak region, points are located corresponding to deflections L/m, where 'm' could have different values ranging from 3000 to 150. 'L' is the span of the beam. The area under the curve up to a deflection of L/m is termed as the total energy i.e. E_{total},m. The pre-peak energy is subtracted from this total energy to obtain the post-peak values i.e. $E_{post,}$m corresponding to a deflection of L/m. The post-crack strength PCS at a deflection of L/m is given as [10]:

$$\mathbf{PCS,m = (E_{post},m).L/[((L/m) - \delta_{Peak})\ b\ h^2]} \qquad \mathbf{(3)}$$

Where: **(δ_{Peak})** is the deflection at peak load, **(b, h)** are the dimensions of beam section, **(L)** is the span of the beam.

Flexural parameters calculated according to JSCE-SF4 and PCS methods are shown in Table (IV). The results in Tables (III) and (IV) indicate that the toughness T^{D}_{150} from ASTM C1609 method shows similar values as presented by toughness value $\mathbf{T}_{JSCE}$ obtained by the JSCE method, but ASTM C1609 method shows more flexural parameters that represent the entire load- deflection behavior. It can be found that PCS method is not successfully represent the toughness of some mixes (mix 1SF). Some researchers [7] determine the total toughness of hybrid fiber reinforced concrete from the total area under the entire load-deflection curve, which represents the total energy. Table (V) shows the total toughness for all mixes studied in this investigation. It can be observed that the total toughness for concrete mixes 1SF and HSF2 is higher than that calculated from ASTM C1609 and JSCE methods since the deflection of these mixes at failure is more than 2mm (L/150) and more than 3mm (L/100). This method appeared to be more effective to represent the toughness of fiber reinforced concrete.

TABLE III
FLEXURAL PARAMETERS ACCORDING TO ASTM C 1609-12

Mix Symbol	P1 (kN)	Pp (kN)	δ1 (mm)	δp (mm)	f1 (MPa)	fp (MPa)	Pave (kN)	f^{D}_{eq} (MPa)	P^{a}_{150} (kN)	f^{D}_{150} (kN)	P^{a}_{300} (kN)	f^{D}_{300} (MPa)	T^{D}_{150} N.m	$R^{a}_{D,150}$ (%)
0F	----	12.46	----	0.0464	----	3.84	---	---	---	---	--	---	0.31	0.0012
1SF	16.71	16.71	0.5	0.5	5.01	5.01	16.71	5.01	12.0	3.60	12.02	3.61	80.49	0.3541
0.25CF	12.99	18.82	0.499	0.6800	3.98	5.76	13.56	4.15	---	---	--	---	18.97	0.0729
HSF1	7.79	15.90	0.1044	0.1149	2.43	4.97	12.90	4.03	---	---	--	---	16.54	0.1063
HSF2	16.61	19.86	0.0384	0.0464	5.12	6.14	19.82	6.13	15.8	4.87	11.51	3.56	92.42	0.5880
HSPPF	12.98	15.25	0.0300	0.500	4.12	4.86	15.25	4.12	---	---	---	---	10.86	0.0122
HSCF	18.46	20.23	0.3700	0.500	5.77	6.32	20.23	6.23	---	---	---	---	27.09	0.0730

TABLE IV
FLEXURAL PARAMETERS ACCORDING TO
(JSCE-SF4) AND (PCS) METHODS

Mix Symbol	Absolute Toughness Value T_{JSCE} N.m (J)	Flexural Toughness Factor FT (JSCE Method) (MPa)	Post crack strength (PCS) (MPa)										
			L/M (mm)										
			0.1	0.2	0.3	0.4	0.5	1	1.5	2	3	4	4.5
0F	0.31	0.047	.....	....	....	....	....	....	...	...			
1SF	80.49	15.90	.....	....	....	....	....	....	...	...	....	0.21	0.21
0.25CF	18.97	2.84	....	....	....	....	....	5.16	4.86	4.91	....	....	....
HSF1	16.54	2.18	....	3.75	3.87	3.87	3.87	3.75	...	...	....	....	....
HSF2	92.42	13.86	5.97	5.69	5.99	5.85	5.97	7.13	4.66	4.99	7.77	8.17	7.33
HSPPF	10.86	1.63	2.46	4.86	4.56	4.57	4.57	...	....	....	....	....	....
HSCF	27.09	4.06	....	...	...	2.27	4.87	6.0	5.1	...	....	....	....

TABLE V
TOTAL TOUGHNESS OF CONCRETE MIXES

Mix Symbol	Total Energy (N.m))(Joule)
0F	0.31
1SF	130.16
0.25CF	18.97
HSF1	16.54
HSF2	188.17
HSPPF	10.86
HSCF	27.09

IV. CONCLOSIONS

The main findings of this research are as follows:

1. The inclusion of fibers increases the flexural strength in both mono and hybrid fiber specimens relative to the plain specimens. Hybrid fiber reinforced HSLWAC specimens show significant increase in flexural strength. The percentages of increase in flexural tensile strength for HSLWAC specimens' prepared from HSF1, HSF2, HSPPF and HSCF mixes are 29.43%, 59.43%, 26.56% and 64.58% respectively.
2. Concrete mix containing hybrid steel and polypropylene fibers (0.75% steel fiber S1+ 0.25% pp fiber) shows a decrease in modulus of elasticity by about 13.7% relative to the plain mix.
3. The UPV values for all HSLWAC mixes are between 3.30 and 4.35 km/ sec, which represent the good quality of concrete.
4. The addition of fibers improves the load-deflection behavior for HSLWAC and consequently shows higher ductility than the nonfibrous mix. Addition of fibers is found to change the brittle nature of the nonfibrous matrix to a composite mass with a plastic behavior.
5. The flexural toughness of mono fibrous HSLWAC increases greatly compared to the plain HSLWAC. It should be emphasized that the flexural toughness of specimens containing mono steel fiber type S1 with volume fraction of 1% consistently increases.
6. Flexural toughness of hybrid fiber HSLWAC mixes incorporating the combination of steel fiber type S2, polypropylene fiber, and carbon fiber as a minor fiber (0.5%, 0.25%, 0.25%) and steel fiber type S1 as a major fiber (0.5%, 0.75%,0.75%) is lower than that for HSLWAC reinforced with 1% volume fraction of mono steel fiber type S1.
7. The use of hybrid fibers in mix HSF2 (0.75% macro steel fiber type S1 + 0.25% macro steel fiber type S2) shows significant and highest improvement in the flexural toughness. The value of the flexural toughness is higher than that for concrete mix reinforced with 1% volume fraction of mono steel fiber type S1.
8. Toughness calculated by ASTM C1609 method shows similar values as that obtained by the JSCE method, but ASTM C1609 method shows more flexural parameters that represent the entire load- deflection behavior.
9. Total toughness of hybrid fiber reinforced concrete obtained from the total area under the entire load-deflection curve, appeared to be more effective to represent the toughness of fiber reinforced concrete.

REFERENCES

[1] N. A. AL-Bayati, K. F. Sarsam, and I. A. S. AL-Sharbaf, "Compressive Strength of Lightweights Porcelanite Aggregate Concrete – New Formulas", Engineering and Technology Journal, vol.31, part A, no.10, pp.1897-1913, 2013.

[2] Y. A. Mohammadi, S. P. Singh, and S. K. Kaushik, "Properties of Steel Fibrous Concrete Containing Mixed Fibers in Fresh and Hardened State", Construction and Building Materials Journal, vol.22, pp.956-965, 2008.

[3] H. T. Wang, and L. C. Wang, "Experimental Study on Static and Dynamic Mechanical Properties of Steel Fiber Reinforced Lightweight Aggregate Concrete", Journal of Construction and Building Materials, vol. 38, no.11, pp.1146-1151, 2013.

[4] W. I. Khalil, and S. A. Mozan, "Some Properties of Hybrid Fiber High Strength Lightweight Aggregate Concrete", Engineering and Technology Journal, vol.33, part (A), no.4, pp.815-829, 2015.

[5] H. M. Saleh, "Using of Porcelinite as Aggregate in Concrete, Tikrite Journal of Engineering Sciences, vol. 19, no.3, pp.33-40, Sep. 2011.

[6] Hollow Stone Building System. Available at: www.worldassociates.com/hollowstone/advantages.

[7] N. A. Libre, M. Shekarchi, M. Mahoutian, and P. Soroushian, "Mechanical Properties of Hybrid Fiber Reinforced Lightweight Aggregate Concrete Made with Natural Pumice", Construction and Building Materials, vol. 25, no.12, pp. 2458-2464, 2011.

[8] J. A. Bogas, M. G. Gomes, and A. Gomes, "Compressive Strength Evaluation of Structural Lightweight Concrete by Non-Destructive Ultrasonic Pulse Velocity Method", Ultrasonic's Journal, vol.53, no.1, pp. 962-972, 2013.

[9] A. E. Yurtseven, "Determination of Mechanical Properties of Hybrid Fiber Reinforced Concrete", M.Sc. Thesis, Middle East Technical University, August 2004.
[10] F. Majdzadeh, "Fracture Toughness of Hybrid Fiber Reinforced Self-Compacting Concrete", M.Sc. Thesis, the University of British Columbia, March 2003.
[11] E. T. Dawood, and M. Ramli, "Contribution of Hybrid Fibers on the Properties of High Strength Concrete Having High Workability, Construction and Building Materials, vol.14, pp. 814-820, 2011.
[12] ASTM C 597 -97, "Standard Test Method for Pulse Velocity through Concrete", Annual Book of ASTM Standards, vol. 04.02, 2007.
[13] M. Hassanpour, P. Shafigh, and H. B. Mahmud, "Lightweight Aggregate Concrete Fiber Reinforcement – A Review", Construction and Building Materials, vol.37, no.9, pp.452-461, 2012.
[14] I. Markovic, "High-Performance Hybrid-Fiber Concrete - Development and Utilization", M.Sc. Thesis, Delft University of Technology, Amsterdam, 2006.
[15] ASTM C1609M–10, "Standard Test Method for Flexural Performance of Fiber –Reinforced Using Beam with Third- Point Loading", Annual Book of ASTM Standards, 2012.
[16] A. A. AL- Attar, "Microstructure and Mechanical Properties of Lightweight Aggregate Concrete Containing Fibers", Ph.D. Thesis, University of Technology, Baghdad, Iraq, p.188, 2008.
[17] P. Muthupriyo, K. Subramunion, and B. G. Vishnuram, "Experimental Investigation on High Performance Reinforced Concrete Columns with Silica Fume and Fly Ash as Admixtures", Asian Journal of Civil Engineering (Building and Housing) , vol.12, no. 5, pp.597-618, 2011.
[18] A. T. Zween, "Performance of Lightweight Aggregate Concrete Incorporating Fiber Exposed to Elevated Temperature", Ph.D. Thesis, University of Technology, Baghdad, Iraq, p.171, 2008.
[19] B. Chen, and J. Liu, "Contribution of Hybrid Fibers on the Properties of High Strength Lightweight Concrete Having Good Workability", Cement and Concrete Research, vol.35, no.6, pp.2458-2464, 2005.
[20] E. T. Dawood, and M. Ramli, "Mechanical Properties of High Strength Flowing Concrete with Hybrid Fibers", Construction and Building Materials, vol.28, no.10, pp. 193-200, 2012.

K. Wasan Ismail B.Sc. in Building and Construction Engineering, University of Technology, Building and Construction Engineering Department, Baghdad, Iraq, 1981. M.Sc. in Structural Engineering, University of Technology, Building and Construction Engineering Department, Baghdad, Iraq, 1989. Ph.D. in Building Materials Engineering, University of Technology, Building and Construction Engineering Department, Baghdad, Iraq, 2006.
She is Asst. Prof. in University of Technology, Building and Construction Engineering Department, Baghdad, Iraq. She has thirty published papers in different journals and international conferences, two innovations, one published book.
Asst. Prof. Khalil is a membership in Iraqi Engineers Union and American Concrete Institute (ACI).

Comparison among Sulaimani, Iraq Cements with Local Neighbour Countries Cements: Implication of Local Climate on Mortar

Dr Nihad B. S. Baban, Ansam M. Altaee and Tara R. Abdul-Wahab

***Abstract*—These The notable changes of weather are caused visible influences on buildings in Iraq. These manifest both external and internal parts of buildings. Mortars are facing the local weather continuously and found to be impacted in Iraqi cities. This research simulates hypothesised conditions of mortars uses cements manufactured locally and cements from Turkey and Iran, where cracks appear and mortar weakness is a recognised problem. Three cements from Sulaimani and two from Iran and Turkey were used to prepare a set of mortar cubes in size of 5.0 x 5.0 x 5.0 cm. cubes are cured for 1, 3, 7 and 28 days and then were divided into three groups with the same treatment format. These groups are only cured, saturation-affected and temperature-affected groups. the second and third groups are lasted for 10 weeks. The continuous local climate variations showed an effect on compressive strength compare to the first group.**

***Index Terms*— Cements, Mortar, Stress-Strain Relation, Compressive Strength, Local Climate.**

I. Introduction

MOST researches involving the cement-based mortar seek to highlight the effect of water content change on the mortar's physical properties. Since the cement-based mortar usually affects by not only the change in water content, but also by the temperature change, it may therefore be advantageous to investigate the effects of both at the same time. There have been a lot of studies that investigated the change in physical properties of cement-based mortar due to environmental changes.

One of the main factors that has an impact on the rate effect of materials like cement-based mortar is water content [9]. Nevertheless, there has not been any consensus as to whether the dynamic behavior changes as the water content increases or decreases. A wet concrete experience increase in substantial strength in a system with reasonable strain rate, unlike the dry concrete which has been proved to be relatively insensitive. The engineering properties of wet concrete are more affected by the increase of water ratio [2], [3], [5], [6], [10].

It was also proven that oven drying damages the pores in hardened cement pastes, especially if it reached 105°C [7]. So, oven drying leads to the formation of cracks in the microstructure of the cement-based mortar specimen, and the crack density tends to increase with the increase of the drying [11]. It is known that the small grain size of 2 mm (maximum size) used in mortar helps preventing the development of large bond cracks between mortar matrix and sand. However, the sever drying that reaches 105°C causes not only the appearance of cracks in the specimen, but also causes the increase of their size if they have appeared already before drying [8].

Cement-based mortar tends to shrink when it loses water while swells when saturated [4], [12]. The volume change accompanies the shrinkage and swelling is relatively close to micro-capillarity action in the pores and interlayer spaces of the specimen. However, the distribution of this volume change across the specimen's section is never uniformed, which leads to what is known as "Differential Volume Change". This differential volume change leads to the generation of strains and stresses, just like thermal stresses do. In such conditions, a cement-based mortar specimen can be "pre-stressed in either tension or compression" in order to trigger the increase the specimen's strength and decrease Young's modulus as the water content is increased gradually.

This research focuses on testing the engineering properties of four types of cement-based mortar existing in Sulaimani Governorate's market. These types are: Bazyan, Karasta from Sulaimani, Iraq factory, Turkish/Kalekim and Iranian/Gharb. The uniqueness of this research lays in the fact that the local environmental effects applied to the mortar are specific to those of Iraq- Sulaimani region. These neighbour countries' cements were selected for two main reasons. First, because they are used in construction work in Sulaimani Governorate. Second, to check the similarities and differences between local cements and the cement of those neighbour countries.

II. MATERIALS AND METHODS

This work consists of searching for the existing cements in Sulaimani City market, collecting all the available types of cement, and seeking for a standard sand. Additionally, it

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Dr Nihad B. S. Baban is an Adjunct Professor, Department of Engineering, American University of Iraq - Sulaimani (AUIS). Phone: 009647701315050; e-mails: nihad_baban@yahoo.com, nihad.bahaaldeen@auis.edu.krd.

Ansam M. Altaee, was a former undergrad student, construction engineering track at the Department of Engineering, American University of Iraq - Sulaimani (AUIS).

Tara Raad Abdul-Wahab, was a former undergrad student, construction engineering track at the Department of Engineering, American University of Iraq - Sulaimani (AUIS).

consists of compressive strength tests that were conducted in the laboratory for the prepared mortar cubes.

The idea of this research is based on the fact that the components of concrete will be able to resist the applied forces, in terms of compressive strength, depending on the achieved adhesion bonds among them. The weak bonds will make it prone to failure or stay in a weak state. Hence, the environmental cases applied on the cubes may play a positive/negative role. So, understanding the real influences will help to discover the ability of cement-based mortar in the specific cases stated before it fails. Additionally, losing the adhesion bonds due to external impacts will weaken the total materials durability. Laboratory work was conducted in the Construction Engineering laboratories of the American University of Iraq-Sulaimani.

A standard sand was selected according to ASTM [1]. Water is added according to the water/cement ratio (0.45 - 0.60) standard range, 0.5 percent was selected, AUIS tap water was used. The prepared mortar cubes' size is 5.0 x 5.0 x 5.0 cm, the prepared cubes before and after tests are shown in the appendices. The cubes were cured in water basins for 1, 3, 7 and 28 days ASTM [1]. After that, the cured cubes were treated as the proposed cases of the research in the following paragraph.

Iraq as most of the world countries is suffering from the new strange climate. The seasonal temperature degrees have been changing if they were compared to the previous century years. Not only the temperature is affected, the rainfall percent in the rain seasons has also changed. All buildings in the region are subjected to be prone to negative changes. The existing climate changes will negatively enhance the weather-subjected materials to be affected. Therefore, discovering the climate change role on the stability and workability of building sections (cement-based mortar is selected) is the highlighted idea of this research. The following paragraphs will explain all the proposed cases.

As Iraq seems to be a semi-arid region, dryness can be harmful to construction materials. The weather is a bit cold with a normal rainy season in winter and very dry and long summer. The wet-dry case of this research is chosen based on this weather change between the two seasons. This case procedure consists of making the mortar cubes go through cycles of wetting and drying environments. The cycles are applied by putting the cubes in a dry room in order to keep them at room temperature (25°C) for one week and then straight away the cubes were put in a water basins for another week. These described dry and wet durations are one cycle. The process is repeated ten times throughout ten weeks.

The temperature degree of Sulaimani Governorate in winter could reach - 10°C and below. While, in summer, it is around 50°C. This big gap in temperature calls to consider this climate situation as a second case of this research work. The procedure consists of making the mortar cubes go through cycles of heating and freezing environments. The cycles are applied by putting the cubes in a refrigerator at - 5.0^{o}C for one week, and then straight away they were put in drying oven at 50°C. These described freezing and thaw durations are one cycle. The process is repeated ten times throughout ten weeks.

After that, a uniaxial compression testing stage was carried out. Then, physical changes in term of volume changes were measured. The compression test was conducted first on the prepared cubes that went through water-curing for 1, 3, 7 & 28. Afterwards, the wet-dry and freezing-thaw cases were tested for uniaxial compression. The rate of the applied load was 4.0 kN/min for all the tested cubes. Tension/compression Instron machine was used. Each specimen was loaded up to 10% of its failure point.

III. RESULTS AND DATA PRESENTATION

The presented results are firstly the normal samples, which cured with the specified periods and then tested for uniaxial compressive strength afterwards. Secondly, samples that were subjected to wet-dry cycles, saturation-affected cubes tested for uniaxial compressive strength. Finally, the samples that were subject to freezing-thaw cycles, temperature-affected cubes, tested for uniaxial compressive strength were presented.

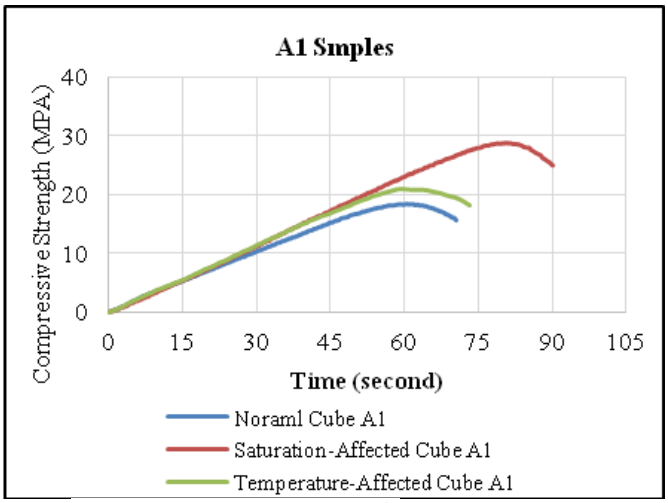

Fig. 1. Time versus compressive strength of Karasta mortar cubes. One day curing.

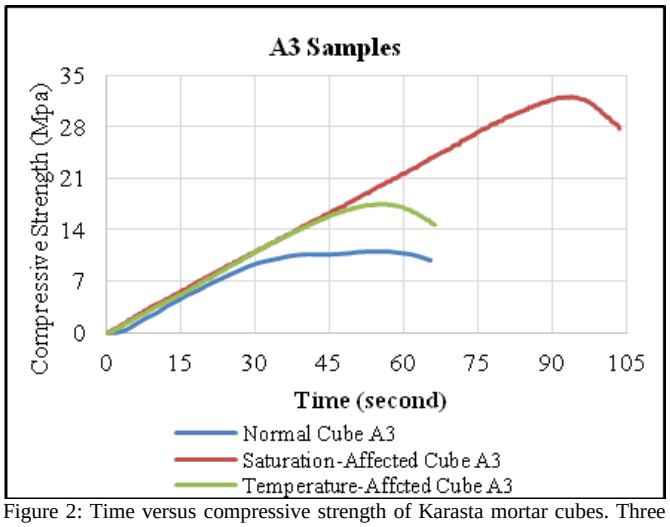

Figure 2: Time versus compressive strength of Karasta mortar cubes. Three days curing.

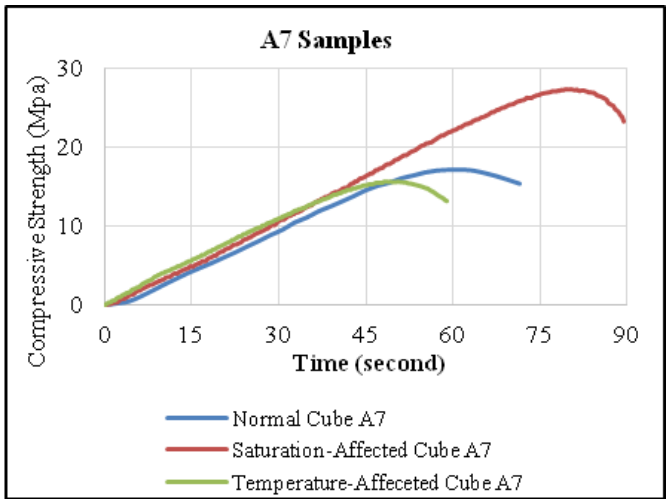

Figure 3: Time versus compressive strength of Karasta mortar cubes. Seven days curing.

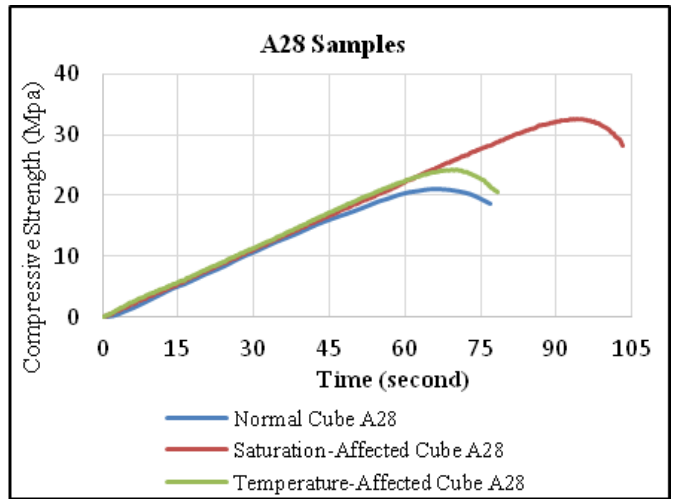

Figure 4: Time versus compressive strength of Karasta mortar cubes. Twenty-eight days curing.

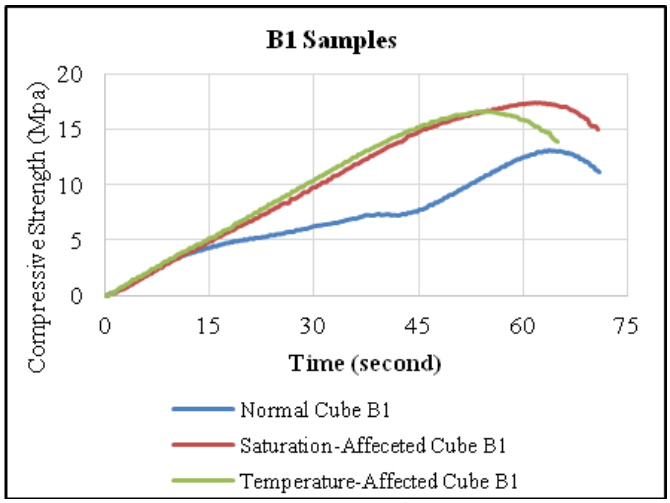

Figure 5: Time versus compressive strength of Bazian-Tasluja mortar cubes. One day curing.

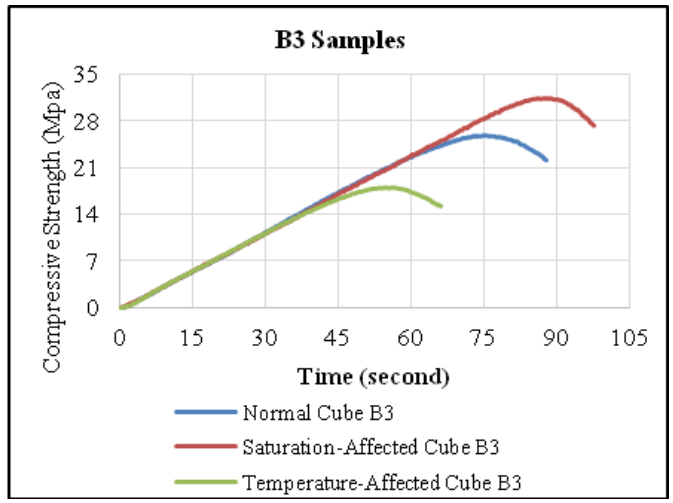

Figure 6: Time versus compressive strength of Bazian-Tasluja mortar cubes. Three days curing.

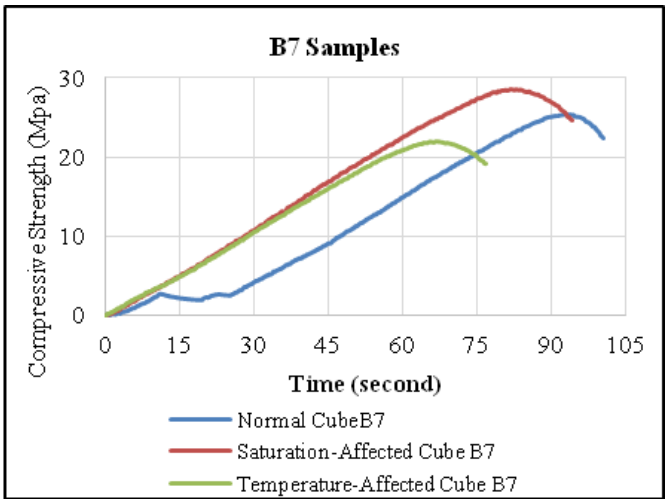

Figure 7: Time versus compressive strength of Bazian-Tasluja mortar cubes. Seven days curing.

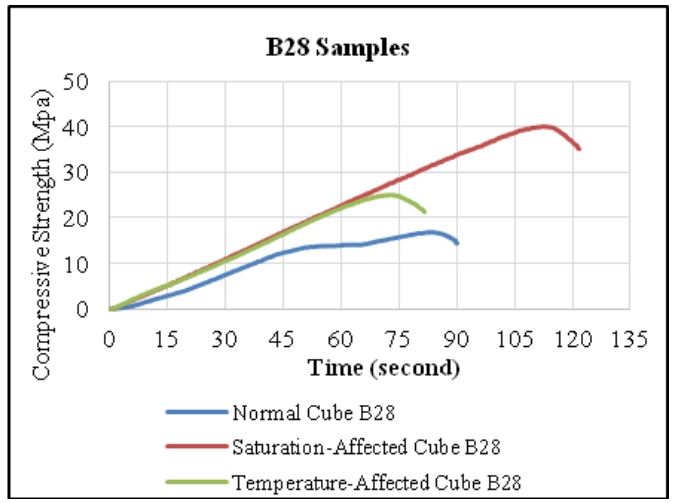

Figure 8: Time versus compressive strength of Bazian-Tasluja mortar cubes. Twenty-eight days curing.

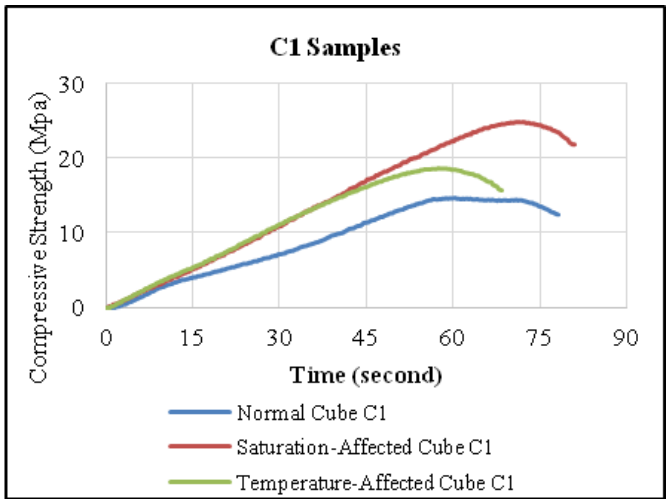

Figure 9: Time versus compressive strength of Gharb mortar cubes. One day curing.

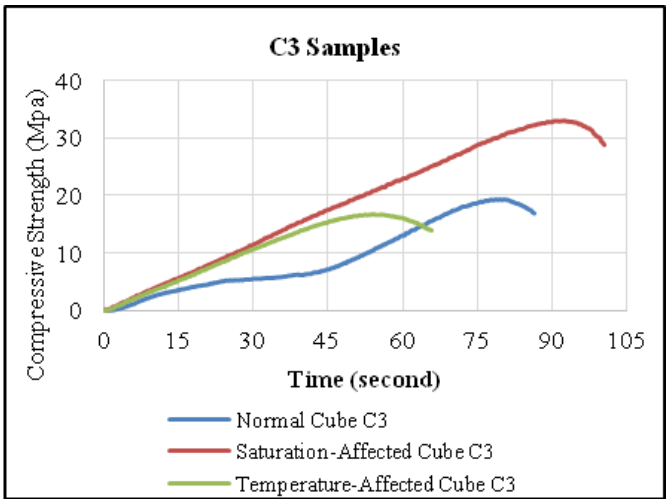

Figure 10: Time versus compressive strength of Gharb mortar cubes. Three days curing.

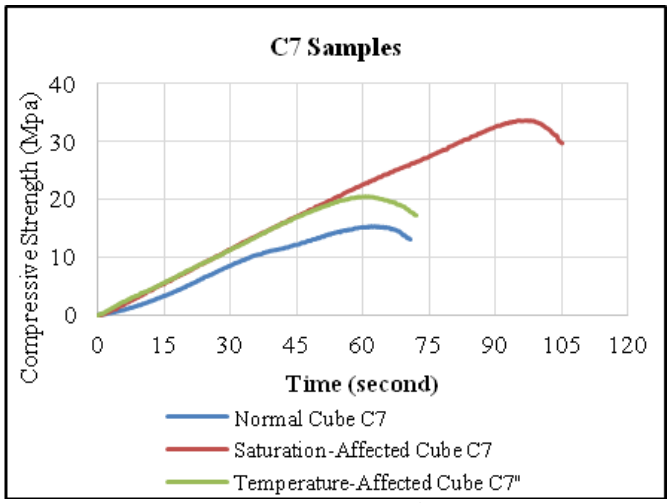

Figure 11: Time versus compressive strength of Gharb mortar cubes. Seven days curing.

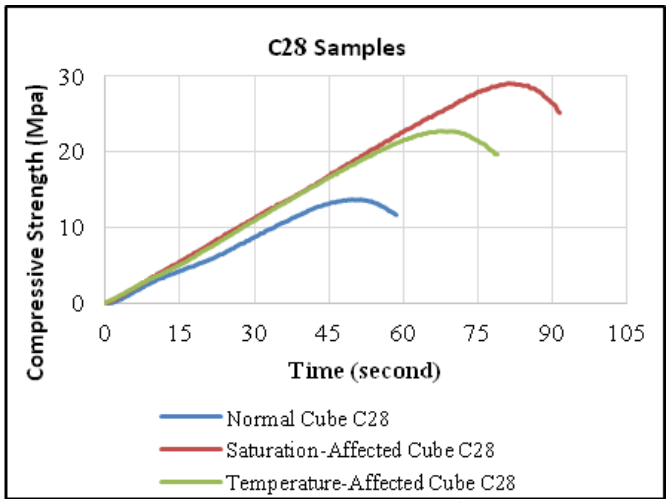

Figure 12: Time versus compressive strength of Gharb mortar cubes. Twenty-eight days curing.

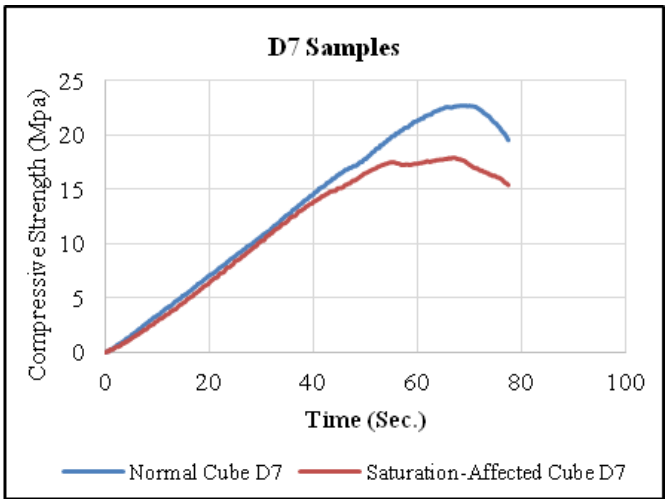

Figure 13: Time versus compressive strength of Kalekim mortar cubes. Seven days curing.

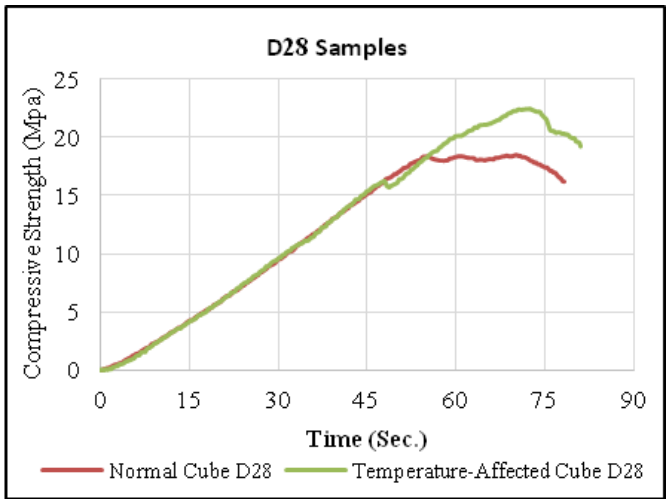

Figure 14: Time versus compressive strength of Kalekim mortar cubes. Twenty-eight days curing.

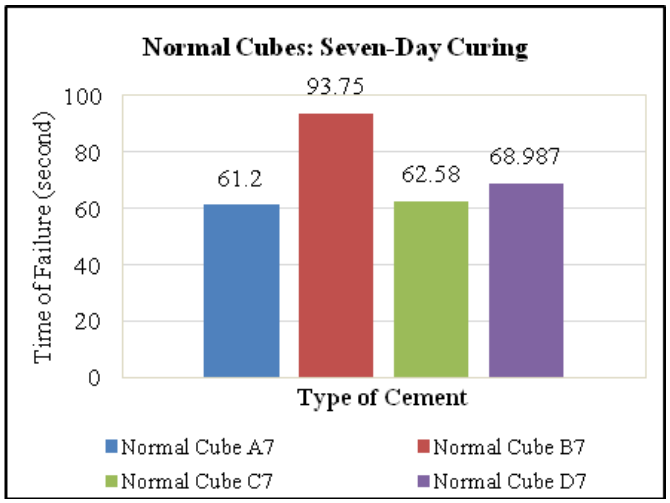

Figure 15: Time of failure values of four different basic-mortar cubes. Seven days curing.

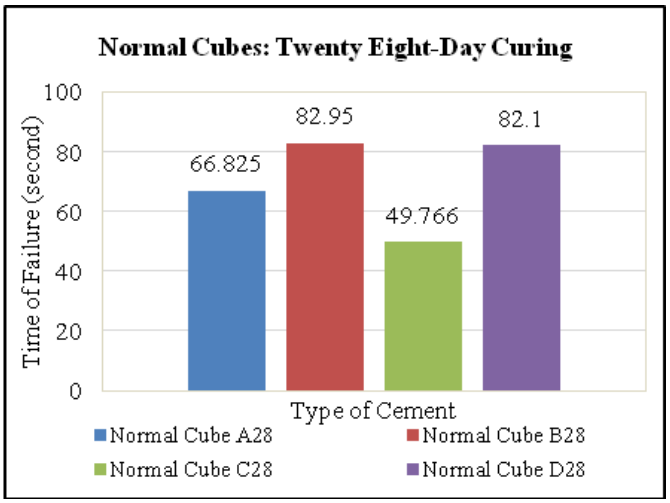

Figure 16: Time of failure values of four different basic-mortar cubes. Twenty-eight days curing.

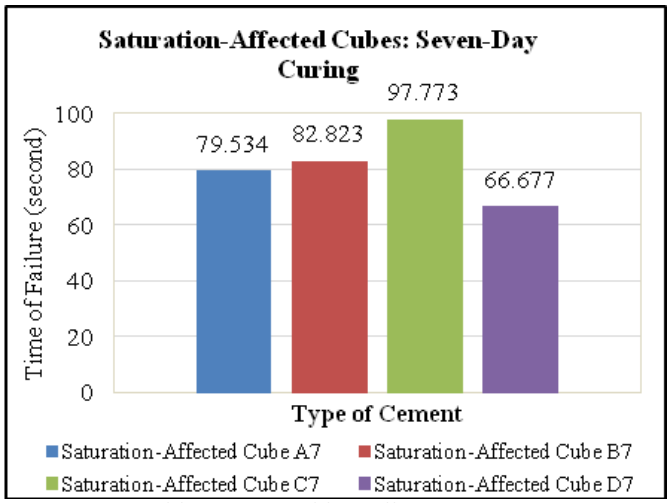

Figure 17: Time of failure values of four different mortar cubes. Seven days curing, and then subjected to wet-dry cycles.

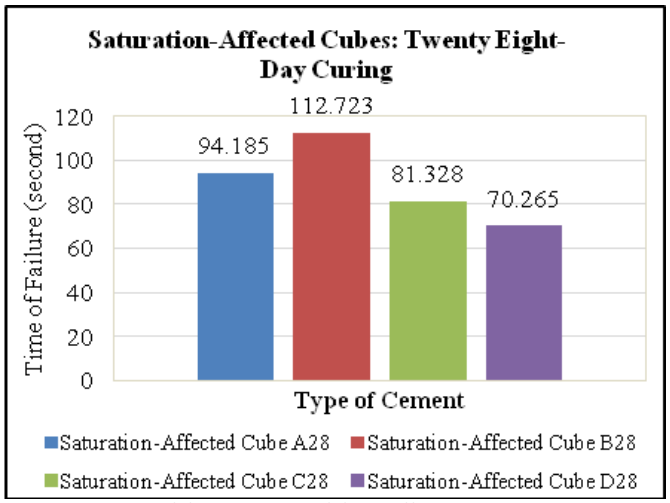

Figure 18: Time of failure values of four different mortar cubes. Twenty-eight days curing, and then subjected to wet-dry cycles.

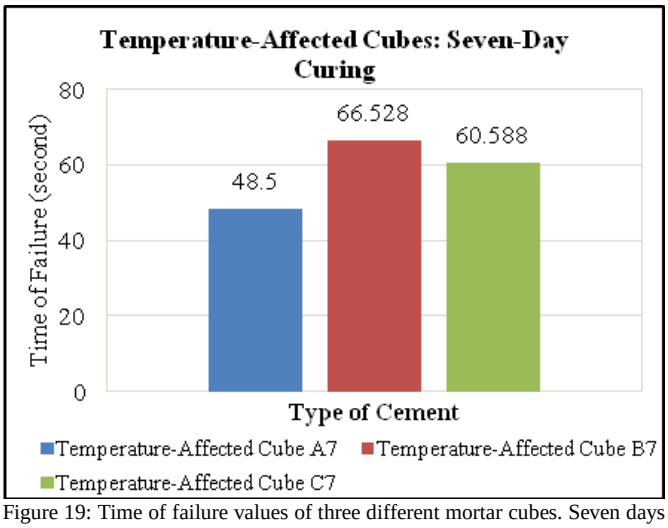

Figure 19: Time of failure values of three different mortar cubes. Seven days curing, and then subject to freezing-thaw cycles.

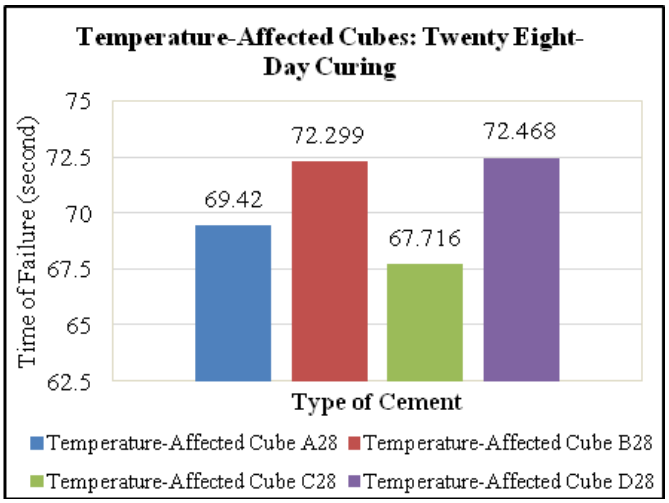

Figure 20: Time of failure values of four different mortar cubes. Twenty-eight days curing and then subject to freezing-thaw cycles.

IV. DISCUSSION

On the mechanical properties, figures 1 to 14, this work suggests that there are 2 major variables that can be controlled to examine the mechanical response of cement-based mortar to local climate: 1) saturation-drying, 2) freezing-thawing impact. The research has a number of implications for cementing bonds among mortar components and hence for the mechanical properties:

1. The role of saturation-drying enhancing changes of cement-based mortar is confirmed. The applied case created as a result of the local environmental changes may be expected to increase the potential for influencing the mechanical properties of mortar. and increase the likelihood of mortar weakening. Similar effects could be induced by wetter conditions caused by climate change.
2. Differences in original materials used in cement manufacturing can have a significant impact on the properties of manufactured cement and hence timing of failure.
3. Cement-based mortar provides a successful method for simulating real local environmental issues.
4. The application of various levels of temperature has a significant impact on the engineering properties of the mortar cubes.
5. The application of various levels of saturation has a significant impact on the engineering properties of the mortar cubes.
6. There are measurable influences of local climate on the behaviour of these mortar cubes under axial stresses after subjection to saturation-drying and freezing-thawing over time.
7. The mechanical properties of cement-based mortar cubes are noticeably influenced by the continuous subjection process to the local climate issues in long term tests.
8. Some similarities among the compressive strength behaviors of cured mortar cubes for all originally-allocated curing durations. The variation is initially linear up to its ultimate strength. Afterwards, the graphs have drooped. While, the behavior of the normal specimen is differ. It starts linearly of its length, and then it becomes non-linear until the ultimate point.

More on above points (figures 15 to 20), some of the saturation-affected (such as groups A1 and C1) cubes recorded high time of failure, while the normal cubes recorded the notably lower. This is mainly due to the fact that the wet-dry cycles (immersing the mortar in water) works mostly like a curing period that allows the immersion water to control the internal process. This will create the right environment for the cement to complete the hydration process. Internally, some percent of heat will be extracted at a time when the surrounding climate will impact too. Therefore, the balance will be activated in the immersion water to positively coat the fresh concrete mortar to finish the hydration process.

The non-linearity of some curves is due mainly to the surface of the cubes being relatively rough. The roughness is caused by the different size of sand grains that were mixed with the cement and water to create the mortar.

The influences of higher applied temperature may cause some expansion and hence a weaker state. Then, the saturation-dry sample in theory should be stronger and in few cases became weaker (group B1). This may be resulted from the increase of the lubrication system surrounding the weak bond after one day curing.

While curing, the water had the chance to intersperse among the cement and sand particles, which led to the increase in its volume. On the other hand, and during the freezing-thaw cycles, the specimen went through expansion and shrinkage several times, which led to the larger increase in volume compared to the saturation-affected specimens. This may be due to the noticeable role of higher temperate (heating) compare to the lower temperature (freezing). The whole duration of the freezing-thaw case actively expands the cubes. As a result, the freezing-thaw case showed a larger volume increase compared to the saturation-dry case.

Notably, for one-day curing, a mortar cube has not gained all its strength because there was not enough time for the hydration process to complete. Therefore, the wet-dry cycles clearly increase the lubrication system internally. This will negatively impact and may cause some weakening of strength and further more on volume change.

On an impact of environmental cases, All subjected cubes to some environmental cases affected, manifested as a change in mass, volume and compressive strength over time. This change does not, however, explain all the weakening that took place. There must also be a contribution from changes taking place within the samples. The wet-dry cycles impact the cubes of groups A, B and C by increasing its compressive strength. While, the freezing-thaw cycles impact is increased the compressive strength of A and D groups compared with the normal group A cubes. For B and C groups, the compressive strength values vary between more and less than that of the normal specimens. The 28 days cubes are almost the strongest in terms of adhesion bonds among sand and cement. Among all temperature-affected cubes that were cured for 28 days, D28 specimen took the longest time to achieve failure; i.e. reaching its ultimate strength.

On a Comparison among cements used, cement A (Karasta) has a range of fineness property between 360 and 380, while cement B group (Bazian-Tasluja) has a fineness range of 325 – 340. The finer the cement is, the smaller its particle size, and the larger in its surface area. The larger surface area allows for more water absorption during curing and more coating, which will positively impact on the hydration process. Hence, this will increase mortar compressive strength and restrict volume change. Looking back at the results of this research, the local cement-based mortar (Bazian-Tasluja) is the strongest among the four other tested types. This may be due to the fact that the materials from which the cement was manufactured are extracted from the same region; therefore, it is more likely to resist its own harsh environment than the foreign cement types.

V. CONCLUSIONS

The research has shown that the evolving behaviour of mortar can be investigated in the laboratory over extended periods. It highlights that the hazard presented by the local environment is not just simply one of mass and volume changes induced mortar cubes. It is also a progressive weakening of the cubes mass itself. The following points have been discovered:

1. Evidence obtained through tests on cement-based mortar cubes proves that both of the mass and volume of cubes have affected by the application of local environmental notable situations.
2. Experiments prove that the Iraqi cement, Bazian-Tasluja, has recorded the highest ability to resist the surrounding local environmental issues.
3. Evidence shows that mechanical properties were affected by the applied local environmental issues on long term timescale.
4. The approach of using several tests on cement-based mortar cubes demonstrates correlations between the findings of those methods and the external climate factors. Strain of mortar cubes is clearly related to adhesion points among mortar components.

It is recommended to apply longer duration environmental situations on cement-based mortar cubes and may extent to hardened concrete, which might result in more useful meanings.

APPENDIX

- Normal Specimens After Comprrestive Strength Test

- Saturation-Affected Specimens After Compressive Strength Test

- Temperature-Affected Specimens After Compressive Strength Test

LIST OF ABBREVIATIONS

Abbreviation	Definition
A1, A3, A7 & A28	Mortar cubes made from local cement type (Karasta), that were cured for 1,3, 7 & 28 days before subjected to any case of local environment issues.
B1, B3, B7 & B28	Mortar cubes made from local cement type (Bazian-Tasluja), that were cured for 1,3, 7 & 28 days before subjected to any case of local environment issues.
C1, C3, C7 & C28	Specimen made from Iranian cement type (Gharb), that were cured for 1,3, 7 & 28 days before subjected to any case of local environment issues.
D7 & D28	Specimen made from Turkish cement type (Kalekim), that was cured for 7 & 28 days before subjected to any case of local environment issues.

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REFERENCES

[1] ASTM International, "Standard Test Method for Compressive Strength of Hydraulic Cement Mortars" (Using 2-in. or [50-mm] Cube Specimens), vol. 04.01.
[2] D. Yan and G. Lin, "Dynamic Properties of Concrete in Direct Tension," in Cement and Concrete Research, vol. 36, 2006 pp. 1371-1378.
[3] E. Cadoni, K. Labibes, C. Albertini, M. Berra, and M. Giangrasso, "Strain-rate effect on the tensile behaviour of concrete at different relative humidity levels," in Materials and Structures, vol. 34, 2001, pp. 21-26.
[4] F. Radjy and C. W. Richards, "Effect of curing and heat treatment history on the dynamic mechanical response and the pore structure of hardened cement paste," Cement Concrete Res., vol. 3, pp. 7-21, 1973.
[5] H. Hotta, and K. Takiguchi, "Influence of drying and water supplying after drying on tensile strength of cement mortar," Nucl. Eng. Des., vol. 156 (6), pp. 218-228, 1995.
[6] J. Zhou, X. Chen, L. WU and X. Kan, "Influence of free water content on the compressive mechanical behaviour of cement mortar under high strain rate," Sadhana, vol. 36, part 3, pp. 357-369, 2011.
[7] L. Konecny and S. J. Naqvi, "The effect of different drying techniques on the pore size distribution of blended cement mortars," Cem. Conc. Res., vol. 23, pp. 1223-1228, 2003.
[8] N. C. Collier, J. H. Sharp, N. B. Milestone, J. Hill and I. H. Godfrey, "The influence of water removal techniques on the compositions and microstructure of hardened cement pastes," Cement Concrete Res., vol. 38, pp. 737-744, 2008.
[9] P. Rossi, "A physical phenomen which can explain the mechanical behaviour of concrete under high stain rates," Mater. Struct., vol. 24, pp. 422-424, 1991.
[10] P. Rossi, J. G. M. Van Mier, C. Boulay and F. L. Maou, "The dynamic behavior of concrete: influence of free water," Material and structures, vol. 25, pp. 509-514, 1992.
[11] R. F. Feldman and J. J. Beaudoin, "Pretreatment of hardened cement paste for mercury intrusion measurements," Cement Concrete Res., vol. 21, pp. 297-308, 1991.
[12] Vu, X. H., Malecot Y., Daudeville L., and Buzaud E.: Effect of the Water/Cement Ratio on Concrete Behaviour under Extreme Loading. Int J. Numr Analysis Methods Geomech, 2009, vol. 33, pp. 1867-1888.

Dr Nihad B. S. Baban The author is obtained his BSc and MSc from Iraq and his PhD from the UK. His main speciality is Civil Engineering and specific Speciality is Geotechnical Engineering. He is working in the academic fields since1999.

He taught many subject of Civil engineering field such as: Soil Mechanics, Concrete Technology, Building Materials and Construction, Foundation Engineering, Construction Engineering and Engineering Laboratory.

Dr Baban research interest is mainly on building materials and earth existing issues and their influences on buildings.

Experimental and Theoretical Study of Temperature Distribution of High Strength Concrete Exposed To Fire Flame

Mohammed K. Al Khafaji[A]

Asst. Prof. Dr. **Waleed A. Abbas**[B] *Prof. Dr.* **Shakir Al-Mishhadani**[C]

Abstract **The mechanical and physical properties of concrete depend mainly upon the micro ponds between mixed constituents of hardened concrete members. Accordingly a great attention should be focused to the exposure of hardened concrete members to high temperature to conceptualize a good understanding of burned members. Two mathematical 3d models are developed to investigate and conceptualize the distribution of burning temperature in concrete models moreover a measured laboratorial burning temperature readings are obtained by using a special burner manufactured for this purpose. Two types of models are prepared; they are cubic (150mm*150mm*150mm) and prismatic (150mm*150mm*500mm) models are heated from one face to 400ºC, 600ºC and 800ºC. The mathematical models and the practical burning system are justified for unique boundary conditions in steady state condition. The mathematical and measured temperatures are matched. It is found that the Thermal differences of all matched nodal point are less than ±10%.**

Index term: Burning Temperature, Cubic Model, Prismatic Model, Thermal Difference, Nodal Point.

I. Introduction

The concrete nowadays becomes the most favorable and the major part in the development of civilization of the new world. However the phenomenon associates with the exposure of concrete members to high temperatures still requires more studies to understand the distribution of temperature with time across the specimens exposed to fire flame. There are some studies investigated this problem, among them Khaled [1], Setyowati et. al. [2], Alkafaji and Alkaizwini [3]. It is preferred to investigate the effect of burning temperature on concrete member. The estimated temperature from the finite element model were used to estimate the thermal strain and potential for cracking using procedure outlined in ACI 207 ,Vik Iso-Ahola, et. al.[4]. Three dimensional nonlinear model that can accurately predict the temperature distribution at any location with beam reinforced with GFRP bars when exposed to the standard fire curve ISO834, Hawileh [5]. The model is validated by comparing the predicated average temperature in the GFRP bars with the measured experimental data obtained by Abbasi and Hogg [6]. Concrete is generally treated as a homogeneous isotropic material in heat analysis, the temperature dependence of concrete thermal properties has an important effect on heat transfer analysis Zhou and Vecchio [7]. Moreover, the temperature-dependence on the mechanical properties will significantly affect the subsequent structural analysis. This job can be done by a comparison between the theoretical and measured temperature of a cubic and prismatic concrete models. Prickett and Lonngquist [8], AL-Assaf [9], and Lateef [10], presented that the max difference between the theoretical and simulated results should not exceed (±10%).It is worth to study the distribution of burning temperature over cubic and prismatic concrete models in steady state condition and making a comparison between a theoretical results based upon the solution of partial differential heat transfer equation and the measured temperature.

II. Properties of Papers Submission

Properties of Concrete Model Exposed To Fire Flame

A concrete cubes of dimensions (150mm*150mm*150mm) and a prism with dimensions of (500mm*150mm*150mm) shown in Fig.(1a and 1b) are prepared by using a high strength concrete mix with a mix proportion by weight of 1:1.22:2 and w/c ratio 0.30 and nano metakaolin cement replacement of 3% with 80 MPa target compressive strength for 28-day, these specimens were exposed to a direct fire in one face and other five faces are left exposed to the resulting kiln space temperature. It is worth mentioning that flame temperature is controlled by the kiln thermometer. Anyhow three phases of temperature are used to test the concrete models; they are 400ºC, 600ºC and 800ºC. After steady state condition (no change in temperature with time) has been reached, the resulting temperature is measured by measuring device over other face of the concrete samples.

Analytical Consideration of Laboratorial and Theoretical Results

The comparison between the burning temperature distribution over the whole body of concrete models requires a:-

1- Simulation of laboratorial measuring temperatures.
2- Simulation of analytical temperatures which based on the solution of the three dimensional partial differential equation in homogenous isotropic media by using a Cartesian coordinate of Fig.(1c).
3- Matching the results of a numerical model based on the solution of the heat transfer equation in 3D media and the practical results obtained from tested specimen by using the laboratorial burner.
4- The maximum thermal difference between the laboratorial and numerical results of temperature

A. Lecturer in Al-Furat Al-Awsat Technical University/ Babylon Institute/ Civil Department.
B. Asst. Prof. in Technical University/ Building and Construction Department.
C. Prof. in Technical University/ Building and Construction Department.

should not exceed ±10% in the concrete models as preceded previously.

Temperature Distribution by Using Surfer Program Development of Mathematical Model

Analytical solutions of partial differential equations for one, two, and three dimensional, non-steady flow of heat transfer in homogenous isotropic media, have been derived for certain initial and boundary conditions by many workers, but for heterogeneous anisotropic media, there is no general analytical solution. The use of numerical solution technique will not only help in dealing with heterogeneous aquifers but represent a sophisticated technique to deal with large scale media development problems. The two famous methods which may be considered as the bases of modeling techniques are:

1- Finite difference method.
2- Finite element method.

These techniques could not be used before the 1960s due to huge computations which are required in their application. However after the rapid development of digital computers, the matter became completely different. Computers are now widely used to solve large sets of simultaneous algebraic equations, that are usually arises in the numerical treatment of boundary-value problems using the computer program surfer (surving program). Mathematical models are prepared for the quantitative study of cubic and prismatic shapes to simulate the heat transfer system. The model solves a set of finite difference equations which are regarded as numerical solution of the partial differential equation governing steady state heat transfer in heterogeneous media.

Mathematical Background of 3D Heat Transfer Equation

The numerical solution of partial differential three dimensional (3D) heat transfer equation begins with the general form of 3D heat transfer equation [Kreyszig (2011)] in any system of the Equations (1 and 2)

Eq.(1 to 4) may be solved in steady state by using a significant boundary conditions. Heat equation over the cube of Fig.(1-a) and prism bar of Fig.(1-b) concrete samples with a fix dimensions in steady state condition simplifies Eq.(2 to4) to be in Eq. (3).

The facilitation of Eq.(3 to 4) and by using the Cartesian coordinates of Fig.(1-c), it is assumed that the concrete models are:-

1- Homogenous isotropic media.
2- Symmetrical corresponding to their geometrical dimensions, accordingly. As in Eq. (4).

Mathematical Background of the Simulation Model Grid System Design

The first step in modeling job is to discretize the model domain by superimposing a mesh of finite difference grid over the area of samples. The size of mesh or in other words the number of rows and columns that is to be adapted depends on the required accuracy. The total dimensions of the grids are defined by NC (the number of columns), and NR (the number of rows) of the model. Non-uniform grid spacing are used in this analysis; however uniform spacing is used in the present work The chosen number of columns and rows in the present models are as in Table (1).

Uniform grid spacing of (1 cm) is selected in X and Y directions. Briefly, Fig.(2) and Fig.(3) shows the discretization of cubic and prismatic models under interest.

Several programs have been written for simulation of heat transfer by a mathematical modeling techniques, using a finite difference approach. It is preferred to write a new one in Fortran Language. It is mainly made to facilitate the modification process of the program and the input/output data settings and filing in addition the computational technique is rather a straightforward procedure.

Laboratorial Thermal Treatment and Measurements

Both concrete models are exposed to heat sources of one face in the kiln up to a maximum temperature of 400ºC, 600 ºC, and 800ºC. While the other five faces exposed to the space formed kiln temperature. The heating process is continued until a steady state heat condition is obtained (no changed in temperature degrees with time) which is approximately obtained after 1.5hr of heating. The heat is measured for many grids within the domain at the mid of concrete sample by using a special measuring device.

Justification of Model Results

The acceptance of thermal difference between the measured and theoretical distributed temperature is based on Eq. (5)

Cubic Model Simulation Results& Discussion

The best temperature representation method is a contour map technique. Surfer 7 software has been chosen to achieve this job.

The measuring temperature over the non-burning faces and the theoretical results in steady state condition are matched for the burning phases of 400ºC, 600ºC and 800ºC as in Figs. (4, 5, and 6) respectively.

The results show a good matching between the simulated analytical results and the practical (measured) results of difference less than ±10% as referenced as shown in Table (2).

Prismatic Concrete Model Simulation Results

The preceding burning flame temperatures in a hardened prism beam are excreted along the long face for enough time until a steady state condition is obtained. The measuring and numerical results are also matched as in Figs.(7, 8, and 9).The Thermal difference (Δf) matched results of the theoretical and numerical results are generally less than ±10% as shown in table (3).

Temperature Distribution by Using ANSYS Program

ANSYS (Analysis System) is a comprehensive general. Purpose finite element computer program that contains over (100,000) lines of code and more than (180) different elements. It can be used in many engineering fields, including structures and thermal, AL-Shimmari et al. [12].

In order to investigate failure in this study, 3D element with 8 nodes was used to model the concrete [solid-65 and solid-70]. In the present study, the ANSYS program of version (11) was employed for analyzing tested cubes and prisms as well as the finite element modeling for concrete.

Concrete is generally treated as a homogeneous isotropic material in heat analysis, the temperature dependence of concrete's thermal properties has an important effect on heat transfer analysis. Moreover, the temperature dependence of the mechanical properties will significantly affect the subsequent structural analysis. Temperature-dependent thermal properties (conductivity and specific heat) and physical properties (density) make heat analysis nonlinear

since the coefficient matrices in the final resultant equation are not constant but dependent on the temperature, Zhou et al.[7].

The basis for thermal analysis in ANSYS is a heat balance equation obtained from the principle of conservation of energy. The distribution of thermal elastic stress components were then calculated by switching the solid (65 and 70) concrete and thermal element, Dahmani[13].

Assuming that no heat generation rate exist in the hardened concrete. The finite element solution performed via ANSYS calculates nodal temperatures and then uses the nodal temperature to obtain other thermal quantities.

Three dimensional incremental finite element thermal analysis were performed using ANSYS software to estimate the temperature distribution within the concrete samples. The estimated temperatures from the finite element model were used to estimate the thermal strain and potential for cracking using procedures outlined in ACI207, Iso-Ahola et al.[4]. Using computer program ANSYS version 11.

The study of specific mass, specific heat, thermal conductivity and diffusivity is important for the development of temperature gradients, thermal deformation and concrete early-age cracking. It is possible to finite solutions more suited to the problem, ANSYS software, that can be used to allows the identification of the highest temperature achieved, heat evolution time and critical temperature spots on structure. Heat generation inside a concrete block generates a temperature gradient in relation will cause cracking and possibly damages to the external edges, Nailde et al. [14]

Thermal Analysis

A thermal version of the model was used to calculate the temperature profile in the concrete cube and prism, a structural version of the model then read the temperature profile to calculate stresses.

The boundary conditions are implemented and the problem is solved using ANSYS program. The temperature distribution results are obtained in the general post processor. The results so obtained are plotted in Fig. (10) to Fig. (15) for the temperature profiles.

A cracking accumulation and propagation around the cube and prism with the increase of temperature. The result obtained by finite element solution showed good agreement with experimental results, AL-Shimmari et al. [12].

Theoretical foundation

One of the main objective of canalizing thermal effect is to determine the temperature field in a medium resulting from the conditions imposed on concrete, the intention is to know the distribution of temperatures, and heat flow conduction that passes through the concrete body, and by consider a homogeneous medium of concrete on any point.

The cracking state of concrete prism and cube at 800°C temperature is illustrated in Figs. (14) and (15). It can be observed that micro cracks uniformly distributed over the concrete prism and cube propagate with the increasing of temperature.

Surface cracks took place increased in number, length on the concrete specimen, specially. When increased temperature and exposure duration. These result agreed with that Abram's, Neville and Brooks.

III. Conclusions

Based on the test results and theoretical analysis of the present work, the following conclusions can be drawn:

1- There is a good matching between the measured and the numerical results of temperature distribution since the thermal differences between them consequently less $\pm$10%.
2- The burning time of 1.5 hrs is proper to obtain a steady state condition, otherwise contours are contradicted.
3- A good agreement between the measured experimental and predicted finite element simulation was obtained for the average temperature in the concrete prism and cube at all stages of fire flame exposure. The finite element modeling could provide full field of results, in terms of 3D temperature distribution, it could be concluded that the developed finite element model is a great tool to aid designers to predict numerically the temperature distribution of concrete specimens. The validated model could be used as a valid tool of experimental testing and further investigation of the fire performance of concrete under different applied fire boundary conditions.
4- The high temperature gradient between the base and the top of the prism and cube induce the appearance of high values of tensile stress which could worsen the static behavior of the concrete prism by causing the cracking of the concrete, with thermal induced stresses in the concrete to high gradient temperature.

IV. Helpful Hint.

A. Tables and Figures.

TABLE 1
NUMBER OF COLUMNS AND ROWS

	NC	NR
Cubic Model	15	15
Prism Model	50	15

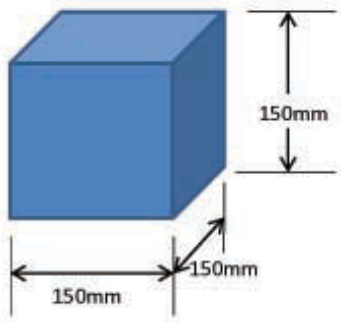
TABLE 2
THERMAL DIFFERENCES OF 800ºC BURNNG TEMPERATURE IN SELECTED NODES OF CUBIC CONCRETE MODEL

Nodal No.	Measured Temp. ºC	Numerical Temp. ºC	Δf %
1	478.061	492.277	2.986
2	587.516	562.235	4.303
3	626.856	575.343	8.217
4	478.061	492.277	2.973
5	372.248	400.132	9.938

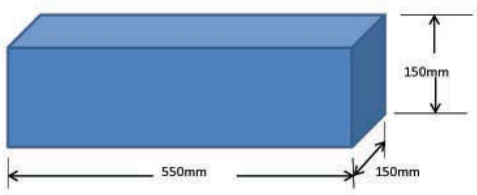
TABLE 3
THERMAL DIFFERENCES OF 800ºC BURNNG TEMPERATURE IN SELECTED NODES OF PRISMATIC CONCRETE MODEL

Nodal No.	Measured Temp. ºC	Numerical Temp. ºC	Δf %
1	603.713	602.971	-0.123
2	695.928	665.517	-4.370
3	660.000	668.155	1.236
4	694.544	665.517	-4.179
5	604.713	603.197	-0.251

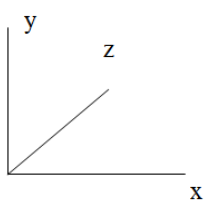
6	373.354	387.139	3.692
7	531.268	481.762	-9.318
8	479.999	485.285	1.101
9	528.459	480.192	-9.134
10	373.354	387.139	3.692



a- Cube Concrete



b- Prism Concrete Bar



c- Cartesian Coordinates

Fig. 1. Concrete Members

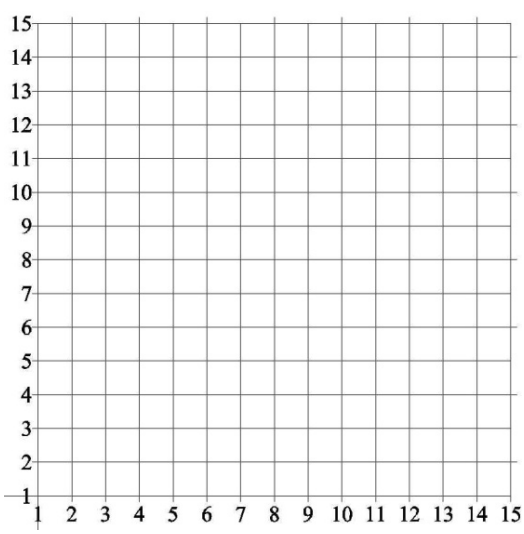


Fig. 2. Mesh Design of the Cubic models

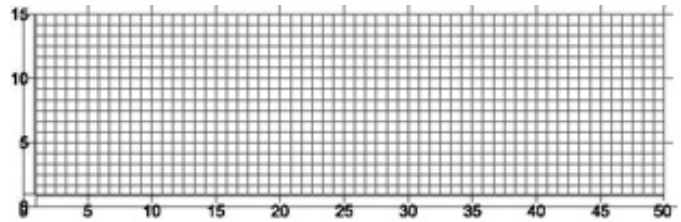


Fig. 3. Mesh Design of Prismatic Models

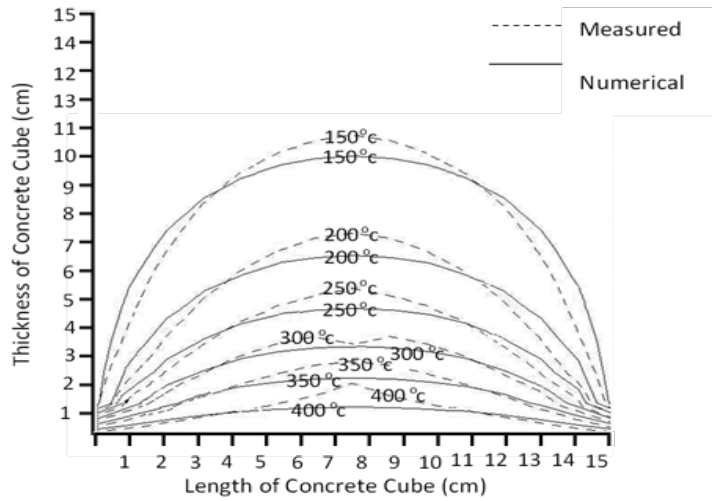


Fig. 4. Contour Map of Burning Temperature Distribution for the Concrete cubes at (400ºC)

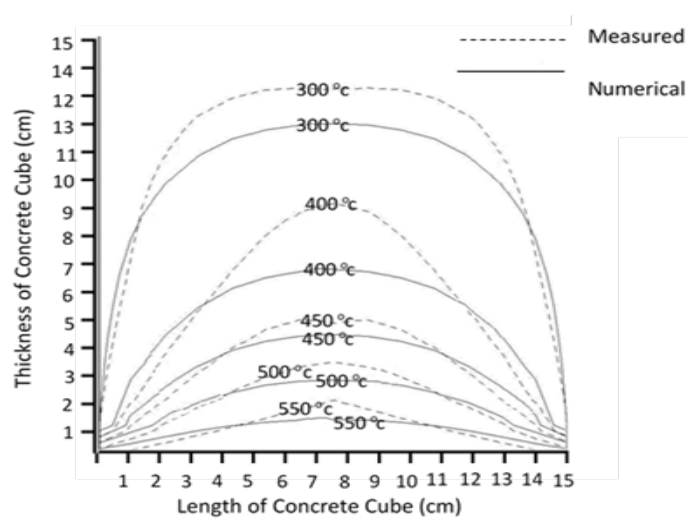


Fig. 5. Contour Map of Burning Temperature Distribution for Concrete Cubes at (600ºC)

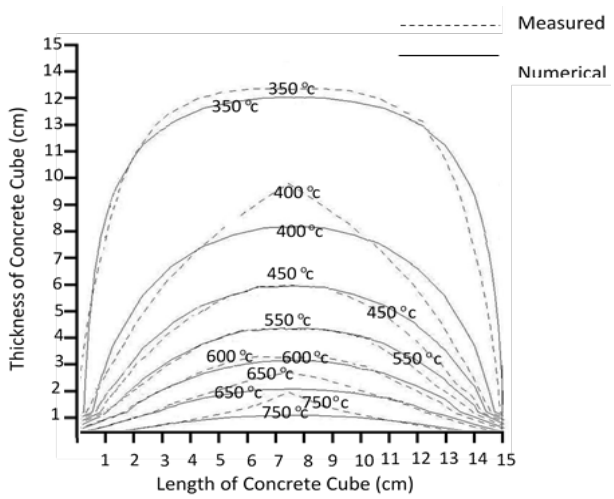


Fig. 6. contour Map of Burning Temperature Distribution for Concrete Cubes at (800ºC)

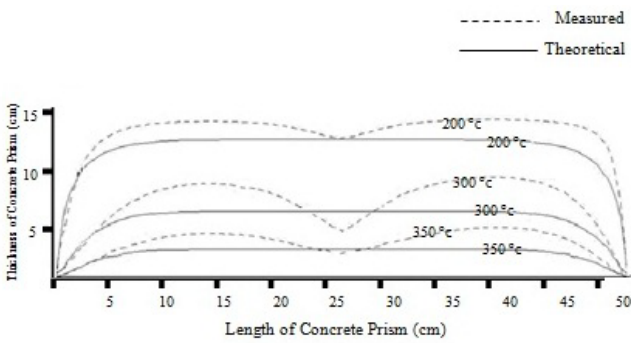


Fig. 7. Contour Map of Burning Temperature Distribution for Concrete Prism at (400ºC)

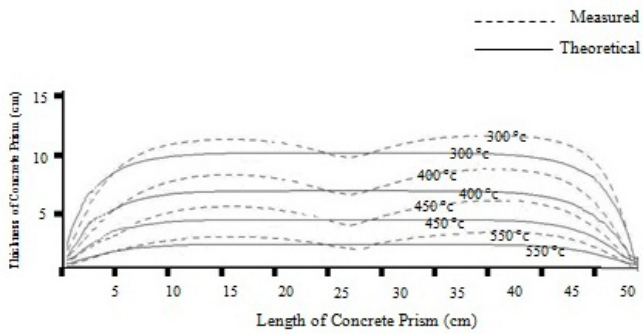


Fig. 8. Contour Map of Burning Temperature Distribution for Concrete Prism at (600ºC)

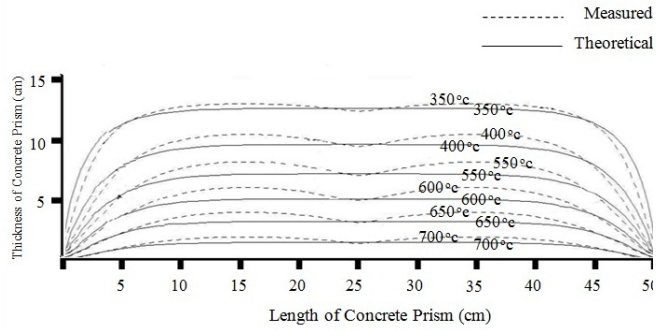


Fig. 9. Contour Map of Burning Temperature Distribution For Concrete Prism at (800ºC)

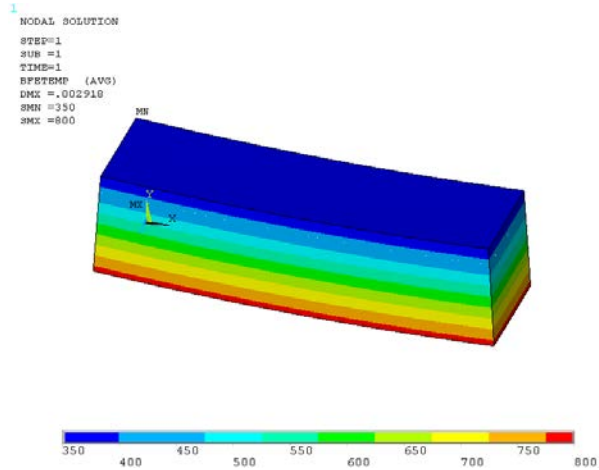


Fig. 10. Temperature distribution along the prism after 2 hours of fire flame exposure

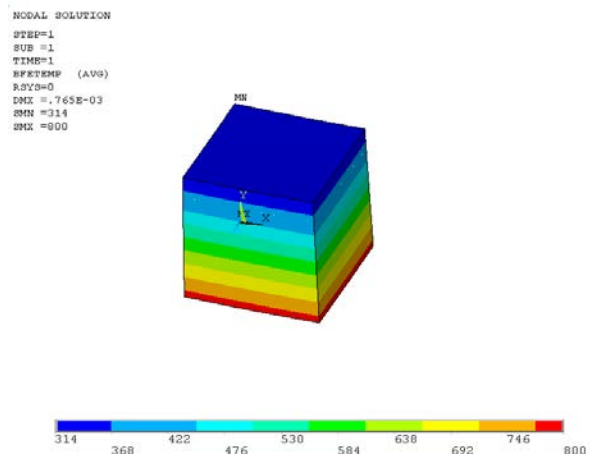


Fig. 11. Temperature distribution along the cube after 2 hours of fire flame exposure

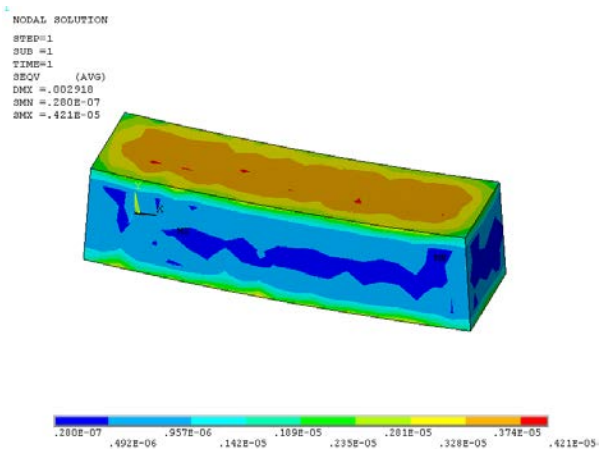


Fig. 12. Deformed shape stresses distribution along the prism after 2 hours of fire flame exposure

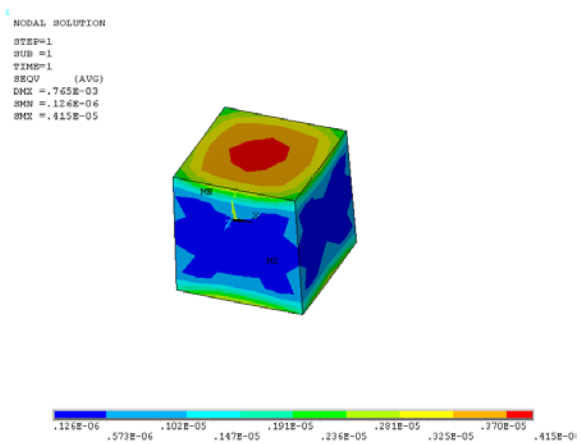


Fig. 13. Deformed shape and stresses distribution along the cube after 2 hours of fire flame exposure

Fig. 14. Thermal cracking of prism at 800°C

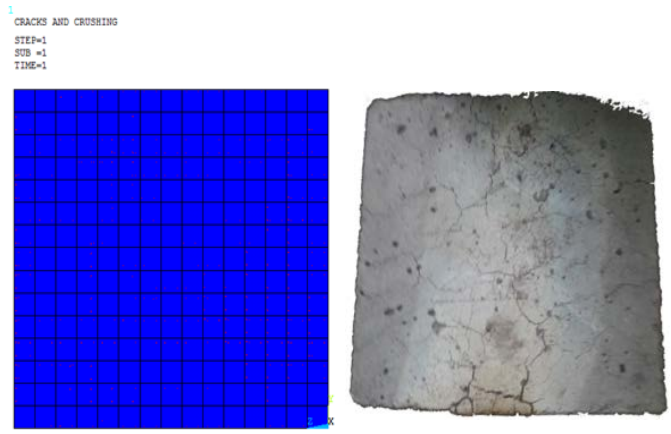


Fig. 15. Thermal cracking of cube at 800°C

B. Equations

$$\frac{\partial u}{\partial t} - \alpha \nabla^2 u = 0 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

In Cartesian system it may be written as:-

$$\frac{\partial u}{\partial t} - \alpha\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2}\right) = 0 \qquad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} = 0 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$k_x = k_y = k \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Where:-

$$\boldsymbol{\alpha} = \frac{\boldsymbol{k}}{\boldsymbol{\rho C_p}}$$

α: *thermal diffusivity*,
k: *heat conductivity*,
ρ: *mass density*,
C_p : *specific heat capacity*

$$\Delta f = \frac{ABS(R - £)}{£} * 100 \qquad \ldots\ldots\ldots\ldots\ldots (\mathbf{5})$$

Where:
Δf: *Thermal difference percentage*,
R: *measured temperature value in degree centigrade* ,
£: *Theoretical temperature value in degree centigrade*

References

[1] Abbasi, A., & PJ. Hogg, ,"Fire testing of concrete beams with fibre reinforced plastic rebar". Composites: part A, Applied Science and Manufacturing, Vol.37, (2006), pp. 1142–1150.

[2] Khaled, A., "Improving Fire Resistance of Reinforced Concrete Columns" A thesis submitted to Civil Eng. Dept., Islamic University of Gaza as a part fulfillments of master degree of science, (2011).

[3] Hawileh, A. Rami, "Heat Transfer Analysis of Reinforced Concrete Beams Reinforced with GFRP Bars", American University of Sharjah, United Arab Emirates, pp. 299-314.

[4] Zhou, C. E.; F. J. Vecchio, "Nonlinear finite element analysis of reinforced concrete structures subjected to transient thermal loads", Computer and Concrete, Vol. 2, No. 6 (2005), pp. 455-479.

[5] Setyowati, E W., A. Soehardjono, A. Zacoeb, A. Fuad, and N. Mufti, "Micro Structure Effect of Concrete Degradation for Compressive Strength of Concrete Burned in High Temperature"International Journal of Emerging Technology and Advanced Engineering Website: www.ijetae.com (ISSN 2250-2459, ISO 9001:2008 Certified Journal,Volume 2, Issue 12, December 2012).

[6] Kreyszig, Erwin ., "Advance Engineering Mathematics" 10th. Ed., 2011.

[7] AL-Shimmari, Kh. Israa, T. Nagham Hamad and A. Waleed Waryosh,"Investigation of the Behavior for Reinforced Concrete Beam Using Non-Linear Three-Dimensional Finite Elements Model". Eng. & Tech. Journal, Vol. 29, No.10, 2011.pp1870-1885.University of Technology, Baghdad, IRAQ.

[8] Dahmani, L., , A. Khennane, , and S. Kaci, "Crack Identification in Reinforced Concrete Beams using ANSYS software " , Strength of Material Journal, Ed. Springer New York, Vol.42, No.2, 2010, pp.232-240.

[9] Alkafaji, M. M. and S. S. Alkaizwini, " Effect of Burning By Fire Flame on Load Carrying Capacity of Self-Compacting Concrete Columns", Journal of Engineering Research and Applications , www.ijera.com Vol. 3, Issue 5, Sep-Oct 2013, pp.135-148.

[10] Lateef, M. Najah, "Management of Surface and subsurface management water of Al Adhaim Basin" Phd. Thesis, Unviversity of Baghdad (2005).

[11] Nailde, de Amorim Coelho, Lineu José Pedroso, João Henrique da Silva Rêgo and Antônio Alberto Nepomuceno, (2014); "Use of ANSYS for Thermal Analysis in Mass Concrete". Journal of Civil Engineering and Architecture, ISSN 1934-7359, Volume 8, No. 7(Serial No. 80), pp. 860-868, USA.

[12] AL Assaf, S. A. , "Application of Computer Technique to Groundwater Flow Problems", Dpt. of Geology, Univ. College London, (1976).

[13] Prickett, T.A., and C. G. Lonngquist, "Selected Digital Computer Techniques for Groundwater Resource Evaluation", (Illinois State Water Survey Bulletin 55.) 62 pp., (1971).

[14] Vik, Iso-Ahola; Bashar Sudah and Vincent Zipparro, "Concrete Thermal Strain, Shrinkage and Cracking Analysis", Innovative Dam and Levee Design and Construction for Sustainable Water Management. 32nd. Annual USSD Conference New Orleans, Louisiana, USA. April 23-27, 2012 pp. 345-377.

Use of Recycled Aggregates and Recycled Glass in Manufacturing Precast Concrete Hollow Blocks

Pierre Matar, Louay El Hassanieh, Marleine Bayssary

***Abstract*—The objective of this study is to determine the influence of the use of recycled aggregates and recycled glass on the compressive strength and the behavior in fire of precast concrete hollow blocks. Tests are carried out on four series of blocks manufactured using concrete mixes with 30% replacement of coarse natural aggregates with recycled aggregates and/or recycled glass, and one series of control blocks manufactured using concrete mixes not containing any recycled materials.**

***Index Terms*—behavior in fire, compressive strength, precast concrete hollow blocks, recycled glass, recycled aggregates**

I. Objectives

THE demolished concrete is a major component of the construction and demolition (C&D) waste. The recycled aggregates obtained by crushing the demolished concrete can be used as a substitute of natural aggregates in new concrete mixes. Another major C&D waste is the flat glass. This glass can be also recycled and used as an aggregate substitute in concrete.

The objective of this study is to determine the influence of the use of recycled aggregates and recycled glass on the compressive strength and the behavior in fire of precast concrete hollow blocks. Tests are carried out on four series of blocks manufactured using concrete mixes with 30% replacement of coarse natural aggregates with recycled aggregates and recycled glass, and one series of control blocks manufactured using concrete mixes not containing any recycled materials.

II. Materials and Methods

The materials used to manufacture the concrete blocks are the following:

--Ordinary Portland cement with specific gravity 3.15.

--Natural aggregates 0/12.5 mm consisted from crushed stones.

--Recycled aggregates 6.3/12.5 mm obtained by crushing demolished precast concrete elements (beams, lintels and paving blocks) made with mixes containing limestone aggregates and having various values of compressive strength.

--Recycled glass 6.3/12.5 mm obtained by crushing demolished flat (windows) glass.

--Water from potable fresh water source free from deleterious materials.

--High range water reducing superplasticizer Viscocrete 20HE provided by Sika.

Five series of four blocks each are tested. Four of these series are fabricated with partial replacement of coarse natural aggregates with recycled materials; the recycled aggregates and recycled glass are used with a replacement rate of 30%. The fifth series (control blocks) does not contain any recycled materials.

The following characteristics of aggregates are determined: the acid soluble chloride according to BS EN 1744 P5, acid soluble sulphate content according to BS EN 1744 P1, the potential alkali-silica reactivity (chemical method) according to ASTM C289, the density, specific gravity and water absorption according to ASTM C127 and ASTM C128, the content of material finer than 75 mm according to ASTM C117, the sand equivalent of soil according to ASTM C2419, and the dry unit weight according to ASTM C29/C29M.

The manufactured concrete hollow blocks are tested according to ASTM C140 to determine their compressive strength. The blocks are weighed, and their dimensions, volume, density and compressive strength are determined. The compression tests are performed at 7 days; the load is applied with a constant speed of 3 kN/s.

A non-standardized test method is used to examine the behavior of the blocks in fire. This test method consists of applying a direct butane flame on the block surface with duration of two hours in order to identify the effect of the fire on the appearance of the blocks.

III. Results

The analysis of results is done based on a comparison of the characteristics of blocks containing different rates of recycled aggregates and recycled glass with a total of 30%, as well as on a comparison of the characteristics of these blocks with the

P. Matar is with the Lebanese University, Faculty of Engineering, Beirut, Lebanon (corresponding author phone: 961-3369960; fax: 961-4872206; e-mail: pmatar@ul.edu.lb).

L. El Hassanieh was a student at the Lebanese University, Faculty of Engineering, Beirut, Lebanon (e-mail: louay.hassanieh@hotmail.com).

M. Bayssary was a student at the Lebanese University, Faculty of Engineering, Beirut, Lebanon (e-mail: marleine_bay@hotmail.com).

characteristics of control blocks.

The test results show that the blocks containing recycled aggregates and/or recycled glass have a density between 2566 kg/m^3 and 2590 kg/m^3. Blocks containing recycled aggregates have the lower density due to the lower density of recycled aggregates.

The blocks not containing recycled materials have the highest compressive strength, i.e. 3.66 MPa. The blocks containing different combinations of recycled materials have almost similar compressive strength values: 2.90 MPa for blocks containing 30% recycled aggregates, 2.86 MPa for blocks containing 20% recycled aggregates and 10% recycled glass, 2.77 MPa for blocks containing 10% recycled aggregates and 20% recycled glass, and 2.75 MPa for blocks containing 30% recycled glass. The loss in the compressive strength compared to the control blocks is equal to 21%, 22%, 24% and 25% respectively.

The fire test shows hair cracks of less than 1 mm thickness on the surface of blocks after five to fifteen minutes and no damage occurs after two hours.

IV. Conclusions and Recommendations

This research shows the influence of the use of recycled aggregates and recycled glass in precast concrete hollow blocks. Based on the obtained results of laboratory tests, the following conclusions can be drawn:

1. The replacement of coarse natural aggregates by recycled aggregates and/or recycled glass at the level of 30% decreases the compressive strength of concrete blocks by 21% to 25% compared to blocks not containing recycled materials.
2. Concrete blocks containing different combinations of recycled aggregates and recycled glass as replacement of coarse natural aggregates at the level of 30% have comparable compressive strength, close to 2.86 MPa.
3. This replacement does not affect the behavior of blocks in fire.
4. Blocks containing recycled aggregates have the lower density due to the lower density of recycled aggregates.
5. It is feasible to use recycled aggregates and recycled glass as coarse aggregates in the production of precast concrete hollow blocks for partition walls or for walls bearing reduced loads.

Acknowledgment

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Using slag as a partial replacement of sand contain high sulfate in concrete mixes

Dr.Faiza A. Albarazanjy Mowaffak Z. yousif Dr.Suhair K. Al-Hubboubi

***Abstract*—Steel slag is a byproduct obtained either from conversion of iron to steel in a Basic Oxygen Furnace (BOF), or by the melting of scrap to make steel in the Electric Arc Furnace (EAF).The local slag is produced from the Electric Arc Furnace (EAF) by slow cooling. Damping away this by product represents a waste of the material and causes serious environmental pollution problems. In this research, concrete mixes were prepared by using deferent percentages of fine slag such as (15, 30, and 45) % by weight of cement as a partial replacement of sand. The using aggregates (sand and gravel) contain high sulfate higher than the limitations of I.Q.S 45.Compressive, flexural strengths and shrinkage tests were performed to evaluate the concrete mixes.Results showed that the use of slag as a partial replacement of sand improve compressive strength up to 18% comparing to reference concrete .Slag also improves the workability of concrete without need to admixtures. Expansion and shrinkage decreased for concrete containing slag, which means more stability and less micro cracking in concrete. Due to the low content of sulfate in slag, this fact means that contaminated sulfate sand can be used when partially replaced by slag.**

***Index Terms*— Concrete, Sand, Slag, Sulfate.**

I. Introduction

Blast furnace slag is a nonmetallic material consisting of silicates and aluminosilicates of calcium and magnesium together with other compounds of sulfur, iron, manganese, and other trace elements. It is produced from a molten state simultaneously with pig iron in a blast furnace.

Depending on the composition of the raw material, the fusion temperature, and the cooling rate, a variety of minerals can form. The glass content is mainly dependent on the cooling rate, with faster cooling resulting in the formation of more glass, whereas slower cooling allows more time for the formation of crystallized minerals. This is very important, as glassy phases are chemically more reactive, and is why rapidly quenched granulated slag can be ground and used as cement.

Air-cooled blast furnace slag (ACBFS) is produced through relatively slow solidification of molten blast furnace slag under atmospheric conditions, resulting in crystalline mineral formation [(1)]. Blast furnace slag can be used in preparing constructional materials, either by adding it to cement clinker in order to produce (Portland blast furnace cement) which increases concrete durability by decreasing corrosion of steel reinforcement and increasing sulfate attack and freezing and thawing resisting.Slag can also use as an additive to concrete when used as aggregate enhancing concrete properties [(2,3,4)].

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Faiza Abdullkader Albarazanjy is with the Building Research Directorate / Ministry of Const. and Housing, Iraq.

Mowaffak Zainalabdin yousif was with the Building Research Directorate / Ministry of Const. and Housing, Iraq. He is now a Consultant Engineer in Canada. e-mail: mowaffakyousif@yahoo.com.

Suhair Kadhem Alhubboubi is with the Building Research Directorate / Ministry of Const. and Housing, Iraq; e-mail: suhairkah@yahoo.com.

II. Experimental work

A. Materials

Ordinary Portland cement conforming to I.Q.S 5:1984. Blast furnace slag from Al Nasir factory was crushed and grounded to the fineness of (75-150) µm. Table (1) showed the chemical composition of slag and cement. Zone (3) natural sand and crushed gravel of 5-20 mm maximum size according to I.Q.S 45. Sulfate content (SO_3) of sand and gravel were (1.22, 0.23) % respectively which are higher than the limitations of I.Q.S 45.

TABLE 1
CHEMICAL COMPOSITION OF SLAG AND CEMENT

Oxide	Slag	ACI 226-IR87 for blast furnace slag	Ordinary Portland cement	I.Q.S 5:1984 Limits for cements
SiO_2	26.01	32-40	19.45	
CaO	26.6	7-17	60.68	
Al_2O_3	8.28	29-42	5.36	
Total iron as (FeO or Fe_2O_3)	24.07	0.1-1.5	3.37	
MgO	8		5	<5
SO_3	0.07	0.7-2.2	1.87	<2.8
Na_2O+K_2O	0.24			
L.O.I	-	-	3.86	<4

B. Mixes preparation and testing

Reference concrete mix (A) prepared with cement: sand: gravel ratio of 1:1.71:3.46. Part of the sand was replaced by (15, 30, 45) % of cement weight (A1, A2, A3) respectively, water/cement ratio for all mixes was 0.55. Cube molds of (10*10*10) cm, prisms of (10*10*60) cm and prisms of (7.5*7.5*28) cm were used for making specimens of compressive, flexural strength and shrinkage tests respectively. Specimens were cured after demolding in water tanks until the ages of tests (7, 28 and 90 days ages for cubes and 90 days age for flexural strength. (7.5*7.5*28) cm prism specimens were divided into two groups one kept in water and the other in an oven at (50°C Temp.) until ages of tests. The first measurements were conducted immediately after demolding. Table 2 represents the concretes mix proportions.

TABLE 2
CONCRETE MIXES PROPORTIONS

Concrete mixes	Mixing percentage Cement: Sand: Gravel	% Slag of cement weight	%Slag of sand weight
A	1: 1.71 :3.46	0	0
A1	1: 1.56 :3.46	15	8.8
A2	1: 1.41 :3.46	30	17.6
A3	1: 1.26 :3.46	45	26.3

III *RESULTS AND DISCUSSION*

A. Consistency and setting time

Partial replacement of sand by slag as a percentage of cement weight 15%, 30% and 45% reduces the mixing water to achieve the same consistency ,also it increases the setting time (initial and final) so the slag acts as a retarder. This is may be because the slag attracts water molecule faster than the cement, then giving it back, this operation delays the hydration reaction of cement compounds though makes setting time longer. Table 3 represents the results.

TABLE 3
CONSISTENCY AND SETTING TIME ACCORDING TO (IQS NO.5/1984)URES AND TABLES

Concrete mixes	Plasticity %	Setting time(Vicate apparatus), Initial setting, hour: minute	Final setting, hour: minute
A	27	1:25	2:00
A1	27	2:15	3:00
A2	26	2:15	3:00
A3	25	2:25	3:00

B. *Compressive strength*

Compressive strength increases in the A1, A2 and A3 mixes when compared with A mix, and for all ages (7, 28 of 90 says) as shown in Table (4).The maximum increase in strength was 18% for A2, this increase is due to:

a -Slag is containing 25% of quarts mineral as grains this means it acts as filler , while Iron oxide containing chrome and Aluminum (sizes less than 150 μm) works as water reducers additive.

b- Calcium silicate hydrates (tobermorite), forms the bulk of hydrations products of cement, and the main strength giving component. To a limited extent Al_2O_3 , Fe_2O_3 can be accepted in the structure of tobermorite , formatting calcium – aluminum silicate hydrates with FeOH gelatinous material adsorbed at the surface of the hydrated crystals, this gives more strength to the structure by blocking the microspores in it . Maximum strength is obtained when the alumina content of the slag is up to 25 percent [7,8].

c- High sulfate content of aggregates (coarse and fine) in the mix A (reference sample) caused the formation of ettringite ($C_3A.3CaSO_4.31H_2O$), this compound causes an expansion in the structure which leads to lose of strength and highly pore spaces production. In mixes A1, A2 and A3 the slag compounds are activated by the presence of sulfate and calcium aluminum ferrite silicate is formed which gives the structure more strength acting as a binding material.

C. Flexural strength

Results presented in Table.4 demonstrating that the 90 days flexural strength of concrete mixes A1, A2 slightly lower than reference concrete mix ,while A3 concrete mix showed a considerable increasing (about 11.4%) compared to reference concrete mix. This behavior is generally due to the better cement-aggregate bond, this being at least in part due to the greater texture or roughness of slag-fine aggregate.

TABLE 4
COMPRESSIVE AND FLEXURAL STRENGTH RESULTS

Concrete mixes	Compressive strength(MPa) for ages 7 days	28days	90 days	Flexural Strength (MPa) for 90 days
A	26.8	32.7	39	5.9
A1	32.5	37	44	5.4
A2	31	37.8	46.2	5.6
A3	28.5	32.5	42.5	6.57

IV EXPANSION AND DRYING SHRINKAGE

Ettringite formation in the mix A caused an expansion in the prism samples up to 0.0006% in the ages of 7, 14, and 28 days, and up to 0.01% at the age of 90 days

A1, A2 and A3 samples expanded 0.001% , 0.004%, and 0.003% accordingly in the age of 7 days only. No changes took place at ages of 14, 28 and 90 days.

This could be due to the stability of the hydrated phases formed at early ages. Slag compounds consumed almost all calcium ions librated for sulfate hydrolysis preventing the formation of any expanding compounds in the mix. Drying shrinkage test results had fluctuated at early ages up to 28 days, then stopped at the age 90 days to be 0.02% except mix A3 with drying shrinkage reached 0.025% at 90 days age. Results are shown in Figure.1

V **CONCLUSIONS**

From the results of this investigation it can be drawn the following conclusions:-

1-Partial replacement of sand by slag as a percentage of cement weight 15%, 30% and 45% reduces the mixing water to achieve the same consistency ,also it increases the setting time (initial and final) so the slag acts as a retarder but still within the limit of I.Q.S.

2- The use of slag as a partial replacement of sand improve compressive strength up to 18% comparing to reference concrete

3- Concrete mix, A3 showed a considerable increasing in flexural strength (about 11.4%) compared to reference concrete mix.

4-Concrete mixes (containing slag) expansion and drying shrinkage are enhanced especially at late ages.

5-Slag addition to concrete with aggregates containing high sulfate percentages enhances physical and mechanical properties.

REFERENCES

1- Morian,D.A, Van Dam,T, Perera ,R.," Use of Air-Cooled Blast Furnace Slag as Coarse Aggregate in Concrete Pavements ,"US. Department of Transportation, Federal Highway Administration, Report No. FHWA-HIF-12-008, March, 2012, pp.148.

2- Rajan,M.S," Study on Strength Properties of Concrete by Partially Replacement of Sand by Steel slag," International Journal on Engineering Technology and Sciences, Vol.1 ,Issue 6, October 2014,pp.4.

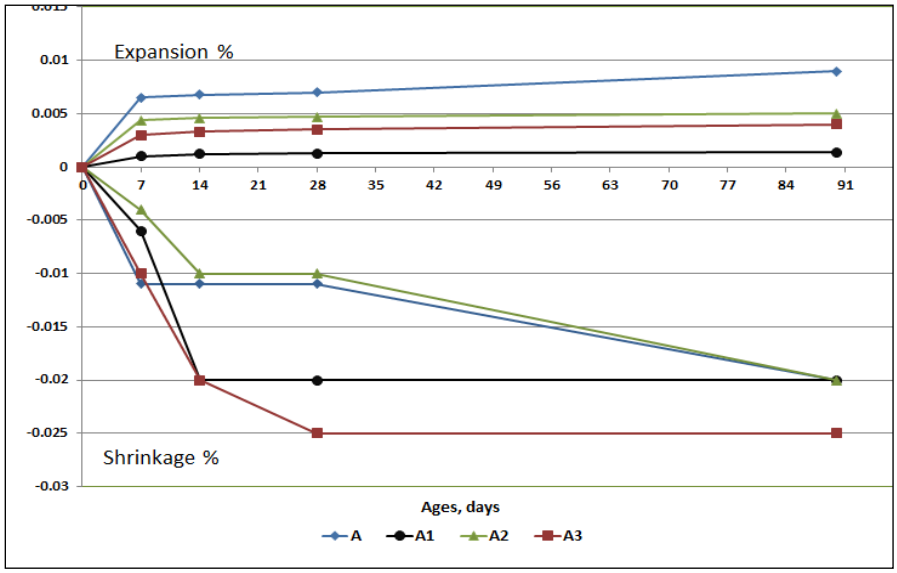


Fig.1 Expansion and shrinkage of concrete mixes

3- Nadeem,M,. Pofale,A.D,"Replacement of Natural Fine Aggregate With Granular Slag - A Waste Industrial By-Product In Cement Mortar Applications As An Alternative Construction Materials," International Journal of Engineering Research and Applications (IJERA),Vol.2, Issue 5, September- October 2012, pp.1258-1264 1258.

4- Kothai,P.S,Malathy,R," Utilization of Steel Slag in Concrete as a Partial Replacement Material for Fine Aggregates," International Journal of Innovative Research in Science, Engineering and Technology, Vol. 3, Issue 4, April 2014,pp.8.

5- Kourounis ,S , Tsivilis , S , Tsakiridis , P.E ,"Properties and Hydration of Blended Cements with Steelmaking Slag," Science Direct, Cement and Concrete Research 37,2007,pp815-822.

6- ACI materials journal "Chemical Admixtures for Concrete ", ACI Committee 2123R-91.

7- Manmohan D. and Metha PK. " Enfluence of Pozzdanic Slag and Chemical Admixtures on Pore Size Distribution and Permeability of Hardened Cement Pastes", Cement and Concrete and Aggregate, 3(1), 1981. PP. 63-67.

8-Murat,M and Negro,A, " Application of Scanning Electron Microscopy to the Study of Slag's and Short- term Slag Hydration Productions " Library Translation 1996, , August 1975.

Performance of Reinforcement Beams Made With Self-Compacting Concrete in Aggressive Environment

Prof. Dr. Mohammed M. Salman, Asst. Prof. Dr. Qais J. Frayyeh and Lect. Luma A. Zghair

Abstract— **Self-Compacting Concrete mixes (SCC) is increasingly being used in many application some of which are susceptible to an Aggressive Environment such as sulfuric acid solution. In this study SCC incorporating silica fume, chalk powder and hybrid fibers were used in casting 16 beams in two groups. The beams in the first group were cured in normal environment for 28 days, while those in the second group were subjected to sulfuric acid solution of 0.5% for six months. The flexural behavior of the beams was evaluated by testing the specimens under two-point loading until failure. It included cracking, failure pattern, deflection, ductility, and flexural strength measurements. The results indicated that the SCC mixes showed comparable structural behavior with respect to the corresponding control mixes in a normal environment. Different SCC mixes in an aggressive environment yielded a different structural performance, depending on the composition of the fillers and the main reinforcement ratio.**

***Index Terms*— self-compacting concrete, silica fume, sulfuric acid, hybrid fibers, flexure**

I. Introduction

Degradation of concrete members exposed to aggressive sulfuric acid environments is a key durability issue that affects the life cycle performance and maintenance cost of vital civil infrastructure .[1] Sulfuric acid in groundwater , chemical waste or generated from the oxidation of sulfur bearing compounds (e . g. pyrite) in backfill can attack substructure concrete members . Moreover, concrete structures in industrial zones are susceptible to deterioration due to acid rain of which sulfuric acid is a chief component [1] Concrete can be attacked by liquids with p^H value below 6.5 but the attacks are severe only at a p^H below 5.5 [2] Sulfuric acid fluids are classified as the most aggressive of natural threats to concrete structures. Generally, they arise from industrial operations, but they can be caused by urban areas activities. Large quantities of acids are existing in sewage systems. Acid attack is influenced by the processes of disintegration and leaching of cement paste constituent. A very important quantity of admixtures in SCC paste can negatively or positively affect its resistance to acid aggression. [3] .The deterioration of concrete in acidic environments is influenced by several factors such as the type of cement used, permeability of concrete, and the surrounding environmental conditions. Furthermore, the solubility of calcium salt produced from the acid – base reaction of the cementitious paste and attacking acid is an important factor. . [4] . Many studies about the possibility of improving the quality and performance of concrete exposed to sulfuric acid solution have been completed. **Bassuoni and Nehdi** [1] examines the resistance to aggressive sulfuric acid solutions (pH of 2.5 and 1.0) by using a variable range of SCC mixtures single (cement only) , binary (cement and silica fume) ,ternary (cement , silica fume and slag) and quaternary (cement , slag , fly ash and limestone powder) binders, with and without fiber reinforcement single and hybrid (macro steel + micro polypropylene) . They concluded that: the resistance of SCC to sulfuric acid attack was improved by using binary, ternary, and quaternary binders. They also reported that, the inclusion of hybrid micro – and macro – fibers can be effective in retaining the cementitous matrix integrity and controlling disruptive pressures resulting from voluminous reaction product. **Dimitri** [5] investigated the influence of the concrete composition on the sulfuric acid attack. They used limestone filler to improve the performance of concrete exposed to sulfuric acid solution. Their Concrete samples have been submerged in a solution, with a pH varying between 1.7 and 2.0. The compressive strength of the submerged samples has been compared with reference samples at an age of 6, 13 and 26 weeks. There Results indicate that SCC with limestone filler shows a significantly higher resistance against sulfuric acid attack, compared with SCC with fly ash and quartz filler and compared with traditional concrete. They also reported that the type of superplasticizer does not have a significant influence. In the case of performance of reinforced RC beam subjected to aggressive environment. **Safan** [6] investigated the performance of beams made of low-cost self-compacting concrete in an aggressive environment of the beams cross section (b = 100 mm) and d is the effective depth (d = 12.7 mm) . In their study**, SCC** incorporating silica fume (SF), fly ash (FA) and dolomite powder (DP) were used in casting two groups of beams. The beams in one group were stored in an open environment, while those in the other group were subjected to salt attack and

successive wet/drying cycles. The beams were stored for about one year under a sustained load. The structural performance of the stored beams was evaluated by testing the specimens under four-point loading until failure. They reported that, The post-cracking stiffness of all test beams subjected to corrosion was smaller than the post-cracking stiffness of the corresponding control beams they also reported that, no serious loss in the area of the main reinforcement occurred under the prescribed exposure conditions. On the other hand, the results suggested that the corrosion affected only the bond characteristics between the steel rebars and concrete. **AL – Zubady** [7] studied the bond strength of self compacting concrete reinforced concrete beams of cross section (140 * 150*600) after exposed to saline water . the specimens were test as a simply supported beams, three groups of beams have been tested to study the effect of bar diameter (8, 12, and 16) mm , compressive strength (30, 60) MPa and type of curing , (tap water continuous curing ,saline water wetting and drying , saline water continuous exposing for a time of 90 days) . He concluded that the bond stress and the corresponding slip for the same steel bar and different concrete compressive strength of concrete causes an increase in the bond strength between the concrete and the steel bar in the case of tap water continuous curing and saline water continues exposing , but in the case of saline water wetting and drying curing , the bond strength between the concrete and the steel bar decrease because the type of curing . Few researchers on flexural performance of reinforced RC beam subjected to aggressive environment were studied. This study aims to investigate the structure performance of reinforcement beams made with self-compacting when exposed to aggressive sulfuric acid solution.

II. RESEARCH SIEGNIFICANCE

Several concrete elements have been reported to be Susceptible to the chemical attack of sulfuric acid, including, industrial floors of chemical plants, superstructures (due to acid rain) , sewage pipe systems , etc . , even though there has been un increased use of SCC in many concrete applications , a comprehensive review of literature indicates that there is lack of information on the flexural resistance of SCC beams to sulfuric acid attack , also the role of hybrid fibers which can be inserted in SCC in such aggressive exposures is unclear

III. MATERIAL

The binders used included ordinary Portland cement (OPC), (ASTM C150 – Type 1) [8]. The cement was tested and checked according to IOS 5:1984[9] , silica fume (SF) of class N pozzolans (ASTM 618-03) [10], and limestone filler (LF). The chemical and physical properties for the various binders are listed in Table 1. The total cementitious materials (binder) content was kept constant at 500 kg/m^3 conforming to common SCC mixture design guidelines [e.g. EFNARC] [11]. The fine aggregate was natural sand with a fineness modulus of 2.6, a specific gravity of 2.65 Crushed coarse aggregate with a maximum nominal size of 10 mm, a fineness modulus of 2.6 a specific gravity of 2.62 was also used.

Table 1: The chemical and physical properties for the various binders

Oxides	cement	silica fume	Limestone powder
SiO_2	18.79	96.68	2.24
Fe_2O_3	3.9	0.069	0.12
Al_2O_3	4.5	0.20	0.42
CaO	66.57	0.54	68.73
MgO	3.57	0.12	0.70
SO_3	2.24	0.61	<0.07
specific gravity	3.2	2.13	2.42
specific surface area m^2/g	4.37	0.157	3.17

To improve flowability of the SCC mixtures, copolymer-based Superplasticizer, (SP) designed for the production of SCC (Glenium 51) with relative density of 1.1 at 20°C was incorporated in all mixtures. The dosages of SP was adjusted to maintain a slump flow of 600 - 750 mm, T50 (4 to 10) sec., L box index (>0.75) (3 Ǿ10 mm with 50 mm gaps) and V-funnel flow (3 to 25) sec. Three types of fibers were used , micro-reinforcement of polypropylene fibrillated fibers, with a specific gravity of 0.91 , length of 12 mm, Fiber thickness of 18 micron and tensile strength of 350 MPa were added at dosage of 0.2% by volume , macro reinforcement of crimped plastic fibers with a specific gravity of 1.14, length of 50 mm, aspect ratio of 63, and tensile strength of 250–350 MPa were used at dosages of 0.2 by volume and straight micro-reinforcement steel fiber with a density of 7800 kg/m^3 , length of 15 mm, diameter of o.2 mm , aspect ratio of 75, and tensile strength of 2600 MPa were added at dosage of 0.3% by volume . Based on trial mixtures, these shapes, lengths, and of micro- and macro-reinforcement have proven adequate for achieving the characteristic flowability and passing ability of SCC with minimal clustering of fibers. The constituents of the selected SCC mixes are given in Table 2.

Table 2: The constituents of the selected SCC mixes

Mix notation	Mix NO.	cement kg/m^3	SF kg/m^3	SP% wt. of cement	Fibers by volume %		
					ST	PL	PP
SCC LP SF	M1	315	35	1.5	-	-	-
SCC LP SFHR1	M2	315	35	3	0.3	-	0.2
SCC LP SFHR2	M3	315	35	2.1	0.3	0.2	-

Note: for all mixes: water = 170l/m^3, sand =778 kg/m^3, gravel = 890 kg/m^3 , and limestone powder =150 kg/m^3

For flexural test, the nominal dimension of the tested beams were (1100mm) in overall length and (200mm) in depth. Fig. (1) shows details of test beams .

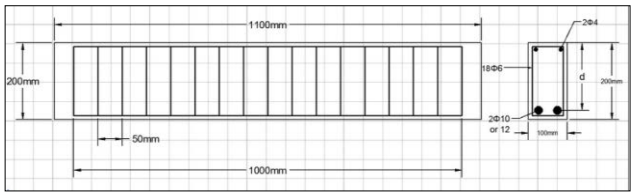


Fig. 1: Details of reinforcement for test beams

Two size of deformed steel bars employed as tension reinforcement for flexure. (Ǿ10 mm) and (Ǿ12 mm). Also , Ǿ 6 mm and (Ǿ 4 mm) smooth mild steel bars which were used as stirrups (spacing 50 mm) and tie bars to hold the upper reinforcement in flexural beams respectively. All flexural beams were designed in accordance with ACI 318-M95 Code [12]. All beams were tested under two points loading the dial gage was placed in its position touching the bottom surface of the beam at mid-span. Fig. (2) shows the actual beam setup

Fig. 2: Actual beam setup

IV. PREPARATION OF ACID SOLUTION

In the present study, chemical immersion tests were adopted to assess the resistance of SCC used in different applications such as foundations, walls, floors, pipes, etc .All concrete specimens were cured in water for 28 days, after which they were immersed in acid solution. The initial pH (2.3) of the solution increased quickly, but was controlled at a maximum threshold value of (3) by titration with concentrated sulfuric acid, and was kept constant thereafter during the period of testing. A digital portable pH meter was used for monitoring the pH levels of the sulfuric acid solutions. Specimens were fully immersed for 24-week, before immersion, the specimens were left to dry under 20 °C and 50% RH. Each group of mixtures had its own acid bath. This is to provide similar acidic environments for the different binder mixtures in each group of mixtures

V. GENERAL BEHAVIOR

Photographs of the tested beams are shown in Figure (3) and the test results are given in Table (3). All beams of this category were designed to fail in flexure with tensile mode The general behavior of the tested beams can be described as follows:

Table 3: Test result of SCC beams before chemical immersion

Mix NO.	Before chemical immersion					
	No. of beam	UL.	ΔU	Fcr	ΔY	Ψ
M1	B1	24.5	-	-	-	-
	Ǿ10B1	92.5	4.8	30	1.3	3.6
	Ǿ12B1	172	4.2	25	1.3	3.2
M2	B2	25	-	-	-	-
	Ǿ12B2	169	3.9	40	1.1	4
M3	B3	31.5	-	-	-	-
	Ǿ12B3	192	5.2	30	1.2	4.2

Not. UL: ultimate load , ΔU : ultimate deflection , Fcr : first crack load ,ΔY: yield deflection , Ψ.: ductility index = ΔU/ ΔY , Ǿ12 and Ǿ10 : steel bars diameter

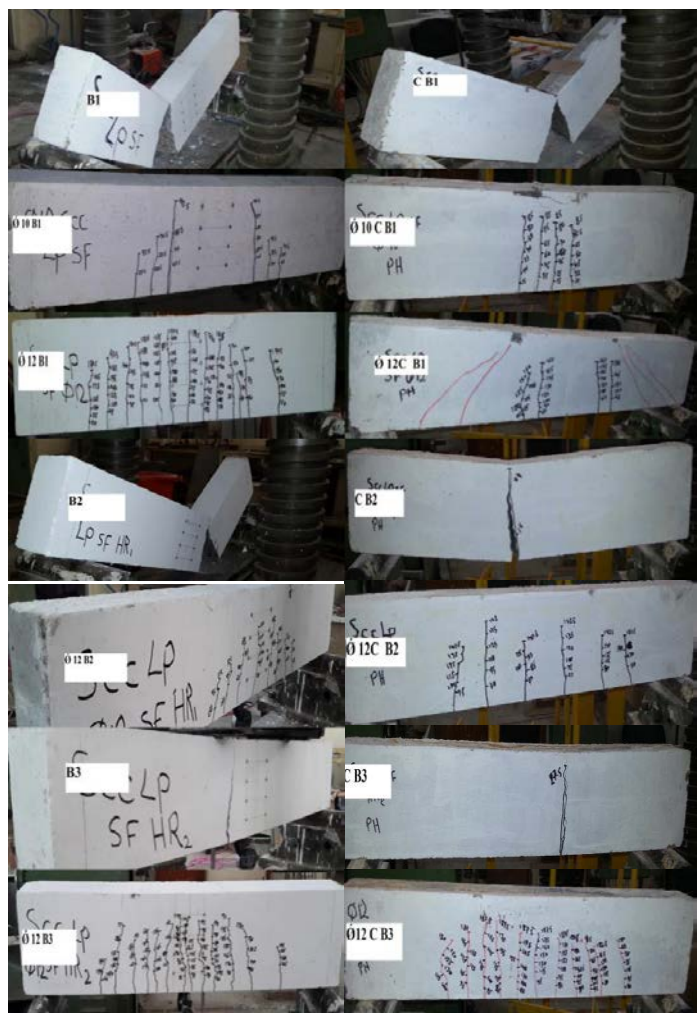


Fig. 3: Photographs of the tested beams

For non steel reinforcement beams : specimens with non fiber reinforcement (B1 , CB1) , the flexural cracks appeared directly in the mid span, and then a sudden tensile failure occurred shortly after formation of these crack, on the other hand specimens with hybrid fibers tend to behave differently (CB2 , B3 , CB3), in fiber reinforced concrete, after cracking of matrices or brittle fracturing, ductile fibers continue to carry and transfer the loads to other fibers; hence they help keeping structural integrity and cohesion of material , only in B2 , this is probably due to the fact that , small fibers may be pulled out after the main flexural crack are formed. For steel reinforcement beams, At early stages of loading, several cracks initiated in the tension zone at the constant moment region, with continue loading, these cracks extended upwards and became wider, more cracks developed at the bottom of the beam, One or more cracks spread faster than the others and reached the compression zone, all beams showed flexural failure

VI. FIRST CRACKING LOAD

The first cracking load**(Fcr)** are presented in Table (3) ,and Table (4)

Table4: Test result of SCC beams after chemical immersion

Mix NO.	**After chemical immersion**					
	No. of beam	UL.	ΔU	Fcr	ΔY	Ψ
	C B1	20	-	-	-	-
	B1					
	Ǿ12C B1	160	3.4	20	1.2	2.9
M2	C B2	24	-	-	-	-
	Ǿ12C B2	142.5	5.3	45	1.2	4.4
M3	C B3	27	-	-		-
	Ǿ12C B3	187.5	7.4	75	1.6	4.6

comparison between (Ǿ 12 B1) and (Ǿ 12 B2 and Ǿ 12 B3)

crack increased 60% and 20% respectively .Also the use of small hybrid fiber , in Ǿ 12 B2 Ǿ , the cracking load that produced first crack increased 33% compared with Ǿ 12 B3 Ǿ .This is attributable to that the presence of microfibers delays the internal micro cracking in the cementitious matrix and as micro cracks develop in the matrix, the microfibers in close to such cracks will try to arrest these cracks and prevent further propagation. Thus the cracks that appear inside the matrix have to converts the path, resulting in demand for more energy for future propagation, which in turn increases the first crack load. The initiation of cracks at higher deflections establishes that presence of fibers improved the tensile response of matrix. This finding is also reported by [13] [14]. The experimental results also show that the increase of the beams longitudinal steel ratio (ρw) caused the cracking load that produced first crack decreased. After exposure to sulfuric acid, the load that produced first crack for beams with hybrid fiber increased while for beams with non hybrid fibers, decreased, this perhaps due to the effectiveness of fiber hybridization in mitigating the first crack formation during bending tests.

VII. LOAD – DEFLECTION CURVES

Before chemical immersion , from Fig (4) , it is evident that , the deflection at ultimate load increase by the present of hybrid fiber The addition of hybrid fibers in beams Ǿ 12B3, Ǿ 12 B2 resulted in an increase in the value of ultimate defection by (19) and (23) %, respectively comparable with Ǿ 12 B1 .This is normally explained by the efficiency of fibers in arresting the propagation and controlling the growth of the flexure cracks within the beam when they are crossed by them, and hence, fibers maintain the beam integrity throughout the post-cracking stages of behavior. Similar behavior was confirmed in other research studies [13][15][16].

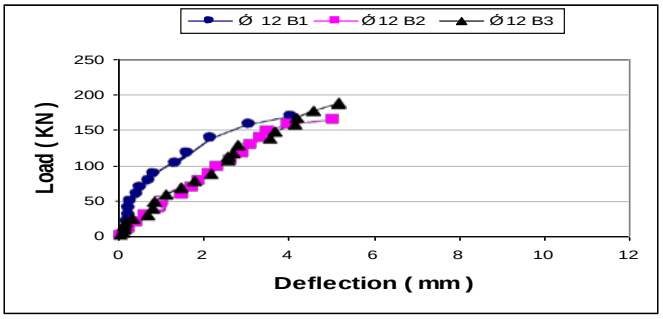


Fig . 4: Effect of hybrid fibers

Fig. (5) shows the effect of (ρw) on the load deflection response for SCC beams at different stages of loading. It can be seen from this figure, at a same load, increasing percentages of (ρw), caused decrease of the deflection of the SCC beam. After chemical immersion

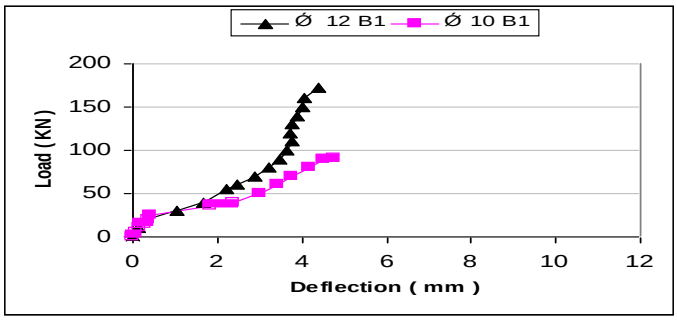


Fig.5 : Effect of longitudinal steel ratio

.From Fig. (6) and Fig . (7) at a same load, the deflection for beams without hybrid fiber were lesser to the corresponding deflection of beams under normal condition while the deflection for beams with hybrid, where higher to the corresponding deflection of beams under normal condition

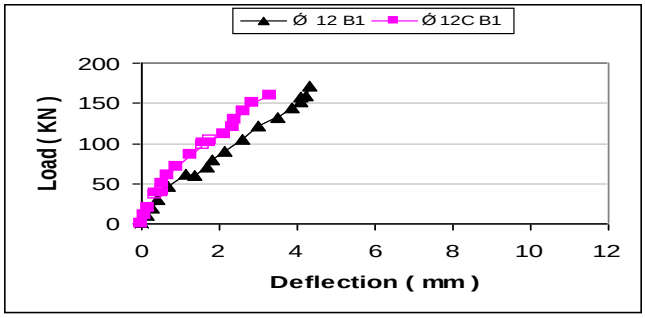


Fig . 6: Effect of acid solution on beam without hybrid fiber

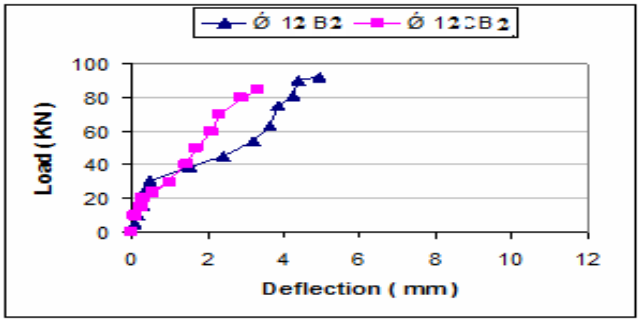


Fig. 7: Effect of acid solution on beam with hybrid fiber

VIII. ULTIMATE LOAD

A comparison between (Ǿ 12B3 and Ǿ 12B1) and another comparison between (B3 , and B1).the used of hybrid fibers with micro and long fibers increase the ultimate load ,the increase was (11)% and (29)% respectively . On the other hand for beam B3 and Ǿ 12B3 , with short hybrid fiber the effect of hybrid fibers on ultimate load was insignificant , The short fibres had a limited effects on the post-peak response of the load deflection curve , it can only affect the pre peak micro cracking on the other hand ,there was an essential effect of long fibres on the post-peak response part of the curve, resulting in a high value of fracture energy. This finding is also reported by [17][18]. The Effect of Longitudinal Reinforcement Ratio (ρw) was also studied, a comparison between Ǿ 10B1 and Ǿ 12B1, the experimental results showed that the increase of the beams longitudinal steel ratio from (0.007) to (0.011) caused increase of the flexural strength of the SCC-beams the increase was 85% . This is attributed to the fact that increasing (*ρw*) increases the dowel capacity of the member by increasing the dowel area and hence decreasing the tensile stresses induced in the surrounding concrete. After chemical immersion, as is given in Table (4) . For non reinforcement steel beams, A comparison between(B1 B2 B3) and between corresponding corroded beams (CB1 , CB2 and CB3)) the use hybrid fibers enhance the resistance to sulfuric solution , the reduction in flexural was about 15 % in CB3 and % 4 in CB2 , while the reduction in flexural was about % 20 in CB1 . This is mainly attributed to that, the inclusion of hybrid fibers can be effective in retention the cementitous matrix integrity and control the disruptive pressures resulting from voluminous reaction product [1] . From Table (4) for reinforcement steel beams the

and Ǿ 12 C B3) under sulfuric acid were close to the corresponding ultimate loads of beams under normal condition , The reduction of ultimate loads for beams was insignificant (8,8%) , (7.5%) and (2.4%) respectively . while the reduction of ultimate loads for beams Ǿ 12 B3 under sulfuric acid was somewhat higher (18.5%) . These results indicated that no serious loss in the area of the main reinforcement occurred under the prescribed exposure conditions.

IX. DUCTILITY

Before chemical immersion from Table (3) it can be concluded that the ductility ratio increased by the presence of hybrid fibers. The ductility ratio increased by (25%) and (31%) for Ǿ 12B2 and Ǿ 12B3, respectively, compared with the non fibrous SCC-beam Ǿ 12B1. this increased ductility perhaps due to the bridging of localized macro cracks [13]. Table (5-1) also shows that the ductility ratio decreases with increasing ρw. The ductility ratio decreases by (9%) in Ǿ 12B2, compared to Ǿ 10B2. This is expected since the increase in ρw causes a reduction in the deflection at ultimate load, thus lowering the ductility ratio. After chemical immersion, All corroded beams with non hybrid fibers showed a reduction in the ductility index compared to the corresponding control beams. This finding is also reported by Safan [6] . On the other hand for corroded beams with hybrid fibers , the ductility index increased compared to the corresponding control beams as shown in Table (4) this perhaps due to the effectiveness of fiber hybridization providing a great deal of energy absorption during bending tests

X. CONCLUSION

1. By using hybrid fibers the cracking load that produced first crack increased. After exposure to sulfuric acid , the load that produced first crack for beams with hybrid fiber increased while for beams with non hybrid fibers , decreased
2. The deflection at ultimate load increase by the present of hybrid fiber**.** After chemical immersion, at a same load, the deflection for beams without hybrid fiber were lesser to the corresponding deflection of beams under normal condition whiles the deflection for beams with hybrid were higher to the corresponding deflection of beams under normal condition.
3. The used of hybrid fibers with micro and long fibers increase the ultimate load ,the with short hybrid fiber the effect of hybrid fibers on ultimate load was insignificant . After chemical immersion , the use hybrid fibers enhance the resistance to sulfuric solution , the reduction in flexural
4. The ductility ratio increased by the presence of hybrid fibers .For corroded beams with hybrid fibers the ductility index increased compared to the corresponding control beams while with non hybrid

fibers a reduction in the ductility index compared to the corresponding control beams

XI. REFERENCE

1 . Bassuoni , M.T. and Nehdiet , M. L. , "Resistance of self-consolidating concrete to sulfuric acid attack with consecutive pH reduction "Cement and Concrete Research 37 (2007) pp 1070-1084

2. Sesha , P. S. , Sehadri , S. T. , Srinivasa R., Sravana , Sarika , P. , "Studies on effect of mineral admixtures on durability properties of high strength self compacting concrete " Volume: 02 Issue: 09 | Sep-2013,

3. Siad , H., Mesbah , H. A. , Khelafi , H. , Kamali , B. S., and Mouli , M. , " Effect of mineral admixture on resistance to sulphuric and hydrochloric attacks in self – compacting concrete " Can. Journal in civil engineering , 37 (2010) pp 441- 449

4. Bassuoni , M. T. , Nehdi , M . , and Amin , M., "Self-compacting concrete: using limestone to resist sulfuric acid ": Proceedings of the ICE - Construction Materials, Volume 160, Issue 3, 01 August (2007) PP 113 –123

5. Dimitri F. , Zanqun L. , , Gert H. , Geert D. S. , Veerle B. , Bram D. , John V. , Özlem C. , Lucie V. , and Dionys V. , G. " Influence of self-compacting concrete composition on sulfuric acid attack " RILM , June , 2009 65 pp.435-443

6. Safan M. A. , " Performance of Beams Made of Low-cost Self-compacting Concrete in an Aggressive Environment "Acta Polytechnica Vol. 51 No. 5/2011

7. Al- zubady S.R. "Bond strength of self –compacting concrete reinforced concrete beams exposed to saline water " MSc. Thesis, Department of Civil Engineering, College of Engineering, AL – Mustansirya University , 2010, pp.1-110

8. ASTM C150, "Specification for Portland Cement" Annual Book of ASTM Standard, Vol. 04-02, 2002

9 . Iraqi Specification, No. 5/1984., "Portland cement".

10 . ASTM C618-03, “Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete”, Annual Book of ASTM Standard, Vol. 04-02, February 2003, p. 3.

11. EFNARC , “Specification and Guidelines for Self-Compacting Concrete” , , February 2002, pp.32

12 . ACI Committee 318 *"Building Code Requirements for Structural Concrete(*ACI 318-M95)", American Concrete Institute, Detroit, USA, 1995

13. Jeenu, G. , Reji U. R. and Syam P. V. ., " Flexural Behaviour Of Hybrid Fibre Reinforced Self Compacting Concrete "28 - 29 August 2007

14. Ivan M. " High-Performance Hybrid-Fibre Concrete, Development and Utilisation " Ph. D. Thesis, Technical university of Delft , 16 Jun. 2006 pp 1-211

15. Dimas de A. S. R. , Flávio A. , S. and Romildo D. T. F." Flexural behavior of hybrid steel fiber reinforced self-consolidating concretes.. REM: R. Esc. Minas, 2014 , 67(1), 2014 pp 27-32,

16. Zhiguo Y., Xiangyu C. and Shuai D., " Ductility and strength of hybrid fiber reinforced self-consolidating concrete beam with low reinforcement ratios " Systems Engineering Procedia 1 (2011) 28–34

17. Wu , Y., Jie , L., and Keru , W., " Mechanical properties of hybrid fiber – reinforced concrete at low fiber volume fraction " Cement and Concrete Research 33 (2003) 27-30

18. Burcu A. , Mehmet , A. T., " Mechanical behavior and fibre dispersion of hybrid steel fibre reinforced self-compacting concrete " Construction and Building Materials 28 (2012) 287–293

Mechanical Properties of Date Palm Leaf – Stem Fibers Reinforced Concrete

Daad S. Dawood Assit. Proff. Dr. Abeer I. Alwared Dr. Suhair K. Al-Hubboubi

***Abstract*-Nowadays, most countries are looking forward at reducing pollution; one of the best solutions is using waste products as recycled materials. This not only can develop sustainable environmental management, but also presenting new materials for general use.**

A study has been conducted to look into the performance of date palm Leaf stem fiber for enhancing concrete characteristics. Leaf stem fiber was used in three percentages (0.4, 0.8 and 1.2) % volume fraction with curing ages of 7, 28 , 90 and 120 days. A comparison was carried out for both untreated and 1% NaOH-treated date palm leaf stem fiber. Results demonstrated that TLS 0.4, at age 120 days improved compressive strength with 4.76% maximum increase, while using ULS0.4 at 120 days cause decrease in compressive strength. However, it improved flexural and splitting tensile strength by 6.86 % and 10.23% for ULS0.4 and TLS0.4, by 7.62 % and 3.59% for ULS0.4 and TLS0.4, by 42.1% and 57.89% for ULS0.8 and TLS0.8, respectively at age 120 days.

***Index Terms* - Leaf-stem Fiber; Concrete; Mechanical Properties.**

1- Introduction

Concrete is the most widely used construction material in the world and steel reinforcement is always desired to meet tensile strength and ductility requirements of concrete structures. The production of concrete and reinforced concrete structures creates many environmental issues associated with the significant release of CO_2 and other greenhouse gases. Besides, the corrosion of steel reinforcement is one of the biggest challenges that current civil engineers are facing. Thus, it is urgent to promote sustainable concrete and structures to reduce their negative impact on the environment. [13]

The use of fibers in concrete became more and more current practice in rehabilitation of structures and the applications and more developed. That is due to the capacity of this new composite material (concretes reinforced by fiber) to limit and to control the cracks, improve the flexural and tensile strengths as well as to enhance the impact resistance. [1]

The development of steel reinforcement has overcome the problem of poor tensile strength. However, it does not completely solve the problem of micro cracks due to drying and plastic shrinkage owing to weathering conditions.

Addition of fibers can increase strength and reduce plastic shrinkage and drying shrinkage by arresting the propagation of cracks. [11]

For health and economic reasons the researchers are actually oriented toward the reinforcement of concretes by vegetal fibers, notably for countries that have these fibers in abundant amounts. [1]

Reference [3] studied the effect of different treatment process on the date palm fiber (DPF). Raw DPF underwent different surface modification methods such as alkali treatment with concentrations 0.5%, 1%, 1.5%, 2.5% and 5%, and acid treatment with 0.3, 0.9 and 1.6 N. All treatments were performed at 100 ºC for 1 hr. The surface morphology, thermal gravimetry analysis (TGA), Fourier transform infrared spectroscopy (FTIR), mechanical properties and chemical analysis, of treated DPF were investigated. Specimen treated with 1% NaOH showed optimum mechanical properties. Hydrochloric acid treatment resulted in deterioration in mechanical properties.

Reference [2] investigated the usage of palm oil fiber as discrete reinforcing fiber in concrete .Fiber reinforced concrete is able to increase its performance against tensile strength and toughness due to ability to absorb energy by reinforcing fibers. The results show that the mix design of palm oil fiber concrete added with and 0.50 and 10% pulverized fly ash (PFA) replacement gave the best compressive strength. It is shows that series with using palm oil, fibers reduces workability and compressive strength, but increases the tensile splitting strength**.**

The present study aimed to investigate the effect of date palm leaf stem fiber on mechanical properties including compressive strength, flexural strength and splitting tensile strength of concrete.

2-Materials and MixDesign

2-1 Materials

***Cement*:** Portland cement type I conforming to I.Q.S No. 5/1984 was used in all of types of concrete content mixes. The chemical and physical properties of the cement are given in tables (1and 2).

Fine aggregate: Natural sand of zone (2) was used as fine aggregate in this study conforming to I.Q.S No.45/1980.The gradation of the used fine aggregate. Its properties are given in table (3).

Coarse aggregate: Crushed river gravel of 19 mm maximum size was used as coarse aggregate. The properties of the coarse aggregate were determined and fulfilled according to Iraqi specification No.45/1984; its properties are given in table (4).

***High Range Water Reducing Admixture [super plasticizer]*:** High range water, reducing admixture (HRWR) super plasticizer, known commercially as (DEGASET AX 5011) was used. It complies with ASTM C 494 type F. The properties of this admixture (as claimed by the producer: YAPICHEM Company) are listed in table (5).

Leaf Stem Fiber (LS): In order to study the possibility of using leaf stem fiber as reinforcement for concrete. The fiber was extracted from stems and then preserved in polyethylene bags. All fibers were washed with tap water for several times in order to remove part of soluble substances as well as residues and dust at fiber surface. Then manually dismantled into bundles of separated fibers. The bundled fibers were dried at room temperature before being cut to the desired length 30 mm using a sharp blade and then collected in polyethylene bags for later use, figure (1). It was added at three proportions (0.4%, 0.8% and 1.2%) as volume fraction of the total mix. The physical; mechanical and chemical properties of the leaf stem fiber were carried out in Advanced Research Materials Center in the Ministry of Science and Technology, table (6).

Table (1): Chemical properties of cement

Compound composition	**Abbreviation**	**Percent by weight**	**Limit of IQS No.5/1984**
Lime	CaO	62.96	-
Silica	SiO_2	20.63	-
Alumina	Al_2O_3	6.96	-
Iron oxide	Fe_2O_3	1.62	-
Magnesia	MgO	2.50	≤ 2.85%
Sulfate	SO_3	2.54	≤ 5%
Loss on ignition	L.O.I	2.58	≤ 4%
Insoluble residue	I.R	0.98	≤ 1.5
Lime saturation factor	L.S.F	0.91	0.66-1.02

Main compounds % by weight

Name of compounds	Abbreviation	Percent by weight
Tri calcium silicate	C_3S	43.13
Di calcium silicate	C_2S	26.69
Tr icalicium laminate	C_3A	15.71
Tetra calcium aluminoferrite	C_4AF	4.92
Free lime	-	1.14

Table (2): Physical properties of cement

Physical properties	Limits of cement	Limits of IQS No. 5/1984
Fineness m2/kg	388	≥ 230
Initial setting time (h:min)	2:00	≥ 00:45
Final setting time (h:min)	4:30	≤ 10:00
Compressive strength(Mpa)		
3 days	24	≥ 15
7 days	37	≥ 23

Table (3):Gradation of fine aggregate

Sieve size (mm)	Percentage passing %	Limit of IQS No.45/1980, zone (2)
10	100	100
4.75	90	90-100
2.36	74.5	75-90
1.18	63	55-90
0.6	44.4	35-55
0.3	14.8	8-30
0.15	3	0-10
Sulfite % (SO_3)	0.047	≤ 0.5%

Table(4):Gradation of course aggregate

Sieve size(mm)	Percentage passing %	Limit of Iraqi specification NO. (45/1984)
20	100	100
10	37	30-60
5	2.8	0-10
Sulfite content	0.07	≥0.1%

Table(5): Properties of HRWR

Chemical content	**Polycarboxylate based**
Color, appearance	Brown liquid
pH	4-8
Density	1.06±0.02 g/cm^3
Chloride content (%)	< 0.1(EN 480-10)
Alkaline content (%)	>10 (EN480-12)

Table (6): Properties of leaf-stem fiber

Properties	Leaf stem
Density(kg/m^3) ¤	720
Water absorption after 24hrs in water	100%
Ultimate tensile strength MPa×	(43-69)
Aspect ratio (L/d)	(27-13.6)
Dimensions mm×	Thick(0.34-0.39) Width (1.1-2.2)
Moisture wt%*	17.7
Volatile matter wt%*	55.3
Fixed carbon wt%*	7.8
Ash wt%*	19.2

¤ This test carried in IBN SINA center
×This test carried out in advanced research materials center in ministry of science and technology.
* Reference [8]

figure(1): Date Palm leaf stem fiber

2-2 Alkalized Treatment of Leaf Fiber

In order to improve the fiber surface to modify the strength of the fiber–matrix interface, Alkali treatment used, which can remove hemicelluloses , lignin and all waste substances and produce a rough surface topography , the procedure of alkalization included:

- The clean fiber with 30mm length was soaked in 1% concentration NaOH solution at 100C° for 1 h then washed thoroughly with water to remove the excess of NaOH sticking to the leaf and leaf stem fibers. [3]
- Fiber was immersed in tap water in saturation before concrete mixing for 24hrs at room temperature.
- Then the fiber was air dried for 15 min. to obtain fiber saturated surface dry condition and then added to the concrete mixture.
- The leaf was used treated and untreated in the concrete mix.

2-3Concrete mix design

Mixture proportioning for concrete mixtures consisted of 750kg/m3 sand, 1035 kg/m3 gravel, 400 kg/m3 cement and a W/C ratio ranged between 0.37-0.411 and 1.2% by wt. HRWR as shown in table (7) and it was designed to achieve compressive strengths of 50 MPa at 28 days. For Leaf stem fiber- containing concrete, the fibers were immersed in tap water for 24hrs, then were air dried at room temperature to obtain saturated surface dry condition and then added to the dry constituents mix at volume fraction (0.4 , 0.8 and 1.2 %) for a period ranging from 1 to 3 min depending on the amount of fiber while the rotary mixer was running for 1.5 min. followed by adding 30% of mix water ranging between 0.384-0.416 with the 1.2% admixture and finally adding the remaining water gradually, over mixing is avoided such procedure results in good dispersion of fiber and prevent balling problems.

The slump was kept within the range 80±10 mm.

Table (7): Concrete mix

Concrete mix	Cement kg/m^3	Sand kg/m^3	Gravel kg/m^3	W/C ratio	HRWR admixture %
R	400	750	1035	0.406	1.2
ULS0.4	400	750	1035	0.384	=
ULS0.8	400	750	1035	0.384	=
ULS1.2	400	750	1035	0.411	=
TLS0.4	400	750	1035	0.373	=
TLS0.8	400	750	1035	0.399	=
TLS1.2	400	750	1035	0.37	=

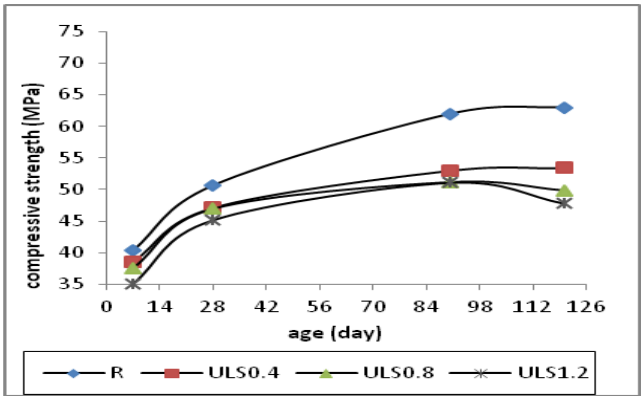
3-Resulrs and Discussion

3-1Compressive Strength

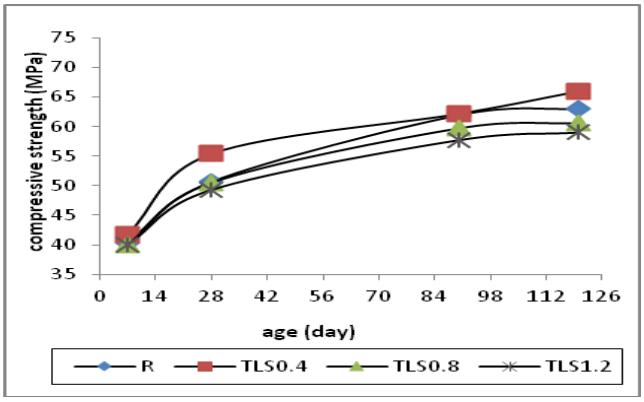
The effect of different content of untreated leaf stem fiber (ULS) on the compressive strength of concrete at different periods is shown in Figure (2a) and their values are tabulated in table (8).It is clear that the compressive strength is lower than the compressive strength of plain concrete and it decreases as a fiber volume fraction (Vf) increased. Such reduction in strength is could be due to the leaf stem fiber have large surface area (low aspect ratio L/d) making the concrete more porous, they make it difficult for the fiber/matrix adhesion, As fiber Vf keeps on increasing, compressive strength tends to decrease. Similar observations were reported by ref. [12]

For treated leaf stem fiber concrete, the compressive strength of TLS -concrete mix with 0.4 % fiber Vf is higher than the compressive strength of plain concrete (R) then it decreased as the fiber Vf increased ,figure (2b) and their values are tabulated in table (8). This can be attributed to:

- Alkali treatment can remove hemicelluloses, lignin and all waste substances and produce a rough surface topography and alkali treatment leads to fiber fibrillation, i.e. breaking down the fiber bundle into smaller fibers. This increases the effective surface area available for wetting with the matrix. Hence, increasing the fiber aspect ratio caused by reducing the fiber diameter and producing a rough surface topography offers better fiber/ matrix interfacial bond and increases mechanical properties.[3]-[9]-[4]
- The decreasing in compressive strength can be attributed to the voids introduction in the mix due to excessive fiber content that may lead to reduced bonding and disintegration [7] also to the lack of dispersion of fibers in the matrix, The increased porosity of the composite material due to the higher addition of fiber and alkaline medium of the concrete wearing out the fiber with age. Reference [6] made the same observations by using Sisal fiber in concrete matrix in different percentages to evaluate the durability of natural fiber (sisal fiber) reinforced mortar composites and also the relative performance of composites.



a. Untreated leaf-stem fiber



b. Treated leaf-stem fiber

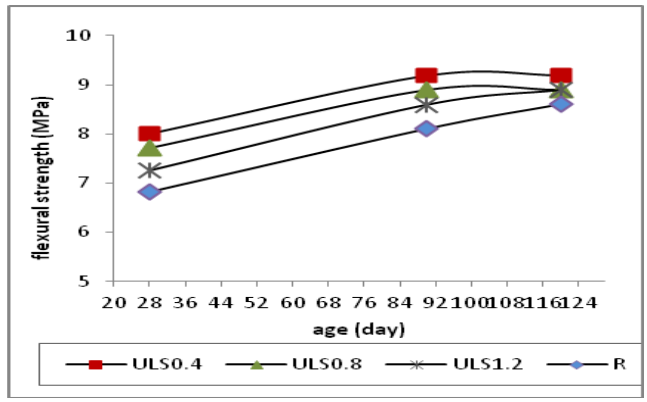
Figure(2): Effect of various content of leaf-stem fiber on compressive strength as compared with plain concrete

Table (8): Compressive strength results of treated and untreated leaf stem fiber - concrete

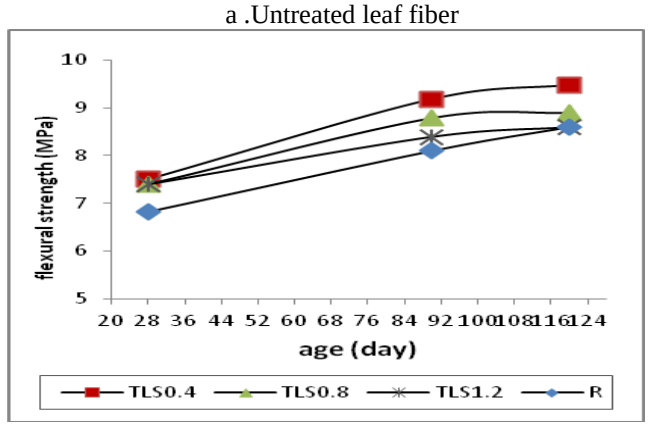
Mix concrete	Compressive strength (Mpa)			
	Curing ages (days)			
	7	28	90	120
R	40.3	50.6	61.9	63.0
ULS0.4	38.5	47.1	53	53.4
ULS0.8	37.6	47.0	51.2	49.9
ULS1.2	35.1	45.2	51.1	47.8
TLS0.4	41.7	55.4	62.2	66.0
TLS0.8	40.0	50.5	59.7	60.6
TLS1.2	40.0	49.3	57.7	59.0

3-2Flexural Strength

It indicates from the test results presented in figure(3) and the tabulated values in table (9), that flexural strength of plain concrete was improved by the addition of untreated leaf stem fiber and increased up to 0.4 % and decreased as the fiber Vf increased, this can be attributed to the distribution of the fibers in each specimen which slow the transmitting force especially if the fibers distribution were horizontally. This is in agreement with the conclusion reported by ref. [5] .The flexural strength of plain concrete enhanced by the inclusion of treated leaf stem fiber, but as the fiber Vf increased, there is a slight decrease in flexural strength but still higher than plain concrete. The flexural strength of TLS-concrete was higher than that of ULS-concrete at 0.4 % Vf for 120 days curing age and the flexural strength for both of them were higher than that of the plain concrete. The alkalization treatment improves the tensile strength of the leaf stem fiber [3] –[4]. This observation agreed with the finding by reference [1] who concluded that Delignified cellulose fibers (treated fiber) can be produced with a tensile strength of up to approximately 2000 MPa and fiber tensile strength of untreated approximately 500 MPa.



a .Untreated leaf fiber



b. Treated leaf fiber

Fig. 3: Effect of various content of leaf-stem fiber on flexural strength as compared with plain concrete

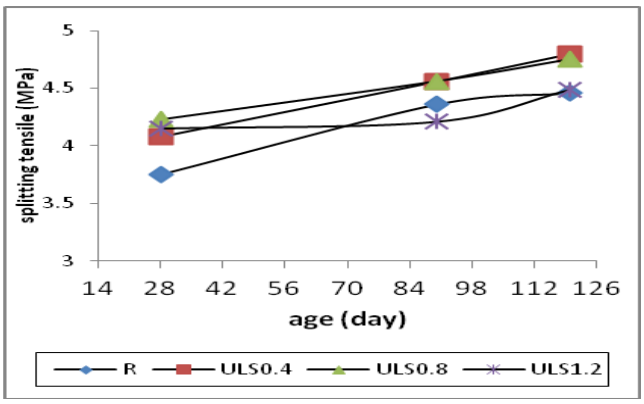
Table (9): Flexural strength results of treated and untreated leaf stem fiber - concrete

Mix concrete	Flexural strength (Mpa)		
	Curing periods (days)		
	28	90	120
R	6.82	8.1	8.6
ULS0.4	8	9.185	9.19

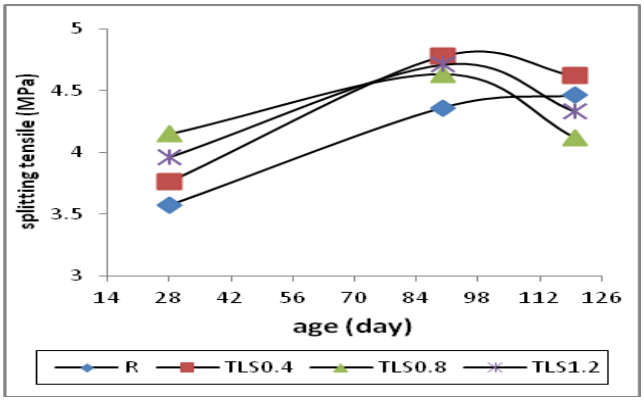
ULS0.8	7.71	8.89	8.98
ULS1.2	7.26	8.59	8.89
TLS0.4	7.5	9.185	9.48
TLS0.8	7.41	8.79	8.9
TLS1.2	7.41	8.393	8.595

3-3Splitting Tensile Strength

It is noticed from the test results presented in figure(4) and the tabulated values in table (10), that the splitting tensile strength of plain concrete enhanced by the addition of ULS fiber, and increased up to 0.4%, and then it decreased as fiber content increased at the ages 28, 90 and 120 days. This is due to the segregation problems as a result of incompetent compaction of the higher fiber Vf concrete mixes. For TLS-concrete, the splitting tensile strength increased considerably at the age 90 day and then drop when reached age 120 days.



a. Untreated leaf-stem fiber



b.Treated leaf- stem fiber

Fiure(4): Effect of various content of leaf-stem fiber on splitting tensile as compared with plain concrete

Table(10): Splitting tensile strength results of treated and untreated leaf stem fiber - concrete

Concrete mix	Splitting tensile strength (Mpa)		
	Curing periods (day)		
	28d	28d	28d
R	3.57	4.36	4.46
ULS0.4	4.08	4.56	4.8
ULS0.8	4.23	4.56	4.75
ULS1.2	4.15	4.21	4.4
TLS0.4	3.76	4.78	4.62
TLS0.8	4.15	4.63	4.12
TLS1.2	3.96	4.71	4.33

4. Conclusions

1-Using treated leaf stem fiber with concrete improved compressive strength with 4.76% maximum increase, while using untreated leaf stem fiber with concrete cause decrease in compressive strength. The best fiber content added to concrete was 0.4% of treated and untreated leaf stem fiber. The best fiber content added to concrete was 0.4% of treated and untreated leaf stem fiber. The addition of leaf stem fibers in concrete improved the flexural tensile strength and the best fiber content was 0.4% of treated and untreated leaf stems fiber; the treated type gave better results at age 120 days than the untreated type. The flexural strength of leaf stem fiber-concrete increased by 6.86% and 10.23% for ULS0.4 and TLS0.4 respectively at age 120 days .The splitting tensile strength improved slightly by the addition of treated and untreated leaf stem fiber by 6.5% and 3.59% for ULS0.8 and TLS0.4 at the age of 120 days, respectively.

2- Integrating the date palm leaf stem fiber in concrete mixtures represent a friendly environmentally alternative for date palm waste management.

REFERENCES

1. Abani,S. Krikerand, A. and Bali,A. ,"Effect of Curing and Mix Design Types on Performance of Date Palm Fibres Reinforcement Concrete under Hot Dry Environment", 11th Int. Inorganic Bonded-fiber Composites Conference (IIBCC),pp. 254-259, 2008.
2. Ahmad, M. H. and Noor, N. M.,"Mix Design of Palm Oil Fiber Concrete", M.Sc. Thesis, Department of Structural and Materials Engineering, Faculty of Civil and Environmental Engineering, University of Tun Hussein Onn Malaysia (UTHM), 2012.
3. Alawar, Hamed, A. A. M. and Al-Kaabi, K., "Characterization of Treated Date Palm Tree Fiber

as Composite Reinforcement", Composites: Part B, vol. 40, pp. 601–606, 2009.

4. Al-Maadeed, M.A., Kahraman, Khanam,R. P. N. and Al-Maadeed, S., "Characterization of Untreated and Treated Male and Female Date Palm Leaves", *Materials and Design*, vol.43,pp.526–531, 2013.
5. Al-Noaimy, S. F. " Duribilty of date palm fiber reinforced concrete", M.Sc. Thesis, University of Technology, Baghdad/ Iraq, 2001.
6. Ramakrishna, G. , Sundararajan, T.and Kothandaraman, S. ,"Evaluation of durability of natural fiber reinforced cement mortar composite- A New Approach", *J. Eng. and App. Sci., Asian Research Publishing Network* (ARPN), vol.5, no.6, pp.44-51 , 2010.
7. Ramli, M. and Dawood, E. T. ,"Effect of Palm Fiber on Mechanical Properties of Lightweight Concrete, Crushed Brick", *American J. Eng. and Appl. Sci.*, vol.3, no.2, pp.489-493, 2010.
8. Sait ,H.H. ,Hussain, A., Arshad, A. S. and Farid , N. A., " Pyrolysis and Combustion Kinetics of Date Palm Biomass Using Thermogravimetric Analysis", Bio-resource Technology, vol.118, pp. 382-389, 2012.
9. Sbiai,A. ,Maazouz, Fleury ,A., E., Sautereau, H.and Kaddami, H., "Oxidized Palm Fiber Composites", *Bio-resources*, vol. 5, no. 2, pp. 672-689, 2010.
10. Sivaraja, M. and Kandasamy, S., "Potential reuse of waste rice husk as fiber composites in concrete", *Asian J. of Civil Eng. (Building and Housing)*, vol.12, no.2, pp. 205-217, 2011.
11. Vajje, S. and Murthy, N. R. K. " Study on Addition of the Natural Fibers into Concrete", *Int. J. Sci. & Tech. Res.*, vol. 2, issue 11, pp. 213-218, 2013.
12. Wang, Z. Li , X. and Wang, Li. "Properties of Hemp Fiber Reinforced Concrete Composites", *Composites: Part A; Appl. Sci. and Man.*, vol.37,pp. 497-505, 2006.
13. Yan, L. and Chouw, N. "Infrastructure Corrosion and Durability – A Sustainability Study", Chapter: Sustainable Concrete and Structures with Natural Fiber Reinforcements, 1st ed, 2014.

Effect of Incorporation Techniques of Nanomaterials on Strength of Cement-based Materials

R. A. Al-Safy
Materials Engineering Department, Al-Mustansiriyah University
Baghdad, Iraq
email:rawaa.alsafy@gmail.com

Abstract— **The attractive properties of nanomaterials have motivated the development of advanced cement-based materials with unique properties. Despite the superior properties of nanoscale materials, the incorporation method of such materials into cementitious matrices is a critical issue which strongly influences the final properties. In this paper, various inclusion methods (B, MW, MP and S) were employed to introduce two different types of nanomaterials into cement-based-matrix. Different concentrations of Halloysite Nanotubes (HNT) (up to 2.5wt%) and low load concentrations of Nano Al_2O_3 (NAl) (up to 0.1 wt%) were used to replace partially the cement in (cement:sand) mortar. The water-to-cement ratio (0.4) was kept constant for all samples. The effectiveness of each incorporating method of HNT and NAl was evaluated by means of compressive strength test. Samples of cement mortars (with and without nanomaterials) were tested in compression and for different periods of curing (7, 14 and 28 days). It was found that the adoption of the most practical mixing procedure B leads to highest gain of compressive strength when compared with other procedures. Highest enhancement (36%) in compressive strength was recorded for samples with 2.5 wt% of HNT at 14 days using procedure B when compared with control samples. In terms of samples with NAl, it was observed that the introduction of very low concentration of NAl (0.02 wt%) using procedure MW results in higher values of compressive strength at age 28 days when compared with other procedures as well as with control samples.**

Index Terms— **cement mortar,,halloysite nanotubes, incorporation technique, nanomaterials**

I. Introduction

Due to their unique properties particularly particle small size as well as large surface area, various types of nanomaterials have been widely introduced into various types of matrices such as ceramic matrices [1], polymeric matrices [2] and cementitious matrices [3]. The main reason from dispersing nanomaterials into any matrix is to improve the properties of the composite such as mechanical properties [4], thermo-mechanical properties [5,6], etc. In terms of cement-based matrices, different nanomaterials can be used as supplementary cementitious materials (SCMs) in cement mortars as well as in concrete. It was reported that the microstructure and performance of cement-based materials are influenced by the addition of nanomaterials. This is due to many factors such as the filer effect of nanomaterials to fill the voids between cement grains. Additionally, nanomaterials participate in the pozzolanic reaction or accelerate such reaction. This will leads to consumption of calcium hydroxide ($Ca(OH)_2$) and formation of an "additional" hydrated calcium silicates (C-S-H) gel [7].

However, the best enhancements in the properties are linked to uniform dispersion of nanomaterials into the matrix. A well-dispersed nanomaterials work as centres of crystallization of cement hydrates which results in accelerating the hydration of cement. Such dispersion is achieved through the proper selection of mixing technique. Some techniques lead to agglomeration of nanomaterials which causes reduction in the final properties of the composites. This agglomeration is due to dominant intermolecular van der Waals interactions between nanoscale materials [8].

Depending on the type and quantity of nanomaterials, various mixing techniques were employed to disperse such nanoscale materials into cement- based composites. In case of carbon –based nanotubes, many studies were carried out to explore the dispersion method. The most popular method was sonication [9]. A group of researchers employed this technique to sonicate Ordinary Portland Cement (OPC) and Carbon nanotubes (CNT) together in isopropanol and then dry the liquid. However, drawbacks were associated with this type of mixing as the surface grain of OPC was damaged which causes lower rate of hydration of cement [10]. Utilization of dispersing agents such as super plasticizers was suggested by other group of researchers [11] in order to increase the repulsive forces between adjacent colloidal particles when nanoparticles are added to water. In this case, materials tend to agglomerate when it contact with water. As a result the rheological behavior of cement paste is affected and the final hardened properties [12] and it was found that the addition of suitable chemical admixtures or by adding extra water is used to disperse the solid particles in aqueous solution.

The main objective of the present study is to explore other mixing methods using other types of nanomaterials; namely, Halloysite Nanotubes (HNT) and nano Al_2O_3 (NAl) and without any admixtures. Additionally, the practicality of mixing procedures needs to be evaluated in order to fill the gap in knowledge in this regard. Thus, in the present study, the possibility to improve compressive strength of cement mortar using two types of nanomaterials and various dispersion techniques without any chemical dispersing agents. Various concentrations of nanomaterials were introduce to either cement matrix or mixing water to produce cement mortar reinforced with nanomaterials using various incorporation techniques. The outcomes will provide guidelines the help to understand the suitability and practicality of inclusion routes of nanomaterials into cement-based materials.

II. EXPERIMENTSL PROGRAM

A. Materials

Ordinary Portland cement (OPC) is used in this study. Its chemical composition is shown in Table (I).The chemical analysis of cement was carried out by Iraqi National Center for Construction Laboratories and Researches (NCCLR). Al-Ekhaider natural sand is used in this work as fine aggregate of 4.75 mm maximum size. Its grading is shown in Table (II). Cement in (cement:sand) mortar, (1:2) by weight, was replaced partially with various concentrations (0,0.5, 1.5 and 2.5 wt%) of Halloysite Nanotubes (HNT) and (0.02% and 0.1 wt%) of Nano Al_2O_3 purchased from Sigma-Aldrich. Halloysite ($Al_2Si_2O_5(OH)_4.2H_2O$) is a two-layered aluminosilicate with hollow nanotubular structure in the submicron range and it is chemically similar to kaolin [13]. According to the supplier [14], the diameter of HNT is 30-70 nm and the length is 1-3 µm, nanotubes. In terms of NAl, it is nano powder with particle size less than 50 nm as provided by the supplier [15].

B. Fabrication of Cement Mortar Specimens

Generally, casting of cement mortars with and without nanomaterials was carried out in accordance to ASTM C109. Cement was mixed with sand using (1:2) mixing ratio with (water/binder) ratio of 0.4. This ratio was kept constant for all mixes. Cubic moulds with (50mm×50mm×50mm) in dimensions were oiled with a thin layer before filling with cement mortar. The latter was poured into two layers. Each layer was compacted manually 32 times. Samples then were kept in ambient conditions and covered with plastic sheet for 24 hrs. Samples were de-moulded and marked before placing in curing tanks for various curing periods (7, 14 and 28 days).

TABLE I
CEMENT CHEMICAL COMPOSITIONS

Oxide Composition	Abbreviation	Content (%)
Lime	CaO	64.25
Silica	SiO_2	19.35
Alumina	Al_2O_3	4.61
Iron oxide	Fe_2O_3	3.19
Sulfate	SO_3	2.85
Magnesia	MgO	2.03
Loss On Ignition	L.O.I	4.00
Insoluble Residue	I.R	1.50
Lime Saturation Factor	L.S.F	1.01

TABLE II
GRADING OF SAND

Sieve size (mm)	Passing (%)
4.75	98.3
2.36	87.06
1.18	79.67
0.6	53.85
0.3	20.26
0.15	5.59

C. Mixing Techniques

Various mixing procedures were used in this work to introduced two types of nanomaterials (HNT and NAl). These procedures are B, MW, MP and S. Details about each procedure are summarized I Table (III). The mixing procedure B, MP and S were used to introduce nanomaterials into cement matrix. However, the introduction of nanomaterials into the mixing water was carried out via mixing procedure MW. For each mixing procedure and for each percentage of nanomaterials (0.02% - 2.5 wt%), three samples were prepared and for each age of curing (7, 14 and 28 days) (Table IV).

TABLE III
DESCRIPTION OF INCLUSION TECHNIQUES OF NANOMATERIALS

Mixing Procedure	Description of Mixing Procedure
B	Cement and the desired amount of nanomaterials are placed inside a plastic bag. The latter is shacked well manually for less than 2 mins. The continent of the bag is added to sand and mixed manually. Water is added gradually and mixed manually to get homogenous mixture.
MW	The desired amount of nanomaterials is added to 90% of mixing water using mechanical mixing at 800 rpm for 30 minute. Then cement was added gradually and mixed manually. Sand is added later with the remaining mixing water (10% of mixing water).
MP	The desired amount of nanomaterials was added to cement and mixed using mechanical mixing at 800 rpm for 30 mins. Sand and water were added later and mixed manually to achieve homogenous mixture.
S	Nanomaterials are added to cement matrix and mixed for 30 mins using ultrasonic bath with ultrasonic power 150 Watt and frequency of 50 Hz. Sand and water were added later and mixed manually until homogenous mixture is obtained.

III. RESULTS AND DISCUSSION

In total one hundred and thirty eight cubic cement mortars were tested in compression. Ninety three of them were for samples with various concentrations of HNT. Other forty- five were for samples with different percentages of NAl. The influence of incorporation method of nanomaterials on the compressive strength of cement mortars is described in the following sections.

A. Effect of mixing procedure on compressive strength of HNT- cement mortars

As illustrates in Fig.1 (top), the adoption of mixing procedure B results in highest gain of compressive strength for cement mortars containing various percentages of HNT when compared with control samples (0% HNT) at 7 days curing. Highest enhancements (5.1%) was recorded for samples with lowest concentrations of HNT (0.5 wt%). This could be attributed to easily disperse HNT into cement matrix using procedure B by reducing the possibility of agglomeration. However, application of other mixing procedures (MW and MP) causes reduction in compressive strength at early age.

TABLE IV
DETAILS ABOUT EXPERIMENTAL PROGRAM

Mixing Procedure	Age of Curing (days)	Nanomaterials (%)		No. of Tested Samples
		HNT	NAl	
B	7	0.5, 1.5, 2.5	0.02, 0.1	15
	14	0.5, 1.5, 2.5	0.02, 0.1	15
	28	0.5, 1.5, 2.5	0.02, 0.1	15
MW	7	0.5, 1.5, 2.5	0.02, 0.1	15
	14	0.5, 1.5, 2.5	0.02, 0.1	15
	28	0.5, 1.5, 2.5	0.02, 0.1	15
MP	7	0.5, 1.5, 2.5		9
	14	0.5, 1.5, 2.5		9
	28	0.5, 1.5, 2.5		9
S	7			
	14	0.5, 1.5, 2.5	0.02, 0.1	15
	28	2.5	0.1	6

The lowest values were recorded for MW procedure. This might mean this procedure cannot easily disperse HNT in water which causes agglomeration. The agglomerated particles may impede the water from being involved in the hydration process which then causes reduction in the early strength [16].

In terms of samples with 14 days periods of curing, the inclusion of various concentrations of HNT using three mixing procedures leads to high compressive strength in comparison with control samples (Fig.1 middle). Highest values were obtained for specimens of mixing procedure B. The enhancements in compressive strength were (25.6%), (33.9%) and (36%) for samples with 0.5 %, 1.5 % and 2.5 wt% respectively. Sonication comes as the second best mixing method to produce samples with HNT as partial replacement of cement. Lowest compressive strength was demonstrated when mixing procedure MW was used to include nanomaterials into 90% mixing water and for all percentages of HNT accept 2.5 wt% which shows high compressive strength and the improvement was (6.6%) in comparison with control samples.

Regarding samples of standard compressive strength (28 days), highest enhancement in compressive strength (21.7%) was recorded for samples with 2.5 wt% of HNT using sonication method. No enough data are available to draw conclusions about this mixing method and for other percentages of HNT (lower than 2.5 wt%). Thus, further work is need in this regard. From trends in Fig.1 (bottom), it can be observed that highest compressive strength was recorded for mixing procedure B and for various concentrations of HNT. Highest increment (17.2%) was obtained for samples with 2.5 wt% of HNT at 28 days periods of curing. Lowest values were obtained when mixing procedure MW was adopted and for samples with 0.5 wt% and 1.5 wt%.

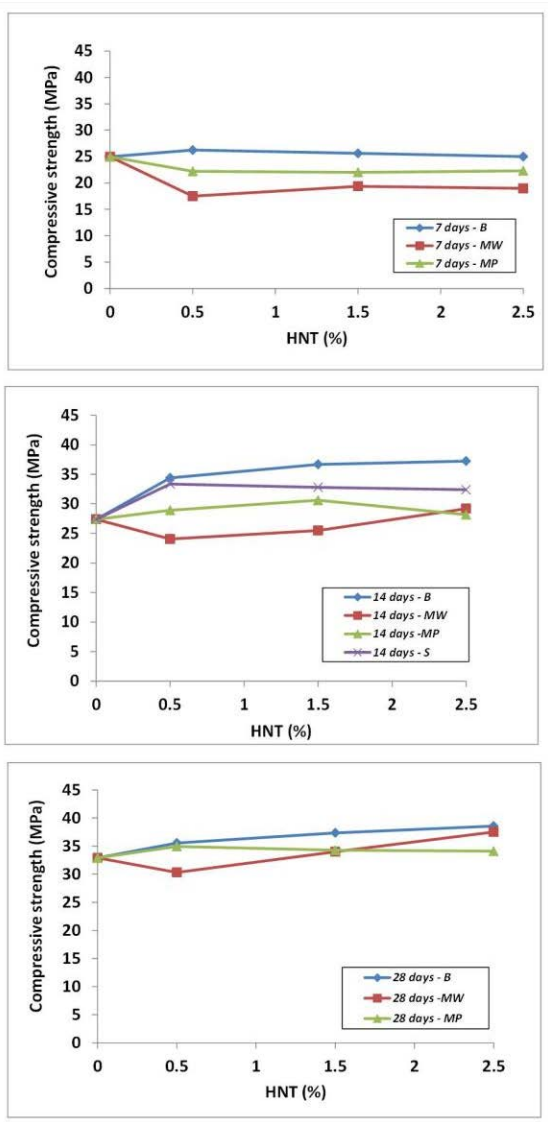


Fig. 1: Compressive strength *verses* percentages of HNT using various mixing procedures at: 7 days (*top*), 14 days (*middle*), and 28 days (*bottom*)

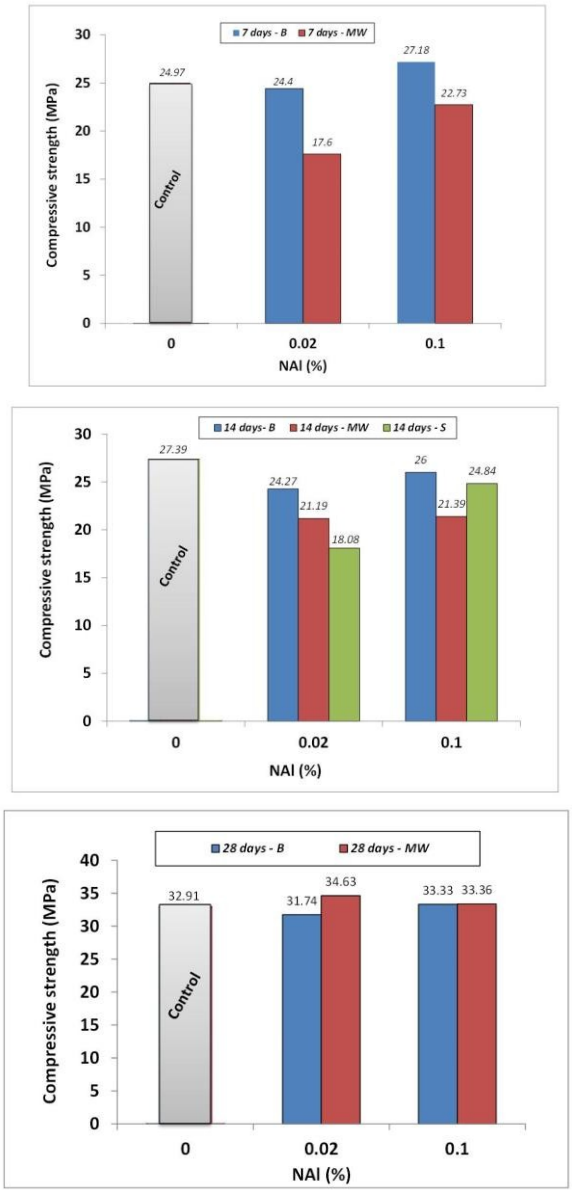


Fig. 2: Compressive strength *verses* percentages of NAl using various mixing procedures at: 7 days (*top*), 14 days (*middle)*, and 28 days (*bottom*)

B. Effect of mixing procedure on compressive strength of NAl-cement mortars:

As a comparison between mixing procedure B and MW, higher gain of compressive strength were recorded for samples containing (0.02wt% and 0.1 wt%) using procedure B at age 7 days and 14 days as illustrated in Fig. 2 (top) and Fig. 2 (middle) respectively. However, at 28 days, higher gain of compressive strength was recorded for procedure MW and the highest compressive strength was observed for samples with very low concentrations of NAl (0.02 wt%) (Fig. 2 bottom). The effectiveness of such mixing procedure was clarified at28 days periods of curing. This means no agglomeration is expected when very small proportion of NAl was mixed with 90% of mixing water using mechanical stirrer for 30 mins. A very slight variation in compressive strength at 28 days was observed when two procedures (B and MW) were used to add 0.1 wt% of NAl.

In terms of sonication method, the obtained compressive strength at 14 days were lower than those obtained from other procedures (B and MW) and for samples with very low percentages of NAl (0.02 wt%). However, the obtained values for samples with 0.1 wt% and for sonication method were lower than those obtained from procedure B but higher than those from procedure MW. The obtained results from this study using sonication are in contrary to reported work by others [16,17] in which highest enhancements in strength was obtained using sonication method. In the present study, it may

attributes to very low concentrations of NAl and the well-dispersion of such percentages might be not easy using sonication method for 30 mins.

As a comparison with control specimens (0 % NAl), enhancements in compressive strength was recorded for samples with 0.1 wt% using mixing procedure B and for curing periods of (7 and 28 days) as well as for samples with (0.02 wt% and 0.1 wt%) using procedure MW at 28 days. Highest improvement (5.2%) was obtained for samples with 0.02 wt% of NAl using mixing procedure MW.

V. Conclusion

The influence of utilize various inclusion procedures of two types of nanomaterials on compressive strength of cement mortars was evaluated. The following could be drawn:

1. The adoption of the most practical mixing procedure B leads to highest gain of compressive strength for samples with various concentrations of HNT and for all periods of curing when compared with other procedures (MW, MP and S).
2. Less variations in the obtained compressive strength of HNT-cement mortars was demonstrated for samples at 28 days using various mixing techniques.
3. Highest enhancement (36%) in compressive strength was achieved for samples with 2.5 wt% of HNT at 14 days using procedure B when compared with control samples.
4. The incorporation of very low concentrations of NAl (0.02 wt% and 0.1 wt%) using procedure (B) results in higher values of compressive strength at age 7 days and 28 days. However, at 28 days, highest compressive strength was recorded for samples with the lowest percentages of NAl (0.02 wt%) using procedure MW when compared with procedure B as well as with control samples.
5. A very slight difference in compressive strength at 28 days was observed when two procedures (B and MW) were used to add 0.1 wt% of NAl.
6. Highest enhancement (21.7%) in compressive strength was recorded for reinforced cement mortars with (2.5 wt%) of at 28 days when procedure S was used. However, further investigation is needed to explore the influence of such mixing method using other percentages of HNT.

References

[1] S.S. Samal, and S. Bal, S., "Carbon Nanotube Reinforced Ceramic Matrix Composites"- A Review". Journal of Minerals & Materials Characterization & Engineering, 2008, Vol. 7, No.4, pp 355-370.

[2] M.S.S. Kumar, N.M.S. Raju, P.S. Sampath, and L.S. Jayakumari, "Effect of Nanomaterials on Polymer Composites-An Expatiate View". E-Journal of Reviews. Advanced Materials Science. 2014, Vol.38, pp.40-45.

[3] M.S. Sadiq, ": Reinforcement of Cement-based Matrices with Graphite Nanomaterials". PhD Thesis, Michigan State University, 2013.

[4] M.S. Morsy, S.H. Alsayed, and M. Aqel, "Effect of Nano-clay on Mechanical Properties and Microstructure of Ordinary Portland Cement Mortar". International Journal of Civil & Environmental Engineering, 2010,Vol.10 No.1, pp.21-25.

[5] R.A. Al-Safy, R. Al-Mahaidi, G.P Simon, and J. Habsud,"Experimental investigation on the thermal and mechanical properties of nanoclay-modified adhesives used for bonding CFRP to concrete substrates". Construction and Building Materials, 2012, Vol. 28, No 1, pp. 769-778.

[6] R.A. Al-Safy, R. Al-Mahaidi, G.P. Simon, and J. Habsuda , "Experimental investigation on the thermo-mechanical properties of VGCF-modified high-functionality based resin for bonding CFRP to concrete subjected to harsh environmental conditions". Journal of Advances in Structural Engineering, 2014, Vol.17 No.12, pp. 1817-1823.

[7] K. Sobolev, and M. Ferrada-Gutiérrez, "How nanotechnology can change the concrete world: Part "2. American Ceramic Society Bulletin, 2005, Vo.11, pp.16-19.

[8] M. Šupová, G.S. Martynková, and K. Barabaszová, "Effect of Nanofillers Dispersion in Polymer Matrices: A Review". Journal of Science of Advanced Materials, 2011, Vo.3, pp.1-25.

[9] J. Bharj, S. Singh, S. Chander, and R. Singh, "Role of Dispersion of Multiwalled Carbon Nanotubes on Compressive Strength of Cement Paste". International Journal of Mathematical, Computational, Natural and Physical Engineering, 2014, Vol.8, No.2, pp. 340-343.

[10] J.M. Makar, and G.W. Chan, " Growth of cement hydration products on single walled carbon nanotubes". Journal of the American Ceramic Society. 2009, Vol.92, pp. 1303-1310.

[11] Y. Qing, Z. Zenan, S. Li, and C. Rongshen, "A comparative study on the pozzolanic activity between nano-SiO_2 and silica fume". Journal of Wuhan University of Technology – Mater. Sci , Ed, 2008, Vol.21, No.,pp. 153-7.

[12] L. Senff, J.A. Labrincha, V.M. Ferreira , D. Hotza , and W.L. Repette, " Effect of nano-silica on rheology and fresh properties of cement pastes and mortars". Construction and Building Materials, 2009, Vol. 23, pp. 2487-2491.

[13] E. Joussein, S. Petit, J. Churchman, B. Theng, D. Righi, and B. Delvaux," Halloysite clay minerals - a review". Clay Minerals, 2005, Vol. 40, pp.383-426.

[14] Materials properties for Halloysite Nanotubes, obtained from Sigma-Aldrich website:

http://www.sigmaaldrich.com/catalog/product/aldrich/685445?lang=en&region=IQ

[15] Materials properties for Nano Al_2O_3, obtained from Sigma-Aldrich website:

http://www.sigmaaldrich.com/catalog/product/aldrich/544833?lang=en&region=IQ

[16] H. Elkady, "Effect of Nano Silica De-agglomeration, and Methods of Adding Super-plasticizer on the Compressive Strength, and Workability of Nano Silica Concrete". The International Institute for Science, Technology and Education (IISTE), 2013, Vol.3, No.2, pp.21-34.

[17] A. Hunashyal., N. Banapurmath, Jain, A., Quadri, S., and Shettar, A.: Experimental investigation on the effect of multiwalled carbon nanotubes and nano-SiO_2 addition on mechanical properties of hardened cement paste. Advances in Materials, 2014, Vol.3, No.5, pp.45-51.

Optimization Process for Using Prepared Nanosilica in Concrete

Najat J. Saleh, Raheek I. Ibrahim, Ali D. Salman

***Abstract*—This study has been devoted to apply the optimization technique involving central composite rotatable design to seek on the optimum conditions for using novel nanosilica (NS) in concrete; this nano silica has been previously prepared from Iraqi sand. Nano silica sand has been shown to improve workability and strength. This research presents the compressive strengths and the microstructure photographs (SEM) of concrete containing nanosilica particles with various sizes of 50, 80 and 100 nm, then compared it with those for control mixture (without nanosilica). Tested results indicated that nanosilica sand significantly improved compressive strength of concrete. The strength improvement was also dependent on the particle size and concentration of nanosilica particles (replacement content). Concrete containing (NS) with 50 nm gave higher compressive strength compared with (NS) of 80 nm and 100 nm. The optimization results proved that the enhancement in compressive strength is 30.149% at optimum conditions. By varying the replacement contents of NS as 2%, 6%, and 10% by weight of cement, the optimum replacement content was shown to be 8% for all nanosilica particle sizes used.**

Keywords: **Silica nanoparticle, Silica sand, Optimization, Portland cement.**

1- INTRODUCTION

Nanotechnology has been defined by Drexler et al. as ‘‘the control of the structure of matter based on molecule-by-molecule control of products and by-products’’. Nanotechnology can be considered as the most modern aspect in the fields of science and technology **[1].** Because nanotechnology has great market potential and economic impact, the need for research and exploration in this field and of its applications has been growing significantly during the last few decades **[2].**

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R. I. Ibrahim is with Electromechanical Engineering Department/ University of Technology/ Baghdad- Iraq (corresponding author: mobile: +9647728469947, E-mail: doctorraheek@yahoo.com)
N. J. Saleh (njt_saleh@yahoo.com) , and A. D. Salman (alidawood327@yahoo.com) are with Chemical Engineering Department/ University of Technology/ Baghdad- Iraq

Nanoparticles of silica sand have been researched progressively and produced due to the unique features as a result of size reduction. Silica sand nanoparticles have proved to be a very effective additive to concrete by improving durability, mechanical properties and workability to produce high performance concretes **[3].**Nano-scale substances have a great impact on concrete mixtures in order to their higher surface area. A lot of researches used various nanomaterial's to show its effect on the concrete. One of the farthest interests is the nanosilica (NS), this has been attributed to its Pozzolanic characteristics **[4].** The aim of this research is to gain better understanding of the behavior of materials on the nano-scale level as well as determine how to improve the microstructure of cementations materials. Incorporation of nanomaterial's into the matrix to improve concrete mechanical properties has emerged as a promising research field. Because there is no present work concentrated along with this silica sand nanoparticles conduct inside the concrete, this study optimized the operating parameters like NS particle size and percentage of addition to the concrete, using experimental design procedure utilize a second order polynomial model equations, the optimization technique employed **[WinQSB]** and **[Statistica]** software to analyze the developed model equation and predicate the optimum conditions, furthermore, the experimental validation of proposed model was also involved.

2- MATERIALS AND METHOD

The prepared nanosilica specifications and characterization are available in our previous work **[5]**.

2-1 Mix Proportions

Thirty Concrete mixtures was made for compressive strength test were included in the study, there are ten experiments to be carried out for every curing age in sequence according to experimental design. All the concrete specimens had a water-to-cementations ratio

(w/c) of 0.48 and the mixture of concrete was mixed by 4 ratio of coarse aggregate and 2 ratio of fine aggregate and 1 ratio of cement by weight according to ASTM specifications (C109)**[6]**. Percentage of the NS varied from 0, 2%, 6% and 10% by mass of the cementations materials. Concrete with NS were compared with that the control mixture to evaluate the influence of dosage and particle size of the (NS). Table (1) shows physical properties of (NS) groups consumed. Specifications of mixing attributions of concrete are shown in Table (2). The initial band represents the CM sample (without NS), while the others represents the addition ratios of NS to concrete as 2%, 6%, and 10% by weight.

Table (1): Physical properties of nanosilica groups.

Physical Properties			
Groups	SSA m^2/g	Particle size group (nm)	Avg. particle size (nm)
NS1	40	(30-100)	50
NS2	28.4	(60-120)	80
NS3	18	(90-170)	100

NS: Nano-silica, SSA: specific surface area

Table (2):Mixing proportion of concrete samples.

Mix proportion 1 : 2 : 4 (550:1100:2200) g							
Mix ID	Perc. % (NS)	Parti. Size (nm)	Nano-silica (gm)	Fine Aggreg (gm)	Cement (gm)	Water (ml)	Coarse aggr. (gm)
M0	0	0	0	1100	550	350	2200
M1	2	50	11	1100	539	350	2200
M2	10	50	55	1100	495	350	2200
M3	2	100	11	1100	539	350	2200
M4	2	80	11	1100	539	350	2200
M5	6	50	33	1100	517	350	2200
M6	10	80	55	1100	495	350	2200
M7	6	100	33	1100	517	350	2200
M8	10	100	55	1100	495	350	2200
M9	6	80	33	1100	517	350	2200

2-2 Specimen Preparation, Curing, and Testing

To prepare concrete specimens mixtures solid materials (fine, coarse aggregate and ordinary Portland cement) were dry mixed first in a Hobart mixer (mechanical mixing) at an ambient temperature of about 30℃. (NS) and water were mixed by using Ultrasonic mixer with 750 Watts power input for 5 min. The sonicated mixture was then mixed in a Hobart mixer, for 1 min at low speed. For each concrete mixture, after that, (100×200mm) cylindrical specimens were cast for compressive test according to ASTM (C39-4). The molded specimens were covered with wet burlap for the first 24 hrs to prevent moisture loss. After remold, the specimens were cured in a fog room at a temperature of about 30 ℃ until the time of testing. Compressive strength of the concrete specimens has been determined at 7, 14, and 28 days according to ASTM C (780-002)[7].

3- OPTIMIZATION PROCESS

The operating variables such as particle size (X_1), and (NS) adding percent (X_2), at 7, 14, 28- day aging times, were correlated in non-linear second order model equation, to show their effects on compressive strength of concrete samples. In the optimization process, the compressive strength represents the objective function (Y) should be maximized, as in Eq. 1.

$$Y = a_0 + a_1 X_1 + a_2 X_2 + a_3 X_1 X_2 + a_4 X_1^{\ 2} + a_5 X_2^{\ 2} \quad ..(1)$$

The model equation coefficients were estimated using [STATISTICA**]** software (version 8.0). The operating variables of the developed model were optimized using [WinQSB] software (version 1.0) in order to find the optimum values of operating conditions for enhancing the compressive strength. Then experimental validation was employed.

3-1 Design of Experiments

Before studying any process, it is necessary to determine the parameters which have a considerable effect on the system behavior, or the factors that influence the system objective function. So one must carry out several experiments to cover the effect of each parameters as well as the interactions between these parameter if they are not independent. The systematic method which satisfies the above function with minimum number of experiments is called "Experimental Design". The application of the experimental design to planning the experiments required to examine the system ,will extract the information from pre-existing data by using a statistical method to interpret the results in a regular form with the minimum number of observations**[13].** The total number of treatment combinations in the above design is equal to (2^k+2k+1),where k is the number of variables , plus additional further treatments to take the lack of fit and experimental error into account, the central composite rotatable

design will reduce the number of the experiments required for the system.

3-2 Experimental Trials

The central composite rotatable design of two variables was used. The coded levels were related to real process values of these variables as follows:

$$X_{coded}=\frac{[Xact.-Xcenter]}{\frac{Xcenter-xmin.}{\sqrt{k}}} \quad \text{.....(2)}$$

$$X1=\frac{P.S-80}{21.21} \quad \text{.....(3)}$$

$$X2=\frac{percenteg-6}{2.82} \quad \text{.....(4)}$$

According to experimental design of the two variables, there are ten experiments carried out for every curing age. The working ranges of coded and corresponding real variables are illustrated in table (3), where the coded values (+1, -1, 0) represent the maximum, minimum, and average values respectively.

Table (3): Working range of coded and corresponding real variables.

Coded level	Real level	
	Particle size (nm) (X1)	Percentage (%) (X2)
-1	50	2
0	80	6
+1	110	10

4- RESULTS AND DISCUSSION

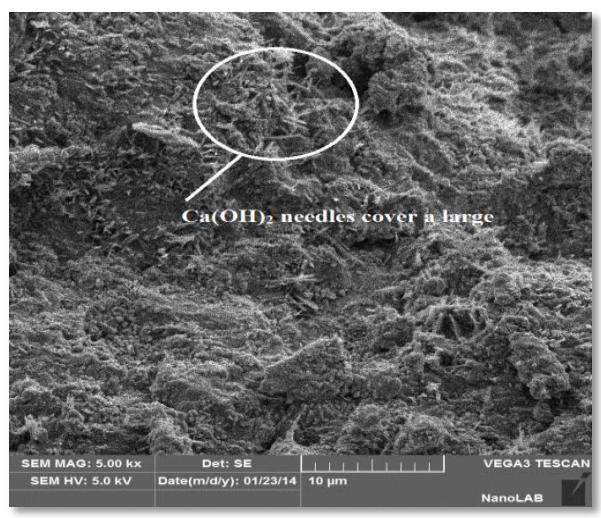
4-1 Characterization And Analysis Of Concrete Mixture

4-1-1 SEM Observations And Microstructure Analysis Of (CM)

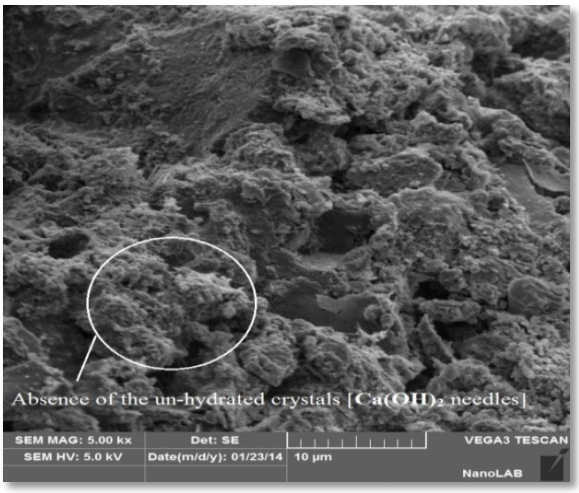
Figure (1,A) shows the SEM micrographs of the control concrete mixture at the age of 28 days demonstrate porous structure that is full of large size pores and presence of $Ca(OH)_2$ that over-shadowed. Also it can be seen from same figure the existence of many CH crystals connected to the C-S-H gel which indicates that the hydration process is not completed and also explains the low records of compressive strengths for the CM. Also, the same photo show that the concentration of the CH is higher than the C-S-H gel concentration and that the CH hydrate needles cover a large area. These results are in good agreement with the results obtained by other researchers **[8], [9]** and **[10].**

4-1-2 SEM Observations And Microstructure Analysis Of Ns6(30-100)

The SEM image of mixture NS6(30-100) (Nano-silica according to the particle size analyzer for this group ranges from (30-100) nm, higher volume percent of particle size is (50) nm, is shown in Figure (1)B that is prepared with 6% NS of nominal particle size group (30-100) nm and demonstrate that the microstructure of the concrete mixture after incorporating (NS) at the age of 28 days is dense and more organized with a small number of Ca(OH)2 crystals and small sized pores while the CM the C-S-H gel existed in the form of clusters lapped and jointed together by many CH needles hydrates. It can also be noticed from the same photo that the CH needles are invisible and there is a compact structure with the absence of the un-hydrated crystals, voids, more uniform and homogeneous than that of the (CM) sample which explains the superior compressive strength results. This could be due to the high activity of many particles that promote the pozzolanic reaction to produce more C-S-H gel in order to record high compressive strength at early age which is confirmed by the strength results. NS consumes calcium hydroxide crystals, fills pores to increase the strength, reduces the size of the crystals at the interface zone and transmute the calcium hydroxide feeble crystals to the C-S-H crystals, and improves the interface zone and cement paste structures. These results are in good agreement with the results obtained by previous works **[9]**,[**10]**and**[11]**.



(A)



(B)

Figure (1): SEM images of concrete (A) before incorporating NS, and ((B) after incorporating NS at 28 days.

Table (4): Compressive strength results at 7, 14 and 28-days.

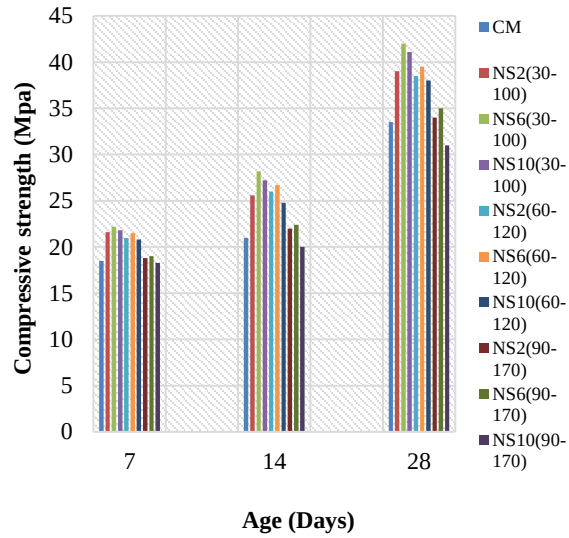
Mixture I.D.	Compressive strength (MPa)		
	7-days	14-days	28-days
CM	18.5	21.0	33.5
NS2(30-100)(50)	21.6	25.6	39
NS6(30-100) (50)	22.2	28.2	42
NS10(30-100) (50)	21.8	27.2	41.1
NS2(60-120) (80)	21	26	38.5
NS6(60-120) (80)	21.5	26.7	39.5
NS10(60-120) (80)	20.8	24.8	38
NS2(90-170) (100)	18.8	22	34
NS6(90-170) (100)	19	22.4	35
NS10(90-170) (100)	18.3	20	31

CM: control mixture, NS: nanosilica mixtures at different adding percents and particle sizes.

4-2 Compressive Strength Development

4-2-1 Effect of Age on Compressive Strength

Table (4) shows compressive strength resultant 7, 14 and 28-days. Concerning the tests, the effect of age on compressive strength for CM, and NS mixtures, it can be concluded from Figure (2) that the compressive strength consistently increases with the increase in age for all mixtures.



Figure(2) : Effect of age on compressive strength of concrete.

Also, it can be seen from the same figure that the mixture NS6(30-100) (50 nm) achieved the highest strength at all ages which confirms the fact that compressive strength increases with both increasing the percentage of NS to optimum percentage and decreasing the NS particle size. Another observation that is clear that adding NS is better than without. An example for the previous conclusion is that, at the age of 14 days the mixture NS6(30-100) (50nm) recorded a strength of 28.2MPa, where the (CM) mixture gave a strength of 21MPa at the same age. This early strength effect of NS is due to its influence in accelerating the pozzolanic transformation of C_3S, C_2S and CH into the C-S-H gel which is responsible for giving the concrete its strength. Gaining NS enormous early strength is also attributed to the high packing efficiency of nanoparticles.

4-2-2 Effect of Particle Size Of (Ns) On Compressive Strength

The variation in the results seems to be a function of the particle size. For instance, the compressive strength of mixture NS6(30-100) nm was 42MPa at 28

days slumped to 39.5MPa for mixture of NS6(60-120) nm and the mixture NS10(60-120)nm was 38MPa dropped to 31MPa for mixture of NS10(90-170) nm at the same age and percentage as shown in Figure (3)A, B and C. A fair explanation for this trend is that as the particle size of NS increases the packing efficiency decreases leading to a drop in the compressive strength. This effect of NS particle size on compressive strength can also be attributed to the large surface area that is available for pozzolanic reaction. That is why in the case of small particle size of NS, moreC-S-H is produced and thus its concentration in the mixture increases, leading to a rise in compressive strength. It can be observed from the results that the surface area decreases with the increase in the particle size as in other mixtures of NS and as in the control mixture (CM) which yielded less compressive strength at the same age

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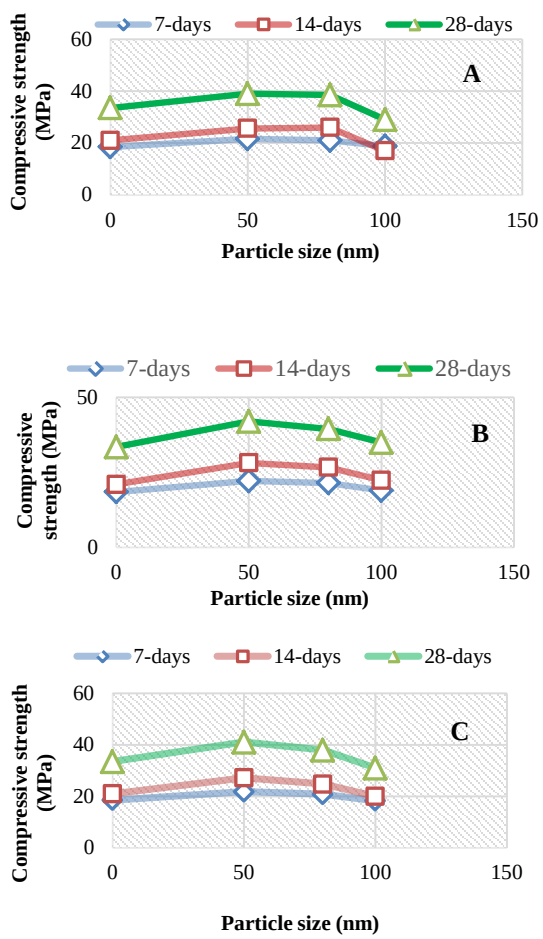
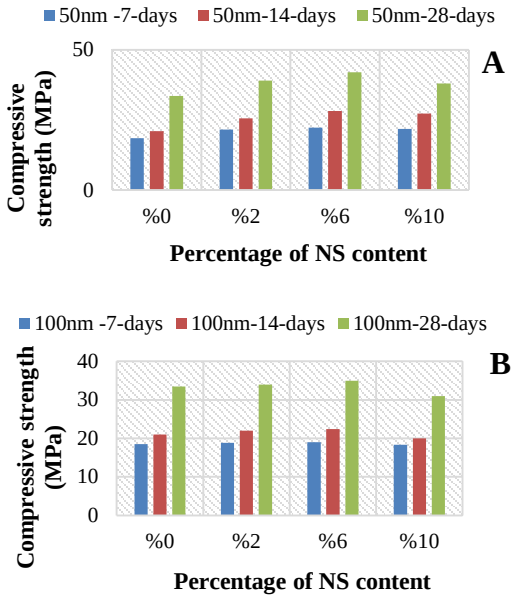


Figure (3): Effect of particle size of (NS) on compressive strength of concrete for A (2%), B(6%), and C(10%) NS.

4-2-3 Effect Of (Ns) Dosage On Compressive Strength

Compressive strength development of the concrete incorporating different dosages of NS is given in Figure (4) A, B and C**.** In comparison with that of CM, the concrete with NS had higher compressive strengths than the CM from 1 to 28 days as expected. However, the strength of the mortars increased with the increase in NS dosages to optimum value up to 28 days. By examining the strength of the concrete specimens prepared with different percentages of NS for the same particle size, it can be seen from Figure (4) that the mixtures with percentage of 6%NS displayed higher compressive strength than their counterparts prepared with low NS percentage which is the same result in the study conducted by Haruehansapong **[12]**. For example, the specimen NS2 (30-100) nm exhibited a compressive strength of 39MPa at 28 day and 2% NS, while recorded 42MPa at 6% NS at the same age. Conversely, a different observation was detected for mixtures which incorporated NS of nominal particle size NS6(90-170) nm where the compressive strength result at 6% and 28 days was 35MPa, whileNS10(90-170) slumped to 31Mpa and mixture NS6(60-120) nm where the compressive strength result at 6% and 14 days was 26.7MPa while NS10(60-120)nm dropped to24.8Mpa. This means that increasing the percentage of NS is useful in increasing strength to a certain limit after which any increase in the NS percentage leads to a decrease in the compressive strength. These results are in good agreement with the results obtained by cox **[13].**



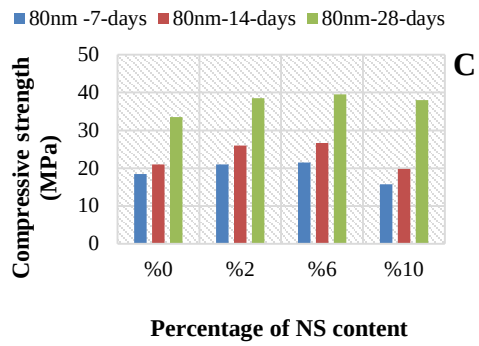


Figure (4): Effect of (NS) percentage on compressive strength for A (50nm), B (80nm), C(100nm) NS.

Table (5): ANOVA of compressive strength results at 7, 14 and 28- days.

ANOVA	7- days	14-days.	28-days
P	0.000171	0.010656	0.015016
F	126.6339	14.99833	12.44144
Ms residual	0.032778	0.916667	1.756667
df residual	4	4	4
SS residual	0.131111	3.666667	7.026667
MS model	4.150778	13.74847	21.85547
df model	5	5	5
SS model	20.75389	68.74233	109.2773
Adjusted (R^2)	0.985875	0.886064	0.864063
Multiple (R^2)	0.993722	0.949362	0.939584
Multiple (R)	0.996856	0.974352	0.969321

MS: mean square error, df: degree of freedom, SS: Sum of square error.

5- Optimization Results

The Optimization results proved that an improvement in compressive strength of 30.149**%** at optimum conditions (8% of NS adding percent and 50nm particle size) in 28 days age. The model results are in good accordance with experimental measurements as verified by analysis of variance (ANOVA). The ANOVA results are explained in Tables (5), the results show higher values of model R^2 than adjusted R^2 for all cases studied which proved the statistical acceptance of the model. Table (6) shows the optimum conditions obtained using [WinQSB] technique.

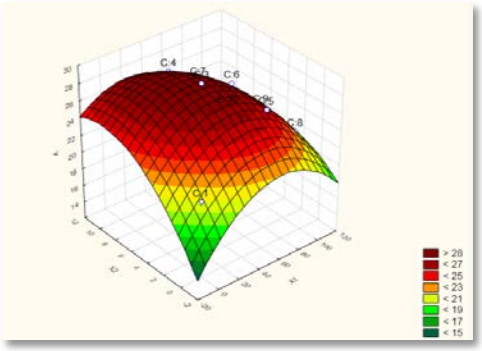
Table (6): Optimum conditions of compressive strength at 7, 14, and 28 days.

The results of multiple regression analysis of the developed model equations and goodness of fit analysis were estimated using [STATISTICA] software and have been illustrated in Table (7). Figures (5 A, B, C), show the effect of operating parameters in a 3-D graphs on the values of compressive strength at 7, 14 and 28-day age. The figures have been shown clearly the positive influence of reducing particle size of NS and influence of addition percent together on compressive strength values.

Age	Optimum Conditions		
	X1 (%)	X2 (nm)	Y max (MPa)
7- days	8	50	23.8
14- days	8	50	28.2
28- days	8	50	43.6

Table (7): Results of multiple regression analysis.

Property	Regression coefficients						Goodness of fit analysis	
	a0	a1	a2	a3	a4	a5	Corr. coeff. (R)	Var. (%)
Comp. Strength at 7-days	18.48682	0.092751	0.465893	-0.001348	-0.000860	-0.031639	0.99967	99.935
Comp. Strength at 14-days	20.83473	0.131836	1.439693	-0.006123	-0.001186	-0.085573	0.99161	98.328
Comp.e Strength at 28-days	33.34306	0.134229	2.009042	-0.009317	-0.001271	-0.110985	0.99469	98.942



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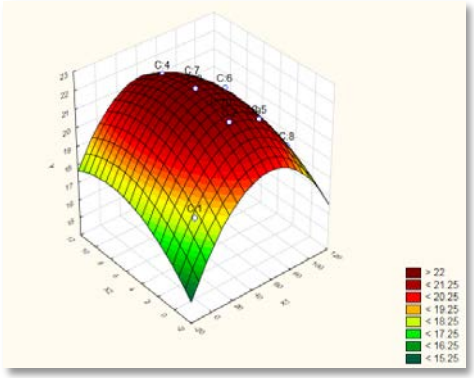

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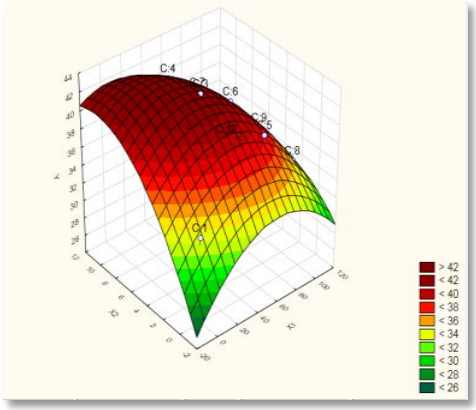

(C)

Figure (5): 3-D graph for the effect of operating variables on compressive strength of concrete at (A) 7-day age, (B) 14-day age, (C) 28-day age.

6- CONCLUSIONS

The following conclusions can be concluded from the experimental result and optimization as:

1. Nano silica (NS) gives both pozzolanic activity and packing ability; thus it increases the compressive to a greater extent than (CM). Moreover, very small (NS) particles can participate in the hydration process to create C–S–H through reaction with $Ca(OH)_2$.

2. Incorporating (NS) to concrete mixtures enhanced their mechanical properties, by yielded higher compressive than control mixture due to its high packing efficiency in filling voids, its effect in promoting pozzolanic reaction because of its high surface area. So, it has been exhibited a relatively high early strengths.

3. Within the particle size and dosage range examined at all curing ages, the strength generally increased with the addition of NS, the optimum percentage of (NS) were (8%)and 50nm particle size gives the higher the strength of concrete mixtures, optimization results proved that an improvement in compressive strength of 30.149**%** at optimum conditions in 28days age.

4. SEM, and compressive strength outcomes from optimization technique, gained a considerable exhaustion in (CH) in the pozzolanic reaction when adding the (NS), while rising in the dosage of (NS) from 2% to 6%, enhances the exhaustion of CH. Submits an enhancement in interpretation. Conversely a different trend observation was detected, when increasing the (NS) dose from 6% to 10% led to slumped strength.

5. Finally, the prepared nanosilica Incorporating to cement mortar mixtures afford excellent boosting in mechanical properties, as well as it has been conducted from local silica sand, this award it high applications range especially in construction materials.

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REFERENCES

[1] Hornyak, Tibbals, Dutta and Moore.,2009, "Introduction to Nanoscience and Nanotechnology" CRC Press, pp.178-196.

[2] L.Filipponi and D.Sutherland.,2013" Nanotechnologies: Principles, Applications, Implications and Hands-on Activities", Directorate-General for Research and Innovation Industrial technologies (NMP),pp.19,137.

[3] Lazaro,A.Quercia,G.Brouwers, H.J.H.,2012." Production And Application of A New Type of Nano-Silica In Concrete" Material innovation institute and Eindhoven,pp.466-473.

[4] K.Sobolev, Ferrada-Gutiérrez, M.,2005,"How Nanotechnology can Change The Concrete world: Part 1." American Ceramic Society Bulletin, 84(10), 14–7.

[5] N. J. Saleh, R. I. Ibrahim, and A. D. Salman, "Characterization of Nanosilica prepared from local silica sand and its application in cement mortar using optimization technique", Adv. Powder Tech. Jou. (under publication),doi: 10.1016/j.apt.2015.05.008

[6] ASTM C (109-08).,2008,"Standard Test Method for Compressive Strength of Hydraulic Cement Mortars". ASTM International.

[7] ASTM C(780–002),2004."Standard Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry" Annual Book of ASTM Standards, Vol. 04-02.

[8] L.P.Singh, S. K. Agarwal, S. K. Bhattacharyya, U. Sharma and S. Ahalawat, 2011."Preparation of Silica Nanoparticles and Its Beneficial Role in CementitiousMaterials",Central Building Research Institute, Vol. 1, No.1,pp. 44-51.

[9] J.S.Belkowitz and D.Armentrout.,2010" An Investigation of Nano Silica in the Cement Hydration Process", Concrete Sustainability Conference,pp.1-15.

[10] G.Quercia,Spiesz,G.Hüsken,andH.J.H.Brouwers, 2014,"SCC Modification by Use of Amorphous Nano-silica" ,Cement and Concrete Composites,45, pp.69–81.

[11] M. Aly, M.S.J. Hashmi, A.G. Olabi, M. Messeiry, E.F. Abadir, A.I. Hussain , 2012."Effect of Colloidal Nano-silica on The Mechanical and Physical Behavior of Waste- glass Cement Mortar", Materials and Design ,33,pp.127–135.

[12] S.Haruehansapong, T.Pulngern and S.Chucheepsakul,2014."Effect of The Particle Size of Nano-silica on the Compressive Strength and the Optimum Replacement Content of Cement Mortar Containing Nano-SiO2"Construction and Building Materials, 50.pp.471–477.

[13] Cox and D.R., 1978,"Planning of experiments ",new York.

[14] ASTM C(39-04).,2004, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens" Annual Book of ASTM Standards, Vol. 04-02.

Effect of Wastewater on Concrete Tanks in Wastewater Plants

Dr. Mohammed Ali I. Al-Hashimi, Sameh Badry Tobeia, Ayat Hussein Mahdi,and Hadel A. Ibrahim

***Abstract*— Sulfates found in wastewater have a great effect on concrete properties in wastewater plants, concrete compressive strength adversely affected by sewage aggressive environment due to the sulfate attack. In order to investigate this action several concrete cubs has been made and cured with ordinary water as well as sewage water taken from different stages of Al-Rustamiyah wastewater plant (grit removal tank, primary sedimentation tank and secondary sedimentation tank). Cubes made in four groups, all samples cured by using ordinary water for 28-days followed by another 28-days curing with sewage water except the reference group.**

Concrete compressive strength test shows a significant descending with concrete compressive strength of 20.1 MPa and 21.2 MPa for cubs curd with sewage water taken from grit removal and primary sedimentation tank respectively(descending ratio 25.6% and 21.5% respectively), in comparing with 27.0 MPa concrete compressive strength of reference group .

***Keywords*: Wastewater Plants, Salt effect, Sulfate, Concrete compressive strength.**

I. Introduction

Concrete is a construction material, consist of a mix almost homogenous. This mix contains solid particles with different sizes called aggregate bonding with hardening cement paste. Concrete also contain gaps fill with air [(1)]. The durability of concrete is one of its most important properties due to its essential that concrete should be capable of withstanding the conditions for which it has been designed throughout the life of structure. Lack of durability can be caused by external agents arising from the environment or by internal agents within the concrete this action can be depend on the salts (sulfates, chloride and others) ability to dissolve in water and their concentrations. Other factors like in (pH) and temperature affect the sulfate attack and its results on concrete, especially for concrete structures exposed to sewage like treatment tanks and sewage transpose pipes. Usually sulfates and chloride found in solid case and has no effect on concrete but in presence of water these salts begin to dissolve and attack the cement paste in concrete [(2,3&4)].

Dr. Mohammed Ali I. Al-Hashimi,Building and Construction Engineering Department, University of Technology /Baghdad. And Head of Building and Construction Engineering Department, Dijlah University College. e-mail:mohashimi2003@yahoo.com.
Sameh Badry Tobeia, Building and Construction Engineering Department, University of Technology /Baghdad.
Ayat Hussein Mahdi, Building and Construction Engineering Department, University of Technology /Baghdad.
Hadel A. Ibrahim Building and Construction Engineer.

II. Aim of work

This work aimed to experimentally investigate the effects of salts (sulfate, chloride, and some other salts) present in wastewater on the concrete compressive strength of Wastewater Plants Concrete Tanks. In order to reach this goal several concrete cubes has been made, cured and tested for compressive strength.

III. Sulfate effect on concrete

Sulfate effect on concrete occurred as internal or external attacks. Internal attack occurred due to internal agents as sulfates present in concrete materials (cement, aggregate and water), in this case the reaction occurs fast and quick and concrete strength adversely affected even at three days age. Where sulfate found in sand is more dangerous on concrete than those found in gravel due to the fineness of the practical[(1)] On the other hand external attack started when concrete exposed to sewage or soil or groundwater, and the mechanism of sulfate attack is similar to the internal attack but its effect take longer time to be obvious. In this type of attacks sulfate concentration still approximately constant due to the continuous compensate. The two major factors affected sulfate attack intensity are: sulfate concentration in solution and the movement of solution, the adversely effects are obvious with the increase of these factors [(1)].

Also, Chlorides, particularly calcium chloride and (to a lesser extent) sodium chloride have been shown to leach calcium hydroxide and cause chemical changes in concrete, leading to loss of strength, as well as attacking the steel reinforcement [(1)].

IV. Treatment tanks concrete in wastewater plants

Generally, the durability of concrete structures decreases due to several chemical and physical factors. Concrete in wastewater plants exposed to a combination of chemical and physical factors leading to rapid concrete deterioration represented by lack in strength, disintegration, cracks and reinforcing steel corrosion. Since concrete in wastewater plants exposed to sulfate from its environment (external attack), therefore the durability and permeability of concrete have significant effect on deterioration and lake in concrete properties. Municipal wastewaters contain different types of materials like: fats, oil, detergents, organic matters, human excreta, food wastes, industrial wastes and different

types of material which cause concrete deterioration like sulfates, chlorides, nitrogen and phosphate. Variations in pH and biological reactions in wastewater also effect on concrete strength [2&4].

V. Mechanism of Sulfate Attack

Sulfate in sewage, groundwater or soil usually found in form of calcium sulfate (gypsum), magnesium sulfate and sodium sulfate which they react with calcium hydroxide $Ca(OH)_2$ and tricalcium aluminate to produce gypsum and calcium sulfoalminate respectively. Calcium sulfate attack only the calcium aluminate and produce calcium sulfoalminate, while magnesium sulfate attack calcium silicate in addition to calcium aluminate and calcium hydroxide as in Eq.1. These reactions accompanied by volumetric increase causing disintegration of concrete.

$$MgSO_4.7H_2O+3CaO.2SiO2.aq. \rightarrow CaSO_4.2H_2O+Mg(OH)_2+SiO_2.aq. \quad \text{------- (1)}$$

Magnesium sulfate has more damaging effect than other sulfates because it leads to the decomposition of the hydrated calcium silicate as well as of calcium hydroxide and calcium aluminate. The sulfate attack becomes more aggressive as their concentration increase as well as concrete permeability [1].

VI. Wastewater Treatment Stages

Wastewaters must be treated from different types and sizes of particles, fats removal (skimming) before disposal to the rivers. Wastewater treatment plants usually consist of screens, comminutor, grit removal, grease removal, flotation, primary sedimentation tank, secondary sedimentation tank [5,6&7].

In this work wastewater used for curing concrete cubes taken from three different stages from Al-Rustamiyah wastewater treatment plant as follows:

1) Grit Removal Tank: this tank is the first part of wastewater treatment plant used for separation of large particles with different densities [5,6&7].

2) Primary Sedimentation Tank: used to separate suspended organic matter, chemicals and may be used to enhance removal of colloids and small particles and to precipitate phosphor [5,6&7].

3) Secondary Sedimentation Tank: used to remove small particles that did not removed in the first stage [5,6&7].

VII. Experimental Work

In order to simulate the effects of sulfate and other compositions (chlorides, magnesium and calcium) usually found in wastewater, concrete cubes has been made and cured in ordinary water as well as sewage. The materials properties used in this work included materials used in concrete in addition to the sewage as follows:

1) Ordinary Portland cement used to made concrete cubes with initial and final setting time of 112 minute and 210 minute respectively, cement compressive strength 24.1 MPa at one week and slump value of 10 mm. Where these result satisfy the requirements of Iraqi Standard Specification 1984[8].

2) Wastewaters experiments have been used to measure wastewater parameters taken from Al-Rustamiyah wastewater treatment plant like: (sulfate, chloride, and magnesium). Table (1), represents experiments results for different wastewater stages.

Table (1): Wastewater parameters used for concrete cubes curing

Sewage tank type	Sulfates mg/L	Chlorides mg/L	Magnesium mg/L	Calcium mg/L	Hardness mg/L	Turbidity (NTU)	pH
Grit chamber tank	570	494.84	146.74	701.4	100	171	7.91
Primary sedimentation tank	303.66	414.87	114.4	621.24	152	91.1	7.95
Secondary sedimentation tank	266.66	364.88	63.47	280.56	20.4	80	8.52

Sulfates, Chlorides and other parameters relationship illustrated in Fig. (1), all parameters decreased gradually except hardness which increased in the primary sedimentation tank this may be occurred due to the sediments that are agitated by influent wastewater velocity.

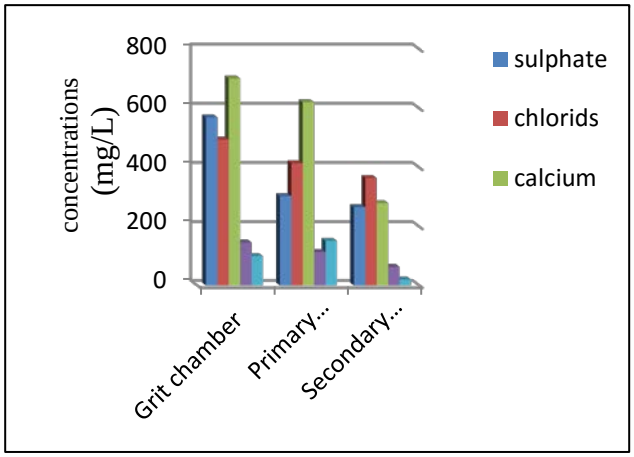

Fig. (1) Concentrations in wastewater treatment plant units

In this work 12 concrete cube with 150 mm has been made, concrete mix ratio 1:1.5:3. These cubes divided to four groups according to the water and wastewater type used in curing process as follows:

A) First group (group A): concrete cubes cured with ordinary water for 28 days.
B) Second group (group B): divided into two stages, first stage curing with ordinary water for 28 days, second stage partial immersing for the same cubes with wastewater from grit chamber tank for another 28 days in order to simulate the same conditions which the concrete exposed like high concentration of sulfates, and chlorides.
C) Third group (group C): Also consists of two stages, first stage full immersing with ordinary water for 28 days, and in the second stage the cubes partial immersing with wastewater taken from primary sedimentation tank for 28 days.
D) Fourth group (group D): first stage is curing with ordinary water for 28 days and second stage is partial immersing for the concrete cubes with wastewaters taken from secondary wastewater tank for another 28 days.

VIII. Concrete Compressive Strength Test Results

In structural design concrete usually assumed to resist compressive strength only, therefore Concrete compressive strength considered the basic criteria to determine the concrete type, durability for concrete structures. Also tensile strength and flexural strength can be measured as a percentage of compressive strength. Therefore, in this work 12 concrete cubes tested for compressive strength by using hydraulic test machine (ELE International ADR 3000) as appear in Fig.(2)

Fig.(2) Compressive strength test machine

All concrete cubes has been tested for compressive strength, group A tested at 28 day other groups tested after curing of 56 day divided in to two stages at first curing with ordinary water for 28 day followed by another 28 day with partial immersing in sewage taken from different tanks of wastewater treatment plant. Fig.(3) through Fig.(6) show the concrete cubes during test procedure and test results can be summarized as in table (2).

Fig.(3) Group A concrete cubes

Fig.(4) Group B concrete cubes

Fig.(5) Group C concrete cube during the test

Fig.(6) Group D concrete cube during the test

Table (2): Concrete compressive strength test results

No.	Group	Concrete cube name	Concrete compressive strength MPa	Concrete compressive strength average for each group MPa,	Concrete compressive strength descending ratio corresponding to group A
1	A	RC1	27.3	27.0	-
2		RC2	26.3		
3		RC3	27.2		
4	B	S1-1	20.4	20.1	25.6 %
5		S1-2	20.6		
6		S1-3	19.4		
7	C	S2-1	22.1	21.2	21.5 %
8		S2-2	20.6		
9		S2-3	21.0		
10	D	S3-1	28.9	27.1	≈ 0 %
11		S3-2	26.1		
12		S3-3	26.4		

The relationship of concrete compressive strength versus sulfate and chlorides concentration illustrated in Fig.(7) and Fig.(8), as well as Fig.(9) show the effect of the increased in magnesium concentration.

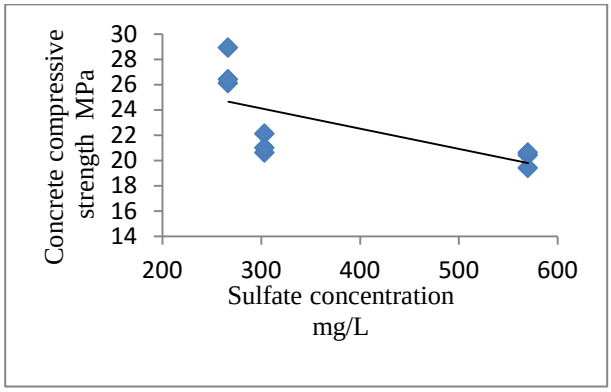

Fig.(7) Concrete compressive strength VS. sulfate concentration

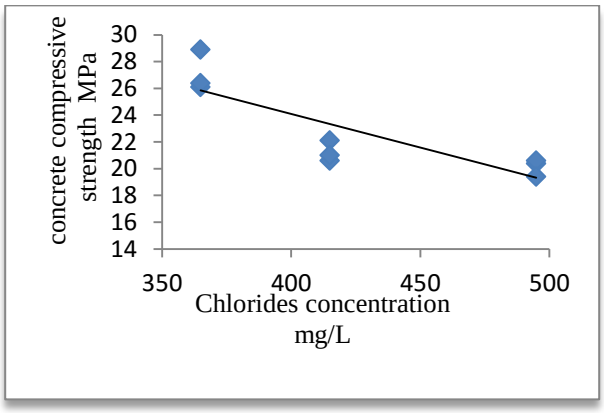

Fig.(8) Concrete compressive strength VS. Chlorides concentration

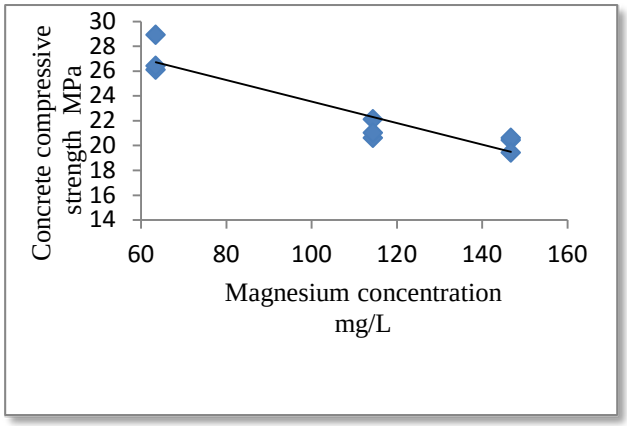

Fig.(9) Concrete compressive strength VS. magnesium concentration

IX. Results and discussion

In this work, four groups of concrete cubes have been made the first group (group A) cured in ordinary water for 28-days. The other three groups (group B, group C and group D) cured in two stages, the first curing stage done by using ordinary water only for 28-days. The second stage done by using sewage water taken from Grit Removal tank, Primary Sedimentation tank and Secondary Sedimentation tank from Al-Rustamiyah wastewater plant in order to simulate concrete behavior in wastewater plant concrete tanks. Curing in wastewater done by partial immersing of the concrete cubes for 28-days because the worst salts attack occur due to the repeated cycles of wet/dry concrete which cause sulfate, chloride and other salts deposition inside concrete pores after water evaporation continuously and leading to disintegration of concrete. The concrete compressive strength test shows that group (A) (cured only with ordinary water) gave the highest compressive strength with average of 27 MPa this value decreased to 20.1 MPa and 21.2 MPa with compressive strength descending ratio of 25.6% and 21.5% for group (B) (cured with ordinary water and sewage from Grit Removal tank) and group (C) (cured with ordinary water and sewage from Primary Sedimentation tank) respectively. The significant reduction in concrete compressive strength occurred for group (B) because this part of wastewater plant does not remove salts or acids therefore, chemical added usually used to reduce the organic materials. On the other hand group (D) cubes (cured with ordinary water and sewage from Secondary Sedimentation tank) gave high concrete compressive strength because most concentrations of sulfate, chloride and other salts have been reduced before this stage.

X. Conclusions

From the results of experimental work the following facts can be concludes:

1. Concrete grit removal tanks highly affected by the presence of sulfate, chloride and other salts in high concentrations which reduce compressive strength of tank concrete structure, in compare with other steps of wastewater plants.
2. The concrete of Grit Removal and Primary Sedimentation tanks deterioration in similar manner due to high concentrations of salts.
3. Secondary Sedimentation concrete tanks in wastewater treatment plants are less affected by the presence of sulfate, chloride and other salts due to the reduction in their concentrations in the first stage of treatment.

XI. References

1-A. M. Neville and J. J. Brooks, "Concrete Technology", Second Edition, Prentice Hall, 2010.

2- WEI Chao-hai., WANG Wen-xiang, DENG Zhi-y iand WU Chao-fei, "Characteristics of high-sulfate wastewater treatment by two-phase anaerobic digestion process with Jet-loop anaerobic fluidized bed", Journal of Environmental Sciences,No.3, V.19,2007 , pp. 264–270.

3- A. K. Parande , P. L. Ramsamy, S. Ethirajan , C. R. K. Rao and N. Palanisamy, “Deterioration of reinforced concrete in sewer Environments”, Institution of Civil Engineers, March 2006 Issue ME, pp. 11–20.

4- Coatings manual, "Basic on corrosion in wastewater collection and treatment system ",2011,pp. 25, Internet.

5-E.W.Steel, "Water Supply and Sewerage ", McGraw-Hill, fifth Edition, 1979.

6- Howard S. Peavy, Donald R. Rowe & George Tchobanoglous, ”Environmental Engineering”, McGraw-Hill, First Edition, 1985.

7- Syed R. Qasim, "Wastewater Treatment Plants: Planning, Design, and Operation", College Publishing, International Edition, 1986.

8- Iraqi standards (No.5)-1984, central organization for standardization and quality control (COSQC) ,Baghdad 1984.

Behavior of Light Weight Concrete Using Polymer Materials

Hussein H. Karim[1], Hisham A. Khalid[2], and Saadia A. Sahii[3]

***Abstract*—Most of the recent works in construction industry in Iraq were focused on investigating the validity of local raw materials as an alternative to the imported materials necessary for some practical applications especially in thermal and sound insulation. Therefore, this research aimed to investigate the possibility of using polyurethane polymer as a foaming agent with different percentages by weight of cement for the production of lightweight concrete. This can be used in different structural applications to obtain required advantage in reduction of dead load and giving the best thermal insulation for concrete structure. The experimental work included various types of lightweight mortars were produced by using foaming agent as a percentage by weight of cement with natural sand. Mechanical properties of the products were studied. Wet and dry density, compressive strength, tensile strength, flexural strength, water absorption, volume change and thermal conductivity were investigated for all types of lightweight mortars. Results indicated that the wet and dry density of various types of mortar mixes decreased with increasing foaming agent content. The dry density for foamed mortars was ranged between (1780- 2066) kg/m^3. Also, compressive strength for all mixes decreases with increasing foaming agent. The compressive strength at 28 days age was ranged between (12.5- 29.24) MPa for foamed mortars. The results also show that the flexural strength, tensile strength and thermal conductivity decreases with increasing foaming agent. Drying shrinkage and swelling increases with increasing foaming agent.**

***Index Terms*—Light Weight Concrete, Foaming Agent, Compressive, Flexural, Tensile, Thermal Conductivity**

I. INTRODUCTION

THE cellular concrete was first developed in Stockholm, Sweden in the early 1900's. The original material was known as "gas concrete" to be used in producing heat-insulated building materials. This led to the development of related lightweight concrete which is now known as, foamed concrete. The applications were for floor, roof and wall units. Having low compressive strengths, it limited this product to fills and insulation only [1].

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[1] Professor, Building and Construction Engineering Department, University of Technology, Baghdad, Iraq. (corresponding author: e-mail: husn_irq@yahoo.com).

[2] Professor, Building and Construction Engineering Department-University of Technology. (e-mail: hish1950@yahoo.com).

[3] Ass. Lecturer, Technical Institute, Dhe Qar-Iraq.

Foamed concrete is an aerated mortar or cementitious matrix made by introducing air or gas into a prepared mortar or slurry. The bulk density of ordinary foamed concrete ranges between 320 to 480 kg/m^3 for insulating purposes, to between 1300 and 1600 kg/m^3 for structural purposes [2]. Aerated concrete is a light weight cellular material. It is unlike other concretes in as much as it does not normally contain aggregate and can be regarded as an aerated mortar [3].

There are several studies on the foaming concrete and light weight aggregate concrete. "Reference [4] showed that the factor of thermal conductivity depends on concrete mixture constituents and percent of humidity in it when temperature degree is changed."

"Reference [5] studied the properties of structural-grade foamed concrete using low-lime fly ash as a cement and filler. It is shown that structural grade foamed concrete is readily achievable at densities of around 1200 kg/m^3."

Few researchers studied the engineering properties at autoclaved cellular concrete block [6] [7]. They concluded that compressive strength was significantly affected by moisture content. While [8] studied synthetic lightweight aggregate produced by melt compounding high concentrations of high carbon fly ash into various thermoplastic binders to produce series of light weight aggregate for use in application such as lightweight concrete. More recently, an investigation was carried out by [9] to study the effect of crumb tires rubber on some properties of foamed concrete.

The aim of this study is to obtain foamed concrete by using different percentages of foaming agent and also studying the characteristics and behavior of this type of concrete when it is used as thermal insulation to reduce the required cost in the air-condition.

II. EXPERIMENTAL WORK

Materials

Cement

The cement used in this investigation was ordinary Portland cement manufactured in Lebanon. Its chemical and physical properties were found to satisfy the limits of Iraqi specification No. (5-1984).

Fine Aggregate

Natural sand brought from Al-Ekhadir region was used and its gradation lying in zone (2) with a maximum aggregate size of 4.75 mm. Table 1 shows chemical and physical properties of the used sand. The grading test results conform to Iraqi specification No. 45/1984 [10].

Table 1. Chemical and physical properties of sand.

Properties	Specification	Test Results	Limits of Specification
Specific gravity	ASTM C 128-88 [11]	2.63	-
Absorption (%)	ASTM C 128-88 [11]	0.75	-
Dry loose- unit weight (kg/m^3)	ASTM C 29-89 [12]	1590	-
Sulfate content as SO_3 (%)	I.S No.45/1984 [10]	0.09	≤ 0.5
Materials finer than 75 µm (%)	I.S No.45/1984 [10]	3.5	≤ 5.0

Water

Potable water was used throughout this study for mixing and curing.

Foaming Agent

Polyrex SP 32/1 is a specially formulated polyol, for the production of blow rigid polyurethane polymer. Reacting polyrex SP 32/1 with Isorex R310/1 renders foams with excellent flow ability and evenly distributed density and good adhesion to the substrates. Table 2 shows typical properties of the used foaming agent.

Table 2. Typical Properties of the used foaming agent.

Property	ISOREX R310/1	POLYREX SP32/1
NCO content	31%	-
Viscosity at 20 ^oC	300m pas	+/-20m pas
Viscosity at 25 ^oC	200m pas	
Density at 20 ^oC	1.23 g/ cm^3	1.17g/cm^3
Storage temperature	20-35 ^oC	18-20 ^oC
Storage ability	6 month	6 month

Mixes

Mortar mixes of different mix proportion (cement: sand; 1:2, and 1:3) were used. The percentage of foaming agents 0%, 3%, 5%, 7%, 10%, 12% and15% by weight of cement.

Moulds

The size of specimens was made within the standard requirement. The details of specimens were as shown:

1. Cubes of 50mm were used for compressive strength test.
2. A prism form with dimensions of 100×100×400mm was used to determine flexural strength, tensile strength and volume change (shrinkage and swelling).
3. Disc of diameter 40mm for determining thermal conductivity.
4. Cube of 100mm for measuring dry density and absorption.

Curing

After casting the moulds were covered with nylon and left in laboratory for about 24 hours, the specimens were then demoulded carefully and stored in water tanks until time of testing specimens.

III. Experimental Tests

Compressive Strength Test

Compression tests were carried out by using testing machine, with capacity of 2000 KN ASTM C- 109.

Flexural Strength Test

The ultimate flexural strength was determined by two-loading points method according to ASTM C-78. The average reading of three specimens was recorded.

Equivalent Tensile Strength Test

This test was made according to British standard 1881: part117:1983 by using the second part of prism which is tested to flexural strength.

Thermal Conductivity

The thermal conductivity is estimated by measuring of temperature value which is transported through unit area of sample through unit time, and the difference in temperature degree between sample faces is measured when the sample reaches thermal equilibrium state [13].

Water Absorption

The test was carried out according to ASTM C642-82 on 100 mm cube specimens.

Drying Shrinkage

This test was carried out in accordance with ASTM C490-89 using 100×100×400mm prism specimen. Demec points for a 200mm mechanical extensometer were fixed by using adhesive materials at selected positions on the sides of the specimens. The first (initial) readings for shrinkage and strains were taken at the age of 2 days for all specimens.

Swelling

It was measured by using the same apparatus of measuring shrinkage. The samples had prism (100×100×400mm) form, and they were soaked in water and measured each 24 hours.

IV. Results and Discussion

Water requirement for standard consistency

The relationship between foaming agent content and the amount of water required for standard consistency of mortars, expressed in terms of W/C ratio is shown in Table 3. The results indicated that the water requirement increases as the percentage of foaming agent increases. The percentage of increase in W/C ratio compared with reference mix for both mixes 1:3 and 1:2 by addition 0 to 15% foaming agent was 13% and 14% respectively. This is mainly due to the fact that high absorption of foaming agent reduces the consistency of mortar.

Table 3. Effect of foaming agent on the water/cement ratio at the same consistency.

Foaming agent ratio %	W/C ratio	
	Mix 1:3	Mix 1:2
0	0.7	0.51
3	0.71	0.53
5	0.72	0.54
7	0.74	0.55
10	0.76	0.56
12	0.78	0.57
15	0.79	0.58
0	0.7	0.51

Density

Generally from results in Table 4, it can be seen that the density of two mixes decreases with increasing of foaming agent content compared with reference mix. The percentage of reduction in dry density for foaming agent content 3,5,7,10,12 and 15% mix for mix 1:3 and 1:2 was ranged from (3.6- 8) % and (1.6-16) % respectively, while for wet density, the percentage of reduction was ranged from 2.2- 20 % and 15.3-33.2% respectively. This may be due to entrapping small cells of air in cementing matrix which in itself may be light in weight [2].

Table 4. Dry and wet density of mix 1:3 and 1:2.

Foaming agent %	Density (kg/m^3) Mix 1:3		Density (kg/m^3) Mix 1:2	
	dry	wet	dry	wet
0	1940	2100	2100	2650
3	1870	2066	2066	2245
5	1870	2060	2060	2200
7	1857	2000	2000	2190
10	1835	1907	1907	2182
12	1820	1856	1856	2170
15	1780	1760	1760	1770

Compressive Strength

The results of the effect of foaming agent content on the compressive strength at various ages are shown in Figs. 1 and 2. Generally, it can be seen for all mixes that the compressive strength of the specimens at all ages decreases gradually with increasing foaming agent content compared with the reference mix. The percentage of reduction in compressive strength at 180 days age of the specimens with foaming agent from 3% to 15% for mix 1:3 was ranged (10- 58)%, while for mix 1:2 was ranged (14- 62)%. "Reference [4] reported similar finding that the strength of cellular concrete can be expressed as a function of the void content taken as the sum of the induced voids and the volume of evaporable water."

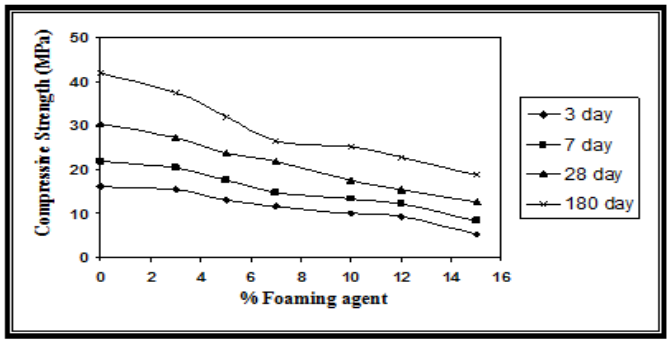

Fig. 1. Effect of foaming agent on the compressive strength of mortar 1:3.

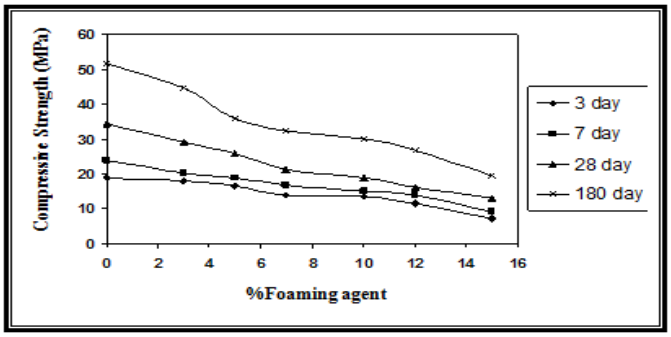

Fig. 2. Effect of foaming agent on the compressive strength of mortar 1:2.

Flexural strength

The flexural strength results of specimens are presented Figs. 3 and 4. Results demonstrate that the flexural strength decreases with the increase in foaming agent content compared with reference mix. The percentages of reduction for 1:3 mix mortar containing 3, 5, 7, 10, 12 and 15% foaming agent and at age 180 days were 3.3, 3.3, 13.3, 14.67, 15.5 and 26.67% respectively. While for mix 1:2, the percentage of reduction in flexural strength was 5.7, 18.6, 20, 22.7, 27.1 and 37.1% respectively. This may be attributed to the lower tensile strength due to the presence of air voids generated by foaming agent.

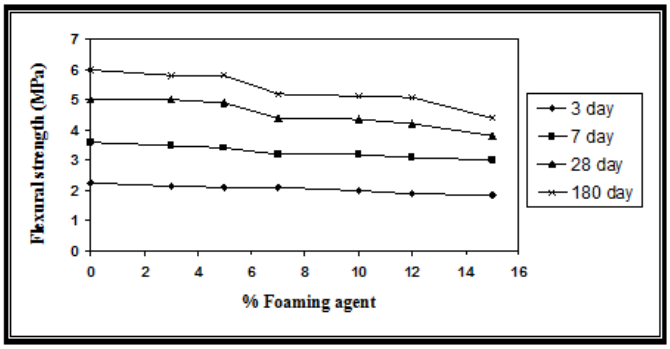

Fig. 3. Effect of foaming agent on the flexural strength of mix 1:3.

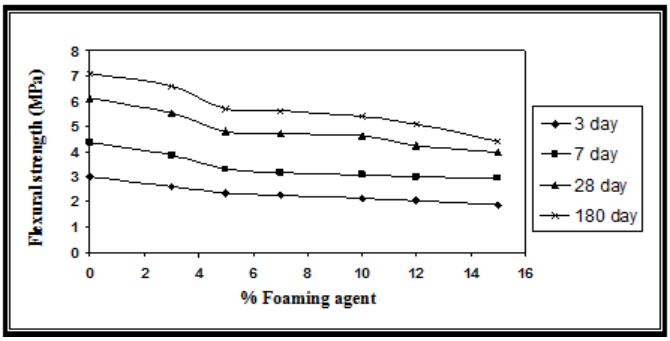

Fig. 4. Effect of foaming agent on the flexural strength of mix. 1:2.

Equivalent Tensile Strength

Figs. 5 and 6 show that the tensile strength decreases with increasing foaming agent when compared with reference mix. The percentage of decrease for specimens containing foaming agent by 3, 5, 7, 10, 12 and 15% for mix 1:3 at age 180 days it ranges from (5.5- 65.5%), while for 1:2 mortar mixes 180 days age it ranged from (12-47)%.

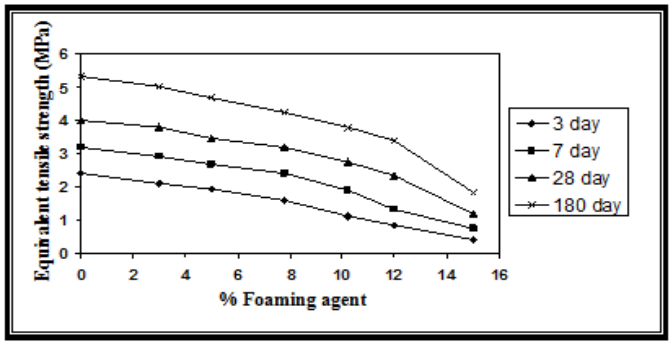

Fig. 5. Effect of foaming agent on the equivalent tensile strength of mortar 1:3.

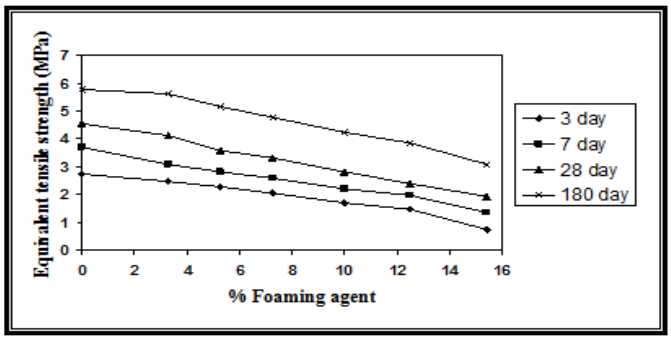

Fig. 6. Effect of foaming agent on the equivalent tensile strength of mortar 1:2.

Thermal Conductivity

The test results of the thermal conductivity of various types of mortar specimens are shown in Table 5. It is noticed that the thermal conductivity decreases with increasing foaming agent. It can be seen that for 1:3 mortar mix, the percentage of reduction in thermal conductivity of specimens with foaming agent by 3, 5, 7, 10, 12 and 15% is about 20, 25, 27, 30, 34 and 36% respectively. While for 1:2 mortar mix with foaming agent 3 and 7%, the percentage of reduction is 13 and 19% and for 5, 7, 10, 12 and 15% the values are between 21 to 33%. This behavior may be attributed to the air-filled pores, air being a very poor conductor of heat, the addition of water which has a conductivity of about 25 times of air, will increase the coefficient of thermal conductivity [14].

Table (5). Thermal conductivity and absorption of mortars containing foaming agent at 28 day.

Agent %	Thermal Conductivity W/(m. K^0)		Absorption %	
	Mix 1:3	Mix 1:2	Mix 1:3	Mix 1:2
0	0.638	0.679	3.50	2.14
3	0.511	0.593	3.80	2.80
5	0.480	0.551	4.0	3.31
7	0.463	0.536	4.60	3.90
10	0.447	0.502	6.22	5.10
12	0.423	0.476	7.36	6.75
15	0.406	0.456	10.90	9.94

Water Absorption

The results obtained from measurement of water absorption at 28 days are presented in Table 5. From the results, it can be observed that the percentage of the absorption of all specimens increases as foaming agent content increases when compared with reference mix. The percentage absorption ratio of 1:3 mortar mix containing 3, 5, 7% foaming agent to reference mix is very little, and the values are 1.8, 2.1 and 3.1 for 10, 12, 15% foaming agent respectively, while for 1:2 mortar mix was about 1.3, 1.5, 1.8, 2.4, 3.2 and 4.6 respectively. On the other hand, the value for two mixes ranges from 2.14-10.9% these results are in agreement with those of other studies for plain mortar, which indicates that the absorption for different mixes ranges from 3.2 to 12.3% [4].

Volume Change

Drying Shrinkage

Results of drying shrinkage of mortar specimens up to 180 days are presented in Figs. 7 and 8. These test results indicate that the drying shrinkage for specimens increases rapidly at early ages; this is mainly due to the rapid loss of moisture from the surface of the specimen. While at later ages, the rate of increase of drying shrinkage is reduced with time depending on the moisture movement of concrete.

The results show that as foaming agent increases the drying shrinkage increases. The percentage of increase in drying shrinkage at age of 180 days for specimens with foaming agent by 3, 5, 7, 10, 12 and 15% for mix 1:3 is 7.7, 13.8, 21.5, 29.2, 38.5 and 44.65% respectively, and for mix 1:2 it is 4.2, 8.3, 15.3, 22.2, 29.2 and 36% respectively. "Reference [15] reported similar finding that the lightweight aerated concrete has high drying shrinkage due to poorly crystallized tobermorite gel."

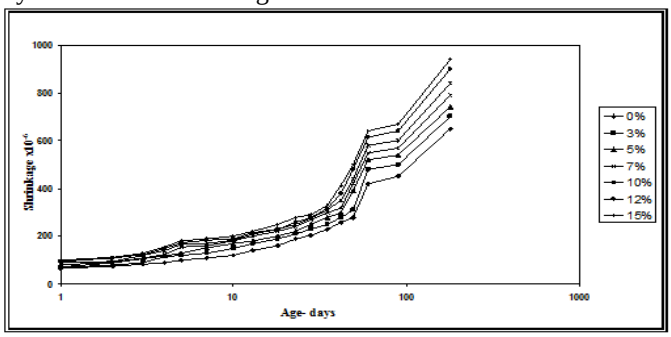

Fig. 7. Relationship between shrinkage and age for (1:3) mortar of different percent of foaming agent.

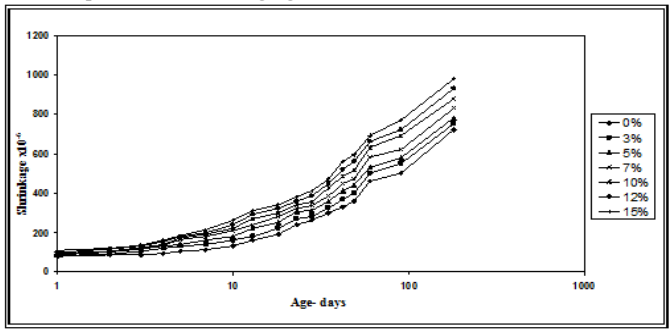

Fig. 8. Relationship between shrinkage and age for (1:2) mortar of different percent of foaming agent.

Swelling

When mortar specimens (prisms) are kept in water, an increase in volume, which is expressed as a linear swelling results. The effect of foaming agent on swelling of 1:3 and 1:2 mortars mix is shown in Figs. 9 and 10.

Generally, it can be seen that the swelling increases with increasing of foaming agent content. For specimens containing foaming agent by 3, 5, 7, 10, 12 and 15%, the percentage of increase in swelling at age 180 day for 1:3 mortar mix is 7, 18, 25, 32, 43 and 51% respectively. For mix 1:2 the percentage of increase is 4.7, 13.3, 21.3, 30.7, 38.7 and 48.7% respectively. This may be due to the physical nature of the foaming agent (polyurethane).

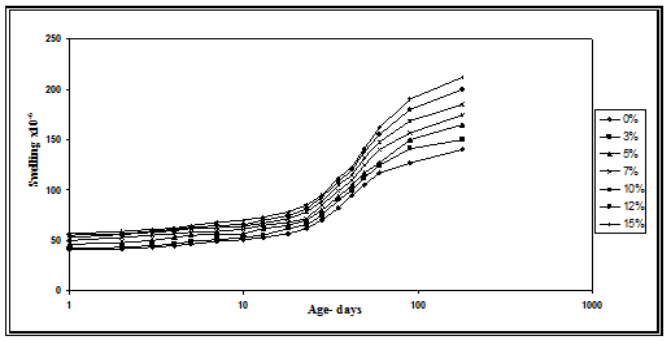

Fig. 9. Relationship between swelling and age for (1:3) mortar of different percent of foaming agent.

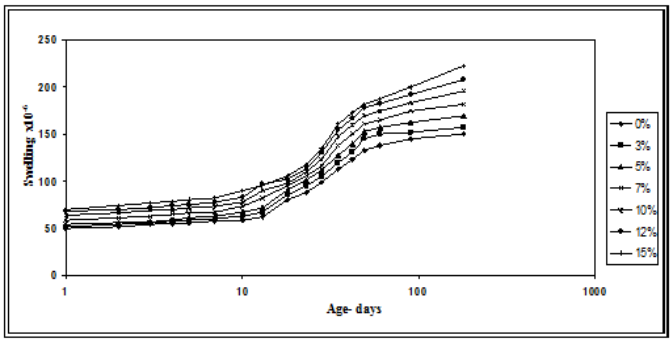

Fig. 10. Relationship between swelling and age for (1:2) mortar of different percent of foaming agent.

V. Conclusion

The major conclusions drawn from the test are summarized as follows:

1. Addition of foaming agent to the mortar reduces consistency and with increasing the percentage of foaming agent, the W/C ratio increases to give standard consistency.
2. At all ages of curing, compressive, flexural and tensile strengths decrease with increasing foaming agent content. The percentage of reduction in compressive, flexural and tensile strengths for 15% foaming agent content at 180 days age is 55.5, 27and 65.5% respectively for mix 1:3 compared with the reference mix. While the percentage of reduction for mix 1:2 with 15% foaming agent content at 180 days age is 62.7, 37 and 47% respectively compared with the reference mix.
3. The higher the percentage of foaming agent, the higher is the rate of shrinkage and swelling compared with those of the corresponding plain mixes and the percentage of increase in drying shrinkage at age of 180 days for 1:3 mix mortars is 44.7%, while the percentages of increase in swelling for 1:3 mortar mixes are 51% compared with those of the corresponding plain mortar.
4. Water absorption increases with the increasing foaming agent content. The percentage of absorption values for 1:3 and1:2 mortar mixes is 10.9% and 9.94% respectively.
5. Thermal conductivity reduces with decreasing density; the minimum coefficient of thermal conductivity is 0.406 W/m. K^{o} 0.456 for mix 1:3 and 1:2 respectively.

References

[1] Light concrete LLC, “Application,” 2003. info@escsi.org.www.escsi.org

[2] W. H. Taylor, “Concrete Technology and Practice,” McGraw Hill Book Company, 4th Edition, 1977.

[3] “Litbuilt Lightweight Aerated Concrete Products,” 2003. http://www.litebuilt.com

[4] A.M. Neville, “Properties of concrete,” Pearson-Prentice Hall, Fourth edition, 2005.

[5] R. Jones, A. Giannakou, and L. Nicol, “Properties of structural-grade foamed concrete using low-lime fly ash as a cement and filler,” http://www.mii.org/.

[6] F. Alani, and R. Ovanessian, “Structural design of thermostone walls,” *Journal of Building Research*, vol.5, No.1, pp.65-83, May 1986, Baghdad.

[7] Hu. Wenyi, D. Ronald, E. Luis, Ch. Kelly, and M. Latona, “Strength properties of autoclaved cellular concrete with high volume fly ash,” *Journal of Energy Engineerin*g, vol. 123, No. 2, August, 1997.

[8] R Malloy, CH. Swan, and M. Kashi, “High carbon fly ash / mixed thermoplastic aggregate for use in lightweight concrete,” www.sintef.no/static/BM/project/.

[9] A.A. Hilal, “Effect of crumb tires rubber on some properties of foamed concrete,” *Anbar Journal for Engineering Sciences*, vol. 4, No. 2, 2011, pp. 1-7.

[10] I.S., No.45/1984, “Aggregates of natural resources used in the concrete and construction,” the Central Organization for Standardization and Quality Control (C.O.S.Q.C.), Iraq.

[11] ASTM C 128-88, “Standard Test Method for Specific Gravity and Absorption of Fine Aggregate”, Annual Book of ASTM Standards, Vol. 04.02, 1989, pp. 68-71.

[12] ASTM C 29-89, “Standard Test Method for Unite Weight and Voids in Aggregate”, Annual Book of ASTM Standards, Vol.04.02, pp. 1-3.

[13] S.A. Al-Nassirallh, “Manufacturing Lightweight Aggregate Concrete from Local Raw Materials,” M.Sc. thesis, University of Technology, 2005.

[14] ACI 213R-87, “Guide for Structural Lightweight Aggregate Concrete,” ACI Manual of Concrete practice, part (1), Materials and general properties of Concrete, 2000.

[15] H.G. Midgley, “Sand–Lime Brick,” Building Research Station London, 1974.

Prof. Dr. Hussein Hameed Karim received the B.S. and M.S. degrees in Earth Science/ Geophysics from the University of Baghdad, in 1978 and 1982 respectively, and the Ph.D. degree in Engineering Geophysics from University of Baghdad in 1996, Iraq.

From 1983 to 1997 he was within the academic staff of University of Basrah and graduated from Asst. lecturer to Asst. Professor. Since 1997 he is within the academic staff of University of Technology. Since 2000, he has been a Professor with the Building & Construction Engineering Department/ University of Technology. He is the supervisor of more than 45 post graduate (M.Sc. and Ph.D) students, the author of 5 books, and one under preparation, more than 100 scientific researches, more than 100 scientific articles, and two patents. His research interests include Engineering Geophysics, Soil & Rock Mechanics, Site Investigation, and Geomatics Engineering.

Prof. Karim was chosen by the Committee of University of Technology as the 1st Professor in the University of Technology for the academic year (2010-2011). Prof. Karim's awards and honors include training session to Bordeaux University- France (4 months, 1984); Visiting Professor to University of Hodeidah University- Yemen (3 months, 2002); Visiting Professor (one month, 2010) to Malaya University- Kuala Lumpur, Malaysia; and 3 Professor Visits sponsored by DAAD (Germany), Bauhaus University- Weimar 2006, Rhur University – Bochum 2010 and 2014 (3-months for each).

Rehabilitation and Strengthening of Reinforced Self-Compacting Concrete Exposed to Elevated Temperature

A. A. Al-Attar, M. B. Adeen, A. S. Khazaal and A. S. Tause, *member, IEEE*

Abstract—**Exposure of reinforced concrete buildings to an accidental fire results in deterioration in concrete strength and to concrete cracking. In this study, samples of reinforced concrete slabs were tested to indicate the effect of rehabilitation using fiber carbon polymer armed retrieval tolerability, after exposure to direct flame temperature. Testing intervals of burning namely were (30, 60, and 120) minutes.**

The result of the repaired reinforced concrete slabs showed that exposed to direct fire temperature level (700)° C at the bottom surface of slabs for exposure duration of (30,60, and 120) minutes leads to decrease in strength by (20.58%, 32.4%,and 45.28%) respectively, compared with control slabs , and the slabs retrofitting using CFRP sheets restored flexural strength values by (23.07%, 32.8%, and 42.23%) for the burning durations (30,60,120) minutes respectively, compared with slabs were tested after burning without retrofitting.

***Index Terms*— Self-Compacting Concrete, Elevated Temperature, Fire Flume.**

I. Introduction

THE fire is one of the most prominent forms of exposure to high temperatures, it may cause accidents, natural disasters, energy, etc. When fire is jasper in a particular building, the flames of the fire will be extended and expanded to succeed the demolition and destruction, but the scale of this mass could depend on a number of factors, such as building design, building, fire, and evacuation methods and the structural performance of reinforced concrete, it is possible to be this successful performance during the fire, depending on two factors, namely the basic properties of building materials and their function in origin, the phenomenon of the fire and the cause of the high temperatures of the phenomena which should be given particular importance because of its impact on durability of structures [1].

When the concrete is exposed to high levels of heat, this will inevitably lead to the apparent quality of the concrete, the concrete in its mechanical properties, as a result of exposure to high temperatures, have a significant decrease occurs in strength, elastic modulus, rigidity, tensile strength of rebar in addition to the loss of bond with reinforcing steel bars [2]. The physical and chemical changes also occur as decomposition of calcium hydroxide to calcium, water, lime stretch through hydrolysis, the formative rule structure (Gel) [3].

II. Objective of Research

The objective of research is to describe the impact of the rehabilitation of self-compacting slabs using different carbon fiber polymer ribbons , after exposure to a flame of fire on the bottom surface of the slabs o with a temperature of 700 °C for different burning periods of (30, 60, and 120) minutes to evaluate structural behaviour after burning.

III. Experimental Program

A. Material Used

1. Cement

Ordinary Portland cement type I was used in all mixes throughout this research. It was stored in airtight plastic containers to avoid exposure to atmospheric conditions like humidity. The physical and chemical properties of this cement are presented in Table [1]. Test results indicate that the adopted cement conforms to the Iraqi specification No. 5/1984.

2. Fine Aggregate

Normal weight natural sand from Al-Tuz region east of Tikrit area was used as fine aggregate. The grading of the sand conforms to the requirements of Iraqi specification No. 45/1984, as shown in Table [2].

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A. A. Al_attar is Assist. Prof. in Technical college of Kirkuk, Kirkuk (corresponding author to provide phone: +9647701323897; e-mail: alattaralya@gmail.com).

M. B. Adeen is Assist. Prof. in Civil Engineering Department, Engineering College, University of Tikrit (Phone: +964772009977, e-mail: mazin_tikrit@yahoo.com).

A. S. Khazzal is Lecturer in Civil Engineering Department, Engineering College, University of Tikrit (Phone: +964775168043, e-mail: akhazaal@gmail.com).

A. S. Tause was with M.Sc. in Civil Engineering Department (structures), (Phone: +9647705158058, e-mailabdalla.saab90@yahoo.com).

TABLE 1.
CHEMICAL AND PHYSICAL PROPERTIES OF CEMENT

Oxide Composition	Content %	Limits of IQ Specification 5/1984.
Al2O3	2.97	8% upper limit
SiO2	17.3	21% upper limit
SO3	2.18	2.8% upper limit
L.O.I	1.02	4% upper limit
I.R	0.78	1.5 % upper limit
Physical Properties	Test Result	Limits of IQS # 5/1984.
Specific Surface Area	303	301.5 lower limit
Initial Setting Time (min.)	60	Not less than 45 min.
Final Setting Time (hr.)	8	Not more than 10 hrs.
Compressive Strength (MPa) at age of 3 days.	26.9	(15 MPa)

TABLE 2
GRADING OF FINE AGGREGATE

Sieve Size (mm)	Cumulative Passing %	Limits of IQS # 45/1984 (zone III)
4.75	100	90-100
2.36	92	85-100
1.18	83	75-100
0.6	62	60-79
0.3	23	12-40
0.15	5.25	0-10

3. Coarse Aggregate

Local naturally uncrushed gravel was used of nominal maximum size 12 mm as coarse aggregate. The grading of coarse aggregate conforms to the requirements of Iraqi Specifications No. 45/1984, as shown in Table [3].

TABLE 3.
GRADING OF COARSE AGGREGATE

Sieve Size (mm)	Cumulative Passing %	Limits of ASTM C33-01
12.5	100	90-100
9.5	92	85-100
4.75	17	10-30
2.36	0	0-10

4. Water

Ordinary tap water was used for mixing and curing all concrete mixes of this investigation.

5. Silica Fume

The Micro-Silica used in this investigation is Elkem Micro Silica MEYCO® MS610, a concrete additive of a new generation in powder form. The physical description, chemical analysis, and test results of the activity index of micro-silica are given in Tables [4].

6. High Range Water Reducing Admixture

(SikaViscoCrete-5930) is used in this research as a high range water reducing admixture (HAWRA) to enhance the early and final strength of concrete.

TABLE 4
ACTIVITY INDEX AND CHEMICAL PROPERTIES OF SILICA-FUME

Properties	Test Results
L.O.I	1.3
Activity Index	105
SiO_2	96.81
AL_2O_3	0.25
CaO	0.16
SO_3	0.14

7. Steel Reinforcing Bars

Reinforcing steel bars were used in current research are of 6mm in both two directions. Table 5 shows that the properties of the rebar used are conforms to ASTM A615/A615M [9].

Table 5
TEST RESULT OF TESTING REBAR

Steel Bar Diameter (mm)	Yield Stress (MPa)	Failure Stress (MPa)	Elongation (%)
6	619	725	7.3

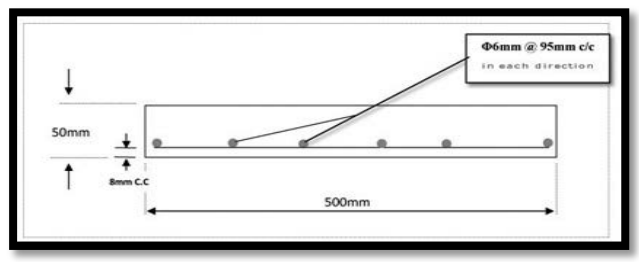


Fig. 1. Details of slab rebar

8. Epoxy

A substance used for bonding polymer fiber with concrete members are available in different types of different characteristics according to the manufacturers, in current study the type (SikaDur-330) was used. This type has a medium viscosity and consists of two parts: glue substance (Resin-A) of white colour, and hardener substance (Hardner-B) of bold gray colour, mixing ratio for this type of bond material is (1: 4) which are mixed (1kg) of hardened (B) with (4kg) of the glue (A) [10].

9. Polymeric Carbon Fibers (CFRP).

Carbon fiber type (Sika Wrap Hex-230c) has been used in strengthening of concrete slabs process. These fibers are used in strengthening of concrete structures and steel structures to increase the shear and bending strength.

B. Proportion of Concrete Mix

Design of self-compacting concrete mixtures requires high accuracy where it should has a high degree of flowability to fill the form and easily flow under its own weight without segregation, taking into consideration increasing cement paste content relatively compared to normal concrete, and that the characteristics of the self-compacting concrete ingredients are strongly affected by properties of materials so should take care in selecting components due to different materials available in the local market, mixtures have been designed according to requirements of European specifications (EFNARC) [12], in

terms of basic raw materials ratios for concrete (cement, coarse aggregate, fine aggregate, and admixtures), water powder ratio (W/p) was fixed for all mixes to match the requirements of the European standard (EFNARC) [12] and previous studies.

TABLE 6
INGREDIENTS OF CONCRETE MIX

	Ingredients kg/m3.			
w/p	Cement	Fine aggregate	Coarse aggregate	HRWRA %
0.34	450	900	750	3.3

C. Mixing, Casting, and Curing

The mixing process is important for flow-able and homogeneous mixture where the mixing of materials using of tilting mixer of capacity (0.07 m3), the required quantities of coarse aggregate (gravel) and fine aggregate (sand) were weighed and packaged in nylon bags to keep the moisture content at the time of mixing, samples were taken from the coarse and fine aggregates, weighed and dried in oven temperature (105°C) for 24 hours and then weighed it again, the moisture content is calculated to adjust the amount of water and materials in the laboratory is prepared and mixed by the following steps:

1) Coarse aggregate (gravel) and water were added to pot of mixer first.
2) Fine aggregate was mixed with filler (mixed cement and silica fume before mixing) and continue mixing for 1 minute.
3) Half the amount of water and Super-plasticizer were added and continue mixing for 2 minutes.
4) The remaining water was added and continue mixing for another three minutes. The moulds were cleaned and oiled to prevent adhesion of concrete, filled with concrete in one layer without using a vibrator, and covered to avoid water evaporation from the surface of the concrete with a layer of nylon for 48 hours, the moulds were then opened.

D. Rehabilitation and Strengthening

This work includes the study of using of outer packing fiber carbon release (CERP) with outer join to resist bending of RSCC (Reinforced Self-Compacting Concrete) slabs under the influence of static load, and its role in the rehabilitation of damaged concrete slabs after the burning to a target temperature of 700 °C for different burning periods (30, 60 and 120) minutes. The study includes as well investigating the more efficient and economical pattern for linking fiber ribbons, where, different patterns are used to linking fiber ribbons, and the more efficient one are chosen to use in strengthening.

E. Paking Using Carbon Fibers

The most important part of any application of rehabilitation applications using fiber carbon ribbons, is the bonding region between the fibers and the surface on which bonded it. To paste strips of fiber on concrete slabs, there are several steps to consider before packing:- after completing the curing of the slabs, they left to dry and prepare to be encapsulated with carbon fiber, the covering surface is cleaned using electrical instrument to become coarse and clean (for the burnt slabs it is necessary to clean the surface of reinforced concrete flame scorched and damaged and clean surface of fiber strips are pasted) then remove the dusts in the roughing by electric air compressor. Processing of carbon fiber strips according to the dimensions that to paste into the form. The bonding material (Epoxy Sika dur 330) is prepared , where the components (A) and (B) are mixed well materials in a container, with the drill machine with low speed with a mixing ratio 1:4 to obtain a homogenous mixture of light gray colour.

The bonding material (Epoxy) is placed on the concrete surface to be backed, then the carbon ribbons are placed above the bonding material, and it is pressed to ensure removing all the air voids that may arise between the fibers.

After this process, the outer surface of fibers are coated with another bonding layer. The (Epoxy) need for seven days (at least) for curing (Figure 2).

Fig. 2. Steps of Strengthen Concrete Slabs using Carbon Fiber Ribbons.

III. STRENGTHENING TECHNIQUE

This part of research including study of using different patterns of carbon fiber ribbons to determining the most efficient pattern, then use it later in strengthening the burned slabs.

Ten concrete slabs were strengthened in addition to the slabs that have been tested at the natural temperature degree under static loading were they are adopted as references specimens. The area of the used fibers was equal for each slab and was equal to (500 cm2), whereas the variable was the method of linking the fibers (ribbons patterns). Five patterns were used in this work as shown in Figure (3).

IV. FLEXURAL STRENGTH TEST

Flexural strength was tested for simply supported slabs of dimensions (500mm x 500mm x 50mm), Universal Testing Machine of capacity (1000kN) with a loading rate of (1.39 mm per minute) was used, each slab applied to point load by a

metal disc of diameter 50 mm, the readings (load-increase) were taken from the display linked with PC. Figure (4) shows the details of the method of allocation and loading of the slabs.

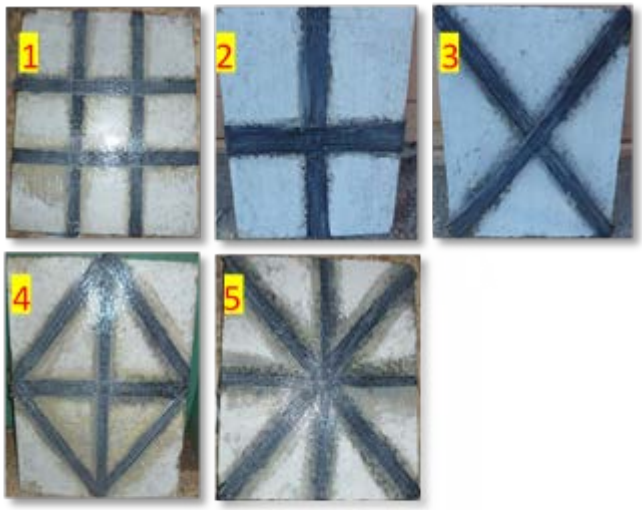


Fig. 3. Techniques of Strengthening that have used in this research SC1,SC2,SC3,SC4 and SC5 respectively .

Fig. 4. Details of Supporting and Loading Concrete Slabs.

V. Results and Discussion

A. Behavior under flexural Loading

It's can be noticed from the relationship between load and deflection for the tested beams under bending loads that the specimens have an elastic response at the first stages of the loading, then, with icreasing the loads, the first crack is happen and become obvious at the maximum moment region. When the loading is increasing, many cracks appear and expanduntil the failure is happen, which is by yielding of reinforcement steel followed by crushing of concrete. For the strengthened slabs, when the load is reach to the stage of steel yielding, the ribbons of carbon fibers begin to resisting the applied load and increase of the slab stiffness until failure which is by spliting of the carbon fiber from slab. Table (7) and Figure (5) shows the results that have been obtained from the flexural test for reference and strengthened slabs.

Table.7
Test Results of Strengthening Reference Concrete Slabs under Influence of Static load.

Slab	Average load at first crack (kN)	Failure load (kN)	% of increase of maximum load
SC	12.25	37.65	-
SC1	12.95	39.5	4.91
SC2	13.21	43.7	16.06
SC3	13.5	42.35	12.48
SC4	13.79	45.59	21.08
SC5	15.57	49.63	31.82

From the results, it can be notice that the slabs of specimen (SC5) gives highest failure load, and increased the failure load by (31.82 kN) compaing with the reference slabs; thus, the slabs of this strengthened pattern have been adopted to reinforcing the slabs later.

B. Burning of the Slabs

Data values are shown in the table (8) and Figure (6) on the slabs tested after burning periods (30, 60 and 120) minutes. It's noted that the values for the temperature (of fire flame direct) clear negative effect on strength of concrete slabs to load causing the failure, it's noted also that the highest rate of slab strength loss occurs in the first 30 minutes upon burning process, the burning periods of 30, 60, and 120 minutes led to a decrease in the values of the loads cause a failure by 20.58%, 32.4%, and 45.28% respectively compared with slabs of reference concrete, the reason behinds that is due to the negative influence of the thermal effects on concrete microstructure, Figure 7 shows the impact of a burning on maximum load carried by the slab.

Table 8
Test Results of Fired and Reference Concrete Slabs under Influence of Static Load

Slab	Period of applying Fire Flame (min.)	Max. Failure load (kN)	Decrease in max. Load with reference to reference concrete.
Ref.	-	37.65	-
F1	30	29.9	20.58
F2	60	25.45	32.4
F3	120	20.6	45.28

C. Rehabilitation after Burning

In order to study the effect of carbon fiber reinforced polymer (CFRP) on the response of concrete slabs that subjected to fire, a nine slabs have been subjected to fire flame from the bottom to a target temperature of (700 °C). The ribbon pattern of specimen (CS5) wich gives maximum failure load (the most efficient technique) as mentioned ealer is used here to strengthening and rehabilitation the the slabs after burining. The burining periods was (30, 60 and 120) minutes. Three slabs was used for every burining period; one of them has been tested sfter burning until failure, while the two others are strengthened with carbon fibers. The slabs the tested for flexural test under static loading. The results for ultimate failue load are shown in Table (9) and Figures (8-10). It can be

seen that the rehabilitation of slab led to increasing in strength of the slabs and increase of maximum failure load. The increasing in failure load was (23.07%, 32.8% and 42.23%) for burning periods (30, 60 and 120) minutes respectively.

TABLE 9
TEST RESULT OF REHABILITATED CONCRETE SLABS UNDER INFLUENCE OF STATIC LOAD

Slab	Burning period (min.)	Ultimate load after burning (kN)	Ultimate load after strengthening (kN)	% of the increase in ultimate load compared to slabs not strengthen
F1	30	29.9	36.8	23.07
F2	60	25.45	33.8	32.8
F3	120	20.6	29.3	42.23

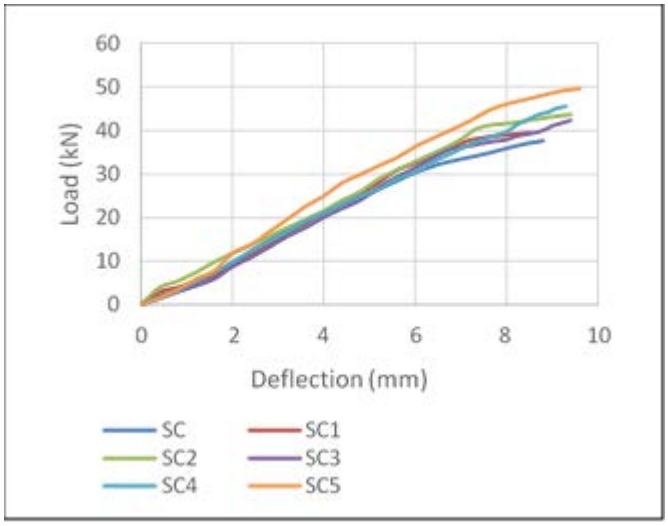


Fig. 5. Load-Deflection Relationship of Strengthen Concrete Slabs and Reference Concrete Slabs

Fig. 6. Relationship of load-deflection of fire concrete slabs for different fire periods compared to reference concrete slabs

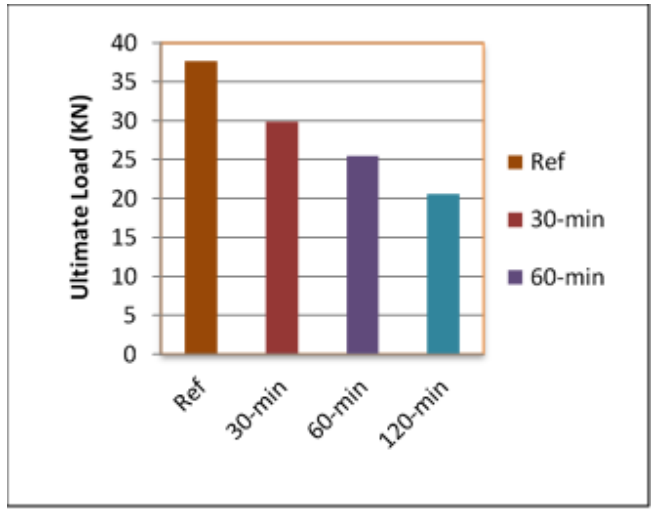


Fig. 7. The effect of the burning time period on the ultimate load compared to reference concrete slabs

Fig. 8. Relationship of load-deflection of fired concrete slabs and rehabilitated for burning period of 30 minutes compared to reference concrete slabs.

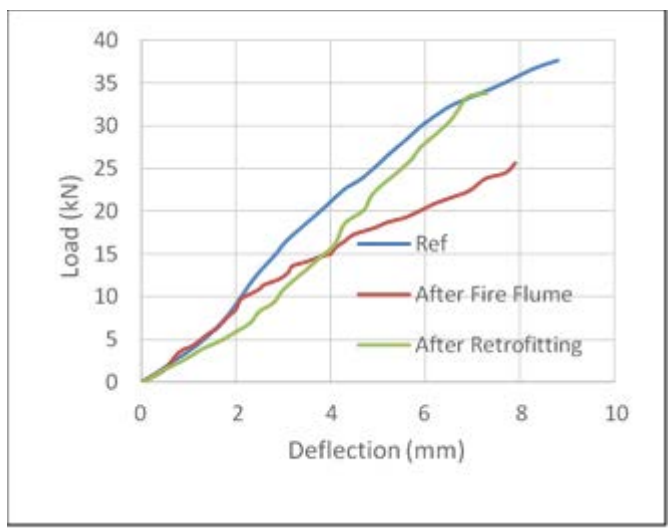


Fig. 9. Relationship of load-deflection of fire concrete slabs and rehabilitated for a burning period of 60 minutes compared to reference concrete slabs

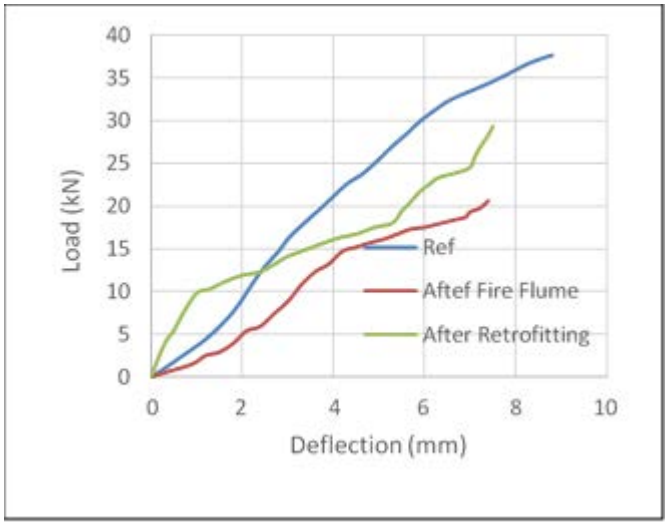


Fig. 10. Relationship of load-deflection of fire concrete slabs and rehabilitated for a burning period of 120 minutes compared to reference concrete slabs

VI. Conclusion

The heat (of fire, flame direct) impact resistant clear concrete slabs of load causing the failure, which led to a decrease in the strength of the slab to loads, it's noted that the highest rate of plate strength loss occurs in the first 30 minutes of the burning, the burning periods of 30, 60, and 120 minutes led to a decrease in the values of loads cause a failure by 45.28%, 32.4%, and 20.58% respectively, compared with the reference slabs. Rehabilitation after burning using carbon fiber ribbons was increased the value of peak load causing the failure rate by 42.23%, 32.8%, and 23.07% compared with the values of the slabs that were tested under the influence of static load directly after the burn.

References

[1] A. H. Ammar , "Effect of fire, flame (High temperature) on the self-compacted concrete (scc)one way slabs", M.Sc. Thesis, Dept. of Building and Construction Eng., Univ. of Anbar, Iraq 2012.

[2] I. Janotka, and T. Nürnbergerova, "Effect of Temperature on Structural Quality of High-Strength Concrete with Silica Fume", Institute of Construction and Architecture, Slovak Academy of Sciences, Bratislava, Slovak Republic, pp. 1 – 8, August, 2003.

[3] R. Ravindrarajah, R. Lopez and H. Reslan, "Effect of Elevated Temperature on the Properties of High Strength Concrete Containing Cement Supplementary Materials", Center of Built Infrastructure Research, University of Technology, Sydney, Australia, pp. 1 – 9, March, 2002.

[4] Properties of Portland Cement Iraqi Standard No. 5, 1984.

[5] Aggregate of Natural Resources used for Concrete and Construction Iraqi Standard, No. 45, 1984.

[6] Standard specification for concrete Aggregates, ASTM C 33-01.

[7] A. Khader, "Evaluation of the Constructability & Performance of Micro-Silica Modified Concrete Bridge Deck Overlays" Wisconsin Department of Transportation, Wisconsin, No.10, December, 2003.

[8] Standard Specification for Use of Silica Fume as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar and Grout, ASTM C1240-03.

[9] Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement, ASTM A615/A615M -09.

[10] Structural Strengthening with Sika Wrap Fabric System", Sika, pp.333, www.Sikaneareast.com

[11] Structural Strengthening with Sika Wrap Fabric System", Sika, pp.332, www.Sikaneareast.com

[12] The European Guidelines for self-compacted concrete specification, production and use, EFNARC, 2005.

Alyaa Abbas Al-Attar: was born in Kirkuk, Iraq, in 1972. She received the B.S in degree in civil Engineering from Salahaddin University in 1993 and the M.SC. in the civil Engineering (Structure) from Tikrit university in 1998 and the Ph.D in the construction and building material Department from University of Technology in 2006.

Mazin Burhan Adeen: was born in Kirkuk, Iraq, in 1973. he received the B. S in degree in civil Engineering from Tikrit University in 1995 and the M.SC. in the civil Engineering (Structure) from Tikrit University in 1998 and the Ph.D in the construction and building l Department from University of Technology in 2007. At this time, he is Assistance Professor at civil engineering in Tikrit University.

Ammar S. Khazaal: was born in Tikrit, Iraq, in 1975. He received the B.S. degree in Civil Engineering from Tikrit University in 1997, M.Sc in Civil Engineering (Geotechnical) from Tikrit University in 2000, and Ph.D degree in Civil Engineering (Building Materials) in 2014.

He participated in 7th Asia PacificYoung Researches and Graduates Symposium, Kuala Lampur, Malaysia in 2014. He published many papers about self-compacting concrete properties. At this time, he is lecturer in Civil Engineering Department, Tikrit University.

Abdullah Saab Tause: was born in Baghdad, Iraq, in 1990. He received the B. S in degree in civil Engineering from Tikrit University in 2012 and the M.SC. in the civil Engineering (Structure) from Tikrit University in 2015

Effect of Internal Curing on Resistance of High Performance Concrete to Internal Sulfate Attack

Muthana A. Saady, Shatha S. Hassan and Tareq S. al-Attar

Abstract— **Internal curing, IC, refers to the process by which the hydration of cement occurs because of the availability of additional internal water that is not part of the mixing water. The additional internal water is typically supplied by using relatively small amounts of saturated particles in concrete.**

In the present study the effect of internal curing on resistance of high performance concrete, HPC, to internal sulfate attack, ISA, is investigated. Two saturated curing agents, Limestone dust and Porcelanite filler, were used to facilitate internal curing for concrete. These agents were used as partial replacement of fine aggregate (sand) in two volumetric percentages (20 and 30 %). Three percentages of SO_3 in fine aggregate were adopted (1, 2, 3 %) by adding natural gypsum to the fine aggregate. The testing program included compressive and splitting tensile strength tests and was extended till the age of 240 days.

The test results showed that internal curing has a negative effect in limiting the rate and magnitude of destructive reaction of sulfates.

Index Terms— **Compressive strength, Internal curing, Limestone, Porcelanite, Sulfate attack**

I. Introduction

Internal curing, IC, refers to the process by which the hydration of cement occurs because of the availability of additional internal water that is not part of the mixing water. The additional internal water is typically supplied by using relatively small amounts of saturated particles in concrete. IC would many beneficial improvements for concrete; such as: reduces autogenous shrinkage and cracking, reduces self-desiccation, increases hydration cement, increases strength from the first 24 hours and beyond and finally improves durability [1]. Moreover, internally cured concrete has improved contact zone between aggregate and cement matrix and shows better thermal properties [2].

Internal sulfate attack, ISA, results from the reaction between sulfate in concrete ingredients (water, cement and aggregate) and cement paste which has calcium aluminates and water to form calcium sulphoaluminate. The hazard is caused by the formation of calcium sulphoaluminate which cause high tensile stresses that lead to expansion and disruption of concrete. Most of the research works done on ISA were concentrated on normal concrete, but very limited or rarely amount of published literature are available on high performance concrete, HPC. Lerch [3] was perhaps, the first to define the optimum gypsum content, OGC, of cement. It was defined as that giving the highest compressive strength, the lowest drying shrinkage and little expansion in water. Alternatively, Lerch defined it as that just enough to prevent the appearance of third peak in the curve of the rate of heat generation versus time after mixing cement with water.

When the gypsum content in cement exceeds the optimum, this would mark the beginning of ISA (compressive strength decreases and expansion in water increases). Lerch reported that the optimum gypsum content of cement increases with increased C_3A content, alkali content and fineness of cement. Al-Rawi [4] confirmed the above statement and showed experimentally that increased curing temperature increases the OGC, therefore, for accelerated cured concrete; the optimum gypsum content is considerably higher than that for normally cured concrete. Gypsum, $CaSO_4.2H_2O$, was considered dangerous for this type of attack because of the addition of gypsum to the cement at the grinding stage to control of the hydration speed and setting of cement paste. Al-Rawi [5] pointed out that the major cause of failure of concrete structures in the Middle East was the contamination of fine aggregate with sulfates in the form of gypsum. The research pointed to the gypsum is normally added to cement to retard early hydration and prevent quick set. The total sulfate in concrete may, therefore, be high enough to cause ISA. This could lead to deterioration and possibly cracking and failure of concrete structures. To avoid the adverse effects of sulfates, several specifications put an upper limit on sulfate content in aggregates or on total sulfates in concrete.

II. Experimental Work

A. Materials

Ordinary and sulfate resisting Portland cements were used throughout this study. They were conforming to the ASTM C150, Type I and V. Natural sand was used as fine aggregate. The fineness modulus, specific gravity (SSD) and sulfate content of this sand were 3.19, 2.6 and 0.1 percent respectively. Crushed gravel with a maximum size of 19 mm was used as coarse aggregate. The unit weight, specific gravity (SSD) and sulfate content of this gravel were 1635 kg/m^3, 2.7 and 0.01 percent respectively. Silicafume, with 10 percent replacement by weight of cement, was used as supplementary cementitious material. The used Silicafume has a specific surface of 20m^2/g. Two saturated curing agents, Limestone

dust, Ld, and Porcelanite filler, P, were used to facilitate internal curing for concrete. These agents were used as partial replacement of fine aggregate in two volumetric percentages, 20 and 30 percent. Absorption values for these agents were, 23 and 33 percent respectively. Table 1 shows the chemical compositions of cements, Silicafume and internal curing agents.

TABLE 1
CHEMICAL COMPOSITION OF CEMENTS, SILICAFUME AND INTERNAL CURING AGENTS

No.	Property		OPC	SRPC	Silicafume	Limestone dust	Porcelanite
1	Oxide Content, %.	CaO	58.98	61.20	1.22	60.01	11.55
		SiO_2	19.74	20.91	90.65	1.22	62.02
		Al_2O_3	3.72	3.60	0.02	0.61	2.70
		Fe_2O_3	3.72	4.60	0.01	0.20	0.87
		SO_3	2.73	2.30	0.24	<0.07	0.30
		MgO	3.78	3.28	0.01	0.32	7.20
2	LOI		1.10	3.46	2.86	36.50	13.86
3	LSF		0.92	0.90	-	-	-
4	IR		0.74	0.96	-	-	-
5	Compound Composition (Bouge's Equations), %.	C_3S	52.26	52.84	-	-	-
		C_2S	17.17	20.17	-	-	-
		C_3A	3.87	1.77	-	-	-
		C_4AF	10.77	13.98	-	-	-

TABLE 2
DETAILS OF THE TESTED MIXES FOR SULFATE EXPOSURE

Series	Mix	Cement type	Internal curing materials content, % by volume of sand	Sulfate content in sand, % by weight
1	SM_0 Sn	SRPC	---	0.1
	SM_0 S1	SRPC	---	1
	SM_0 S2	SRPC	---	2
	SM_0 S3	SRPC	---	3
2	SM_{20L} Sn	SRPC	20 Ld	0.1
	SM_{20L} S1	SRPC	20 Ld	1
	SM_{20L} S2	SRPC	20 Ld	2
	SM_{20L} S3	SRPC	20 Ld	3
3	SM_{30L} Sn	SRPC	30 Ld	0.1
	SM_{30L} S1	SRPC	30 Ld	1
	SM_{30L} S2	SRPC	30 Ld	2
	SM_{30L} S3	SRPC	30 Ld	3
4	SM_{20P} Sn	SRPC	20 P	0.1
	SM_{20P} S1	SRPC	20 P	1
	SM_{20P} S2	SRPC	20 P	2
	SM_{20P} S3	SRPC	20 P	3
5	SM_{30P} Sn	SRPC	30 P	0.1
	SM_{30P} S1	SRPC	30 P	1
	SM_{30P} S2	SRPC	30 P	2
	SM_{30P} S3	SRPC	30 P	3
6	OM_0 S3	OPC	---	3
	OM_{30L} S3	OPC	30 Ld	3
	OM_{30P} S3	OPC	30 P	3

B. *Concrete Mix*

The used concrete mix has the proportions of 1: 1.4: 1.9: 0.25 (CM: FA: CA: W/CM) by weight. The cementitious materials content of this mix was 500 kg/m^3. This mix was designed to attain an average compressive strength of 60 MPa at 28 days. This mix was adopted to produce of 6 series (23 batches) of specimens with the details shown in Table 2.

C. *Curing of Specimens*

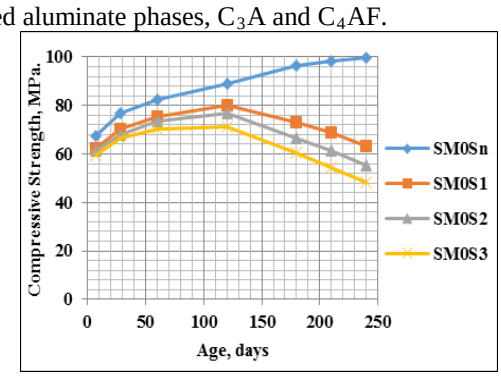
After the demolding of specimens, there were two methods of curing to facilitate the effect of internal curing on concrete behavior. In the first, the specimens were completely immersed in tap water until the time of testing (water-cured specimens). The curing water was replaced at least once a month. The second method of curing was to put specimens in plastic bags and prevent any water to penetrate the bags and to make the internal moisture is the only source of water for hydration to proceed (sealed specimens) – for more details sees reference 6.

D. *Internal Sulfate Exposure*

Natural gypsum was added to sand as partial replacement by weight to simulate 1, 2 and 3 percent SO_3 content of sand (see reference 7). When chemically analyzed, the used gypsum was found to consist of 44.5 percent SO_3. In the present work, the notations Sn, S1, S2 and S3 will indicate the 0.1, 1, 2 and 3 percent SO_3 content of sand respectively.

III. Results and Discussion

Table 3 shows the compressive strength development for the tested concrete mixes of high performance concrete, which contain different sulfate percentages in fine aggregate, for water-cured and sealed specimens respectively. In general, all mixes in all series showed reductions in compressive strength with respect to the reference mix, SM_0Sn. That behavior is mainly due to the destructive reaction of sulfates with the hydrated aluminate phases, C_3A and C_4AF.



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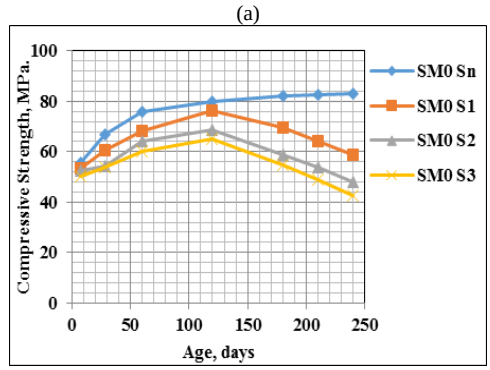


(b)

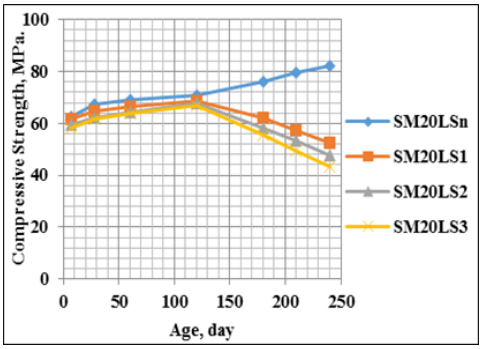
Fig.1. Compressive strength development for mixes in Series 1 (a) for water-cured specimens, (b) for sealed specimens

Figure 1displays the development of strength for mixes in Series 1. In this series no internal curing agent was used. As

shown in Figure 1, there is a reduction in compressive strength at all ages and it starts as early as the age of 7 days.

In addition to that, two opposite reactions could be assumed to start in the microstructure of concrete almost at the same time. The first is the building of the skeleton of the paste due to hydration of cement and the supplementary cementitious materials. This reaction has a positive effect on the development of compressive strength. The second reaction is that of internal sulfates with the aluminate phases (C_3A and C_4AF) to form expansive products (calcium sulfoaluminate and calcium aluminoferret). The production of these expansive compounds will inversely affect the microstructure of the paste and cause reduction in strength. The development of strength with age, whether positive or negative, will depend on the net summation of the two aforementioned reactions.

In the present work, it seems that the age of 120 days represents the inflection point at which the second reaction (reduction) dominates the behavior of concrete.



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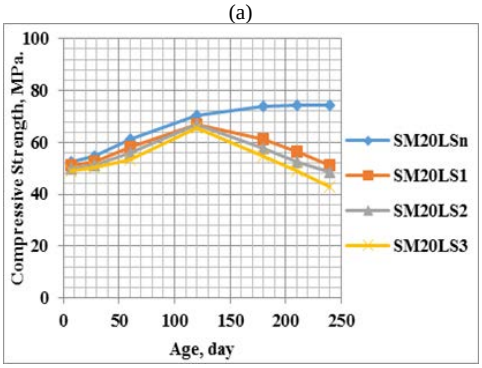


(b)

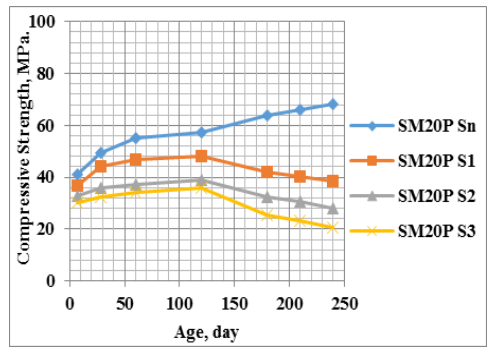
Fig.2. Compressive strength development for mixes in Series 2 (a) for water-cured specimens, (b) for sealed specimens

Figure 2 represents the development of compressive strength for mixes in Series 2. The results in this series summarize the effect of sulfate variation on the strength of HPC mixes contain Limestone dust as internal curing agent. This agent was used as a replacement for fine aggregate by 20 percent.

According to Table 3 and Figure 2, it could be concluded that the inclusion of Limestone dust in concrete has two distinct effects on strength development. For the age of less than 120 days, the sealed specimens showed higher reduction in strength than the water-cured specimens. This could be attributed to the high rate of sulfate reaction with aluminates due to the availability of water from the aggregate. The external curing water has no role in this reaction due to the low permeability of concrete. Also, if taking into consideration that the sealed specimens have lowers compressive and tensile strengths than the water-cured ones at the same age. The situation is reversed after the age of 120 days where the water-cured specimens show higher reduction in strength than the sealed specimens at the same age. At that time, the internal water seems to be consumed therefore decreasing the sulfate reaction rate, meanwhile the external water will have some role due to the long period of immersion.

Increasing the Limestone dust percentage to 30 percent (Series 3) does not affect much the previously described behavior of HPC against internal sulfate attack.

Series 4 and 5 were cast to visualize the effect of using lightweight Porcelanite as internal curing agent on the resistance of HPC to ISA. Two percentages were investigated, 20 and 30 percent, as partial replacement of fine aggregate.



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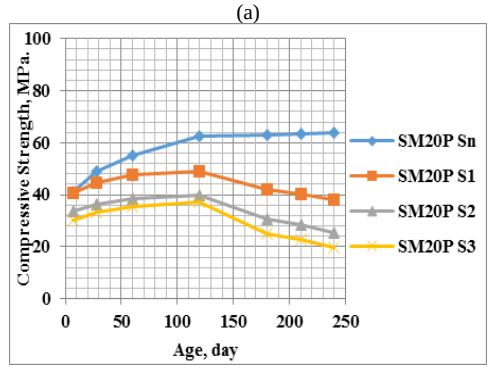


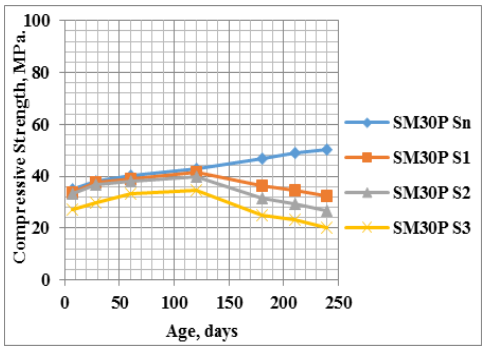
(b)

Fig.3. Compressive strength development for mixes in Series 4 (a) for water-cured specimens, (b) for sealed specimens

Figures 3 and 4 show the compressive strength development for the mixes in these series for both water-cured and sealed specimens. The observed decrease in strength with the incorporation of lightweight Porcelanite is mainly due to that this material is not as strong as the normal aggregate (fine aggregate) or as Limestone, especially if one keeps in mind the replacement percentages (20 and 30 percent) which is considered relatively high.

In Figure 3 and for Series 4, the sealed specimens showed higher reduction values than the water-cured specimens for the same age. The reductions for the sealed specimens showed almost a constant rate with time, meanwhile this stability in rate was not observed foe water-cured specimens. This could be attributed to the high rate of sulfate reaction with aluminates due to the availability of water from the

Porcelanite. The external curing water has no role in this reaction due to the low permeability of concrete.



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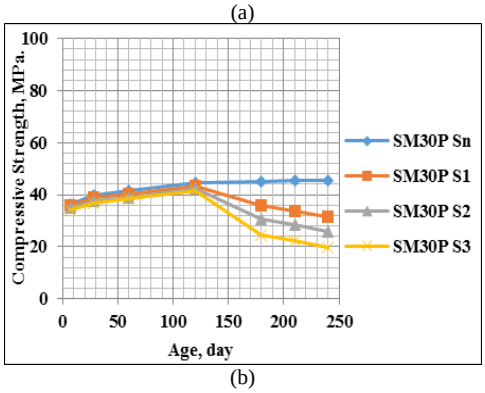


(b)

Fig.4. Compressive strength development for mixes in Series 5 (a) for water-cured specimens, (b) for sealed specimens

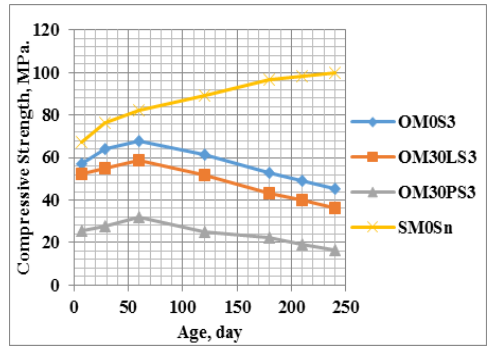
With respect to the type of internal curing agent (comparisons of Series 2, 3, 4 and 5), Limestone dust always showed lower reduction in strength values than Porcelanite for the same age. Two factors may be considered in this respect. The first is the water absorption of the agent. Higher water absorption would accelerate the sulfate harmful reaction. The second factor is the chemical reaction between Limestone and cement paste. Many research works [8] showed that this reaction would improve the interfacial zone and increase the bond between aggregate and paste. Such bond improvement increases the resistance to the damage caused by sulfate reaction. Porcelanite mixes lack this bond improvement.

Series 6 was produced to study the effect of type of cement as an additional factor. Figure 5 displays the compressive strength development for mixes OM for water-cured and sealed specimens respectively. These mixes were produced with ordinary Portland cement (ASTM C150, Type I). The reduction in strength starts at the age of 60 days. This could be resulted from the higher C_3A content of used cement which permits earlier destructive reaction with sulfates.

It is worthy to note that concretes made with both types of cement, sulfate resisting and ordinary Portland cement (Type V and Type I - ASTM C150), have suffered of degradation in compressive strength. Using Type V cement would not protect concrete against internal sulfate attack. This behavior could be explained by:

a. The reaction of aluminates (C_3A and C_4AF) with gypsum and water takes many months to accomplish [9]. Moreover, in rich mixes (high performance concrete), even small ratios of C_3A represent considerable amounts for this reaction to take place and continue.

b. Finer cement grains yield more active C_3A and lead to a higher optimum gypsum content [10]. The fineness of the used Type V cement in the present work was 4500 cm^2/g (Blaine method) which is considered at the upper limit for similar cements.



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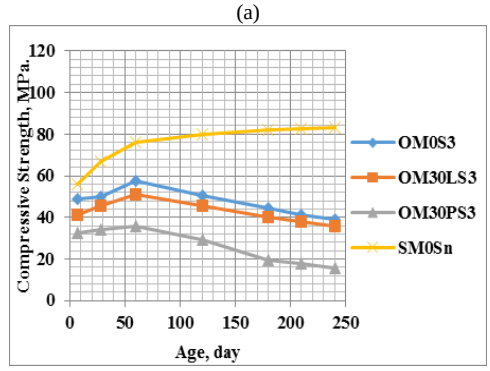

(b)

Fig.5. Compressive strength development for mixes in Series 6 (a) for water-cured specimens, (b) for sealed specimens

IV. Conclusions

The following are the conclusions reached throughout this research work:

1. The internal sulfate attack starts early and its effect is obvious even at the age of 7 days. In addition to that, two opposite reactions could be assumed to start in the microstructure of concrete almost at the same time. The first is the building of the skeleton of the paste due to hydration of cement and the supplementary cementitious materials. This reaction has a positive effect on the development of strength. The second reaction is that of internal sulfates with the aluminate phases (C_3A and C_4AF) to form expansive products (calcium sulfoaluminate and calcium sulfoaluminoferret). The production of these expansive compounds will inversely affect the microstructure of the paste and cause reduction in strength. The development of strength with age, whether positive or negative, will depend on the net summation of the two aforementioned reactions. In the present work, it seems that there is an age represents the inflection point at which the second reaction (reduction) dominates the behavior of concrete. This point of inflection can be specified by the

age of 60-day for mixes made with Type I cement (OPC) and by the age of 120-day for mixes made with Type V cement (SRPC).

2. Irrespective of being water-cured or sealed, the specimens have suffered degradation in compressive strength and this degradation is positively connected to the percentage of sulfate in fine aggregate.
3. The inclusion of internal curing agents in concrete has two distinct effects on strength development. For the age of less than 120 days, the sealed specimens showed higher reduction in strength than the water-cured specimens. This could be attributed to the high rate of sulfate reaction with aluminates due to the availability of water from the aggregate. The external curing water has no role in this reaction due to the low permeability of concrete. Also, if taking into considerations that the sealed specimens have lower compressive strength than the water-cured ones at the same age. The situation is reversed after the age of 120 days where the water-cured specimens show higher reduction in strength than the sealed specimens at the same age. At that time the internal water seems to be consumed therefore decreasing the sulfate reaction rate, meanwhile the external water will have some role due to the long period of immersion.
4. With respect to the type of internal curing agent, Limestone dust always showed lower reduction in strength values than Porcelanite for the same age. Two factors may be considered in this respect. The first is the water absorption of the agent (23 and 33 percent for Limestone dust and Porcelanite respectively). Higher water absorption would accelerate the sulfate harmful reaction. The second factor is the chemical reaction between Limestone and cement paste. Many research works showed that this reaction would improve the interfacial zone and increase the bond between aggregate and paste. Such bond improvement increases the resistance to the damage caused by sulfate reaction. Porcelanite mixes lack this bond improvement.
5. Concretes made with both types of used cement, sulfate resisting and ordinary Portland cement (Type V and Type I - ASTM C150), have suffered of degradation in strength. Using Type V cement would not protect concrete against internal sulfate attack.

REFERENCES

[1] B. Wolfe, R. Acampora and J. Kowalewski, “Sustainable Concrete through Internal Curing” Connecticut Ready Mixed Concrete Association, Wethersfield, USA, 2010. http://ctconstruction.org.

[2] T. R. Naik and F. Canpolat, "Self-curing Concrete", CBU Report, April 2006, pp. 11.

[3] W. Lerch, "The influence of gypsum on properties of concrete", ASTM proceedings, V. 46 1946, pp. 1252-1292.

[4] R. S. Al - Rawi, “Gypsum Content of Cement Used in Concrete Cured by Accelerated Methods” ASTM Journal of Testing and Evaluation, V. 1.5 N.3 May 1977, pp. 231 -237.

[5] R. S. Al-Rawi, "Internal Sulfate Attack in Concrete Related to Gypsum Content of Cement with Pozzolan Addition" ACI-RILEM Joint Symposium, Monterey, Mexico, 1985, pp. 543- 556.

[6] M. A. Saady, “Effect of Internal Curing on Sustainability of High Performance Concrete” MSc Thesis, University of Technology, 2015, Iraq.

[7] T. K. Al – Kadimi and F. Abood, "Effect of Gypsum Present in Sand on the Properties of Concrete" Building Research Center Journal, Nov. 1983, PP.17-41, Iraq.

[8] K. M. Alexander, “Strength of the Cement –Aggregate Bond”, American Concrete Institute Journal, V.56, 1959, pp. 377-390.

[9] F. M. Lea, *Chemistry of Cement and Concrete*, 4th Edition, Elsevier, USA, 2004.

[10] A. M. Neville, *Properties of Concrete*, 5th Edition, Longman Group, UK, 2010.

TABLE 3

COMPRESSIVE STRENGTH DEVELOPMENT FOR MIXES CONTAIN DIFFERENT PERCENTAGES OF SULFATE IN FINE AGGREGATE (WATER-CURED AND SEALED SPECIMENS)

Mix	Compressive Strength of Concrete Mixes, MPa, at Ages (in days):													
	for water-cured specimens							for sealed specimens						
	7	28	60	120	180	210	240	7	28	60	120	180	210	240
SM_0S_n	67.3	76.6	82.2	89.2	96.6	98.4	99.8	55.7	67.0	76.1	80.1	82.0	82.5	83.1
SM_0S_1	62.2	70.1	75.5	80.1	73.3	68.8	63.5	53.2	60.4	68.3	76.2	69.5	64.4	58.8
SM_0S_2	61.5	68.1	73.7	76.9	66.7	61.3	55.1	52.3	54.1	64.1	68.8	58.9	53.7	47.9
SM_0S_3	59.6	66.7	70.3	71.4	60.4	54.6	48.2	50.2	51	60.2	65.3	54.7	48.9	42.5
$SM_{20L}S_n$	62.5	67.4	69.1	71.0	76.1	79.5	82.3	52.3	54.9	61.2	70.3	74.0	74.2	74.5
$SM_{20L}S_1$	61.1	64.7	66.4	68.5	62.2	57.4	52.5	51.2	52.6	58.3	67.1	61.3	56.5	51.4
$SM_{20L}S_2$	59.7	62.1	64.4	67.7	58.2	53.5	47.8	50.1	51	55.8	66.7	57.6	52.5	48.6
$SM_{20L}S_3$	58	61.8	63.7	66.8	55.5	49.6	43.5	49	50.5	53.4	65.4	54.8	48.9	42.7
$SM_{30L}S_n$	60.1	64.7	67.7	70.1	75.2	77.9	80.1	50.0	52.1	60.1	69.6	73.1	73.5	73.7
$SM_{30L}S_1$	59	62.1	65.5	67.5	61.4	56.7	51.8	48.7	50.5	56.9	66.5	60.3	55.6	50.5
$SM_{30L}S_2$	57.7	60.3	63.3	65.9	56.7	51.8	45.5	46.3	48.9	52.1	63.1	54.8	49.8	42.7
$SM_{30L}S_3$	54.9	57.9	60.1	64	52.9	46.8	40.7	44.1	47.5	51.4	62.1	51	45.3	39.9
$SM_{20P}S_n$	41.2	49.3	55	57.3	64.0	66.2	68.1	41.0	48.8	55.1	62.4	63.1	63.4	63.7
$SM_{20P}S_1$	36.6	44	46.7	48.1	42.2	40.4	38.5	40.5	44.5	47.8	49.2	42	40.2	38.1
$SM_{20P}S_2$	32.9	36.1	37.2	38.9	32.6	34.8	27.9	33.6	36.5	38.5	39.9	30.5	28.6	25.5
$SM_{20P}S_3$	30.1	32.6	34	35.7	25.4	23.3	20.7	30.4	33.1	35.5	37	24.9	22.7	19.8
$SM_{30P}S_n$	34.9	38.1	40.1	42.9	47.0	49.1	50.2	36.2	39.7	41.2	44.7	45.1	45.4	45.6
$SM_{30P}S_1$	33.9	37.3	38.9	41.1	36.5	34.6	32.4	35.7	38.9	40.1	43.2	35.8	33.7	31.6
$SM_{30P}S_2$	33.4	36.8	38.2	39.9	31.5	29.4	26.8	35.2	37.8	39	42.4	30.7	28.6	25.7
$SM_{30P}S_3$	27.3	29.8	33.1	34.7	25.1	23.3	20.2	34	36.9	38.5	41.5	24.6	22.5	19.7
OM_0S_3	57.1	63.8	67.6	61.2	52.9	49.2	45.1	48.8	50.1	57.6	50.2	44.6	41.3	38.9
$OM_{30L}S_3$	52.1	54.9	58.9	51.6	43.2	39.9	36.3	41.3	45.6	50.9	45.6	40.3	38.1	35.5
$OM_{30P}S_3$	25.5	27.8	32.1	24.8	22.4	19.1	16.2	32.4	33.9	35.7	29.2	19.6	17.8	15.6

Optimization of Geopolymer Concrete Based on Local Iraqi Metakaolin

Basil S. Al-Shathr, Tareq S. Al-Attar, and Zaid A. Hasan

Abstract**—** Geopolymer concrete is one of the building materials that has become more popular in recent years due to the fact that it is significantly more environmentally friendly than normal concrete. Geopolymer is manufactured by using aluminate and silicate compounds that react with alkaline material (sodium hydroxide and sodium silicate or potassium hydroxide with potassium silicate). This paper presents an optimization for mix design purposes for Iraqi Metakaolin based Geopolymer concrete. The concentration of the alkaline solution used was 10 Molar. Two ratios of sodium silicate to sodium hydroxide solutions by weight were adopted. They were 3.5 and 1.0. Naphthalene sulphonate-based superplasticizer was used. Specimens were cured by different methods inside and outside the laboratory. The optimum mix proportions of Geopolymer concrete based on compressive strength at 7 days were 400 kg Metakaolin, 180 kg alkaline solution (sodium hydroxide and sodium silicate), 40 kg extra water, 1100 kg coarse aggregate and 720 kg fine aggregate. The results showed that the optimum dose of superplasticizer was 12 kg/m^3. By increasing this content, the compressive strength will be decreased by 32.5%. Results indicated also that the fineness of Metakaolin has an important role on strength development of the Geopolymer. Compressive strength at 7 days age for the optimum mix was 27.53 MPa. This result was recorded for specimens made with Metakaolin had a specific surface area of 23m^2/g and cured under ambient environment outside the laboratory.

Keywords — alkaline solution, compressive strength, Geopolymer concrete, Metakaolin.

I. INTRODUCTION

IN order to reduce carbon dioxide emission into the air as a side effect of producing Portland cement, Geopolymer concrete now has been developed. Researchers [1,2] reported that the production of one ton of Portland cement produces one ton of carbon dioxide to the air and that contributes much to the global warming. Therefore, now Geopolymer concrete becomes popular material because it does not use Portland cement as a binder. But it uses natural materials such as Metakaolin, Fly Ash and Rice Husk Ash as a binder. Davidovits [3] stated that natural material for replacing Portland cement in Geopolymer concrete must contain high percent of silica and alumina. These elements react with alkaline liquids to develop a polymerization process results in producing Geopolymer concrete.

The Metakaolin, which has high content of silica and alumina, reacts with alkaline solution like sodium hydroxide NaOH or potassium hydroxide KOH, and sodium silicate Na_2SiO_3 or potassium silicate K_2SiO_3, to form a gel which binds the fine and coarse aggregate. Geopolymer concrete do not require any water for matrix bonding, instead the alkaline solution react with silicon and aluminum present in the Metakaolin or fly ash. The polymerization process involves a substantially fast chemical reaction under alkaline condition on Si-Al minerals [4].

Geopolymer concrete produced without using elevated heat for curing will widen its application to the areas beyond precast members. Hence this study aims to produce Geopolymer concrete suitable for ambient curing condition under sunlight and compare it with other types of available curing systems.

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Basil S. Al-Shathr, is with UOT team, Iraq.
Email:basil1958a@yahoo.com
Tareq S. Al-Attar, is with UOT team, Iraq, Email: drtattar@yahoo.com
Zaid A. Hasan, Diwanyah Technical Institute, Iraq.

II. RESEARCH SIGNIFICANCE

There are too limited previous studies on Metakaolin based Geopolymer concrete. In Iraq, kaolin clays are available in large deposits and need only to be calcined to transform to Metakaolin. Thus these clays after calcination could be a good resource of silica and alumina and could be employed in producing Geopolymer concrete. This research may be considered as the first trials to produce sustainable Geopolymer concrete by using Iraqi Metakaolin.

III. MATERIALS

The origin of Metakaolin was Iraqi kaolin clay brought from Dewekhla region, Al-Anbar Governorate. Metakaolin prepared by grinding the clay by air blasting and then burnt in furnace up to 700ºC ± 20ºC for 1 hour, then the Metakaolin was cooled to room temperature for 24 hrs. This preparation procedure was recommended by Ibrahem and Wahab [5].The chemical composition of Metakaolin is shown in Table 1. Specific surface area of the prepared Metakaolin was 16.5m^2/g.

The alkaline solution consisted of sodium hydroxide NaOH and sodium silicate Na_2SiO_3. Sodium hydroxide is available in the local markets in the pellet form with purity of more than 98 percent. 10 Molar solution was prepared for this study. The sodium silicate solution has a ratio of SiO_2 to Na_2O of 2.4, which include 32.5 percent SiO_2, 13.4 percent Na_2O and 54.1 percent of water.

A high-range water reducer, was used for the production of rheoplastic concrete. It is based on a sulphonated naphthalene polymer.

The fine aggregate (sand) were obtained from Al-Ukhadir region, Karbala Governorate. The grading and sulfate content were conforming to the requirements of Iraqi Standard, IQS No.45/ 1984 – Zone 2.The coarse aggregate was crushed gravel with maximum size of 19 mm. The grading of this aggregate conforms to the Iraqi Standard, IQS No.45/ 1984, (5-19 mm).

Table 1
Chemical composition for Metakaolin

Oxide composition	Oxide content %
SiO_2	51.59
Al_2O_3	38.11
Fe_2O_3	1.82
CaO	0.45
MgO	0.23
SO_3	0.14
K_2O	0.43
Na_2O	0.11
L.O.I	6.12

IV. MIXING and CASTING of GEOPOLYMER CONCRETE

The Metakaolin, fine and coarse aggregate were mixed by pan mixer in dry condition for three minutes and then the alkaline solution was added (with superplasticizer , with or without extra water) and mix for another four minutes to prepare the Geopolymer concrete [6]. The Geopolymer concrete was casted in 100×100×100 mm cube molds and compacted by a vibrating table. After 24 hours, the specimens were demolded and started the specified curing system until the age of test. The Geopolymer concrete was white in color with an acceptable appearance.

V. MIX PROPORTIONS OPTIMIZATION

Twelve mixes were made to study the effect of different variables on strength. These mixes were produced with Metakaolin has the fineness of 16.5 m^2/g and cured by sunlight. The studied variables were: alkaline solution to Metakaolin ratio, extra water to Metakaolin ratio, superplasticizer to Metakaolin ratio and Metakaolin content. The details of these mixes and the resulted compressive strength are listed in Table 2.

The optimization for mix proportions was according to the following factors:

1. Silicate to hydroxide ratio of the alkaline solution.
2. Curing system.
3. Specific surface area (fineness) of Metakaolin.

Davidovits [3] recommended that the sodium silicate solution and the sodium hydroxide solution are mixed together one day prior to the use in preparing the Geopolymer concrete. This recommendation was followed for this study. Many trials have been made to select the optimum silicate to hydroxide ratio. This selection was made according to the compressive strength of concrete at 7 days age, as shown in Table 3, the optimum ratio of sodium silicate solution to sodium hydroxide solution is 3.5.

Different systems of curing were studied in this research. The sunlight curing started after removing specimens from molds, where the specimens kept under sunlight outside the laboratory in June when the ambient temperature ranged from 34 to 48 °C. The halogen curing system was implemented by using 500 watt halogen lamp concentrated on specimens till the age of test. In addition to that, a mixed system for curing was investigated. It consisted of a heat curing in oven at 60 °C for 6 hours, then starting either sunlight or halogen curing and continues for the age of test. Table 4 displays the effect of different curing systems on compressive strength of Geopolymer concrete.

Table 2
Details of the Geopolymer concrete mixes produced with different variables

Mix	MK, kg/m²	Alkaline solution, (% of MK)	Extra Water, (% of MK)	Coarse Aggregate, kg/m²	Fine Aggregate, kg/m²	Superplasticizer, kg/m²	Comp. Strength at 7 days, MPa
M1	400	0.45	0.15	1100	720	12	10.3
M2	400	0.4	0.2	1100	720	6	2.84
M3	400	0.4	0.3	1100	720	8	1.72
M4	400	0.4	0.25	1100	720	10	2.10
M5	400	0.45	0.25	1100	720	10	3.16
M6	400	0.4	0.25	1100	720	10	2.48
M7	400	0.45	0.1	1100	720	12	13.72
M8	400	0.45	0.1	1100	720	16	9.26
M9	400	0.45	0.1	1100	720	8	9.24
M10	400	0.45	0.05	1100	720	12	11.03
M11	500	0.45	0.05	1100	720	12	11.96
M12	400	0.45	0.0	1100	720	12	8.49

Table 3
Effect of hydroxide to silicate ratio on Geopolymer concrete strength

NaOH to Na_2SiO_3 ratio	Compressive Strength at 7 Days Age, MPa
1:2.5	7.32
1:3.5	13.72
1:4.5	4.40

Table 4
Effect of curing systems on compressive strength of Geopolymer concrete

Type of curing	Compressive Strength at 7 Days Age, MPa
Sunlight	27.53
Halogen (500W)	26.48
Heat curing (60°C for 6hrs) +sunlight	15.60
Heat curing (60°C for 6hrs) + halogen	18.64

Table 5
Effect fineness of Metakaolin on concrete compressive strength

Specific surface area of Metakaolin, m^2/g	Compressive Strength at 7 Days Age, MPa
16.5	13.72
19.6	22.69
23.0	27.53

To study the effect of specific surface area (fineness) of Metakaolin on Geopolymer strength, mix M7 (in Table 2) was reproduced by using Metakaolin with fineness of 19.6, and 23m^2/g. The results are shown in Table 5.

VI. RESULTS AND DISCUSSION

Table 2 shows the compressive strength of the different Geopolymer mixes produced with different variables.

1. Superplasticizer dose:

The increase in superplasticizer content leads to increase the compressive strength of Geopolymer up to an optimum level, 12 kg/m^3, at which strength reaches 13.72 MPa at 7 days age. After that limit, the strength decreases, as shown in Figure 1. By increasing the dose from 12 kg/m^3 to 16 kg/m^3, mix M7 and M8, the reduction in compressive strength was 32.5 percent. Moreover, decreasing superplasticizer content from 12 kg/m^3 to 8 kg/m^3, mix M7 and M9, the reduction was 32.6 percent.

2. Extra water content:
Table 2 and Figure 2 show that it is preferable to add extra of water to the mix, 10 percent by the weight of Metakaolin, to get the optimum strength. This may be due to some impurities in Iraqi Metakaolin that absorb water causing reduction in workability which lead to incomplete compaction without this extra water content. For mixes M7 and M10, when the extra water reduced from 10 to 5 percent, the strength reduced by 19.6 percent. Moreover, mix M12, which was made with no extra water, showed 38.1 percent reduction in compressive strength as compared with the optimum mix M7.
On the other hand, increasing extra water more than 10 percent by weight of Metakaolin caused the lowering of strength. For mixes M7 and M1, by increasing extra water from 10 to 15 percent, the compressive strength decreased by 25 percent. The reduction in compressive strength of Metakaolin based Geopolymer concrete could be attributed to the increase in water in alkaline solution which lowers the NaOH concentration. This behavior is similar to normal concrete as the effect of increasing the water to cement ratio would lower the compressive strength [7].

3. Alkaline solution to Metakaolin ratio:
The results indicate that increasing the ratio of solution to Metakaolin from 40 to 45 percent causes an improvement in strength by 33.5 percent. For the alkaline Geopolymerisation reactions, more Si aids in the production of Si-O-Si bonds, and significantly increases the compressive strength of the Geopolymer. If the content of Si exceeds the suitable limit, the Geopolymerisation rate is negatively affected, leading to Geopolymer of low strength [8].

4. Curing system:
According to Table 4, it can be seen that curing the specimens outside the laboratory under sunlight gives the highest strength. This is reasonable because Geopolymerisation need high temperature. Such a case is very convenient and economical in hot weather countries such as Iraq. In cold weather it is recommended to adopt another system of curing such as curing with halogen light. The compressive strength at 7 days of specimens cured by this system was 26.48 MPa which is too near to the value of hot weather sunlight curing system.

5. Specific surface area (fineness) of Metakaolin:
Table (4) shows that the Metakaolin fineness has a great role in gaining Geopolymer strength, as the increase in surface area from 16.5 to 23 m^2/g causes an increase in strength by about 100 percent to reach 27.53 MPa at 7 days, which could be considered as a good early strength that encouraging producing precast units from this material.

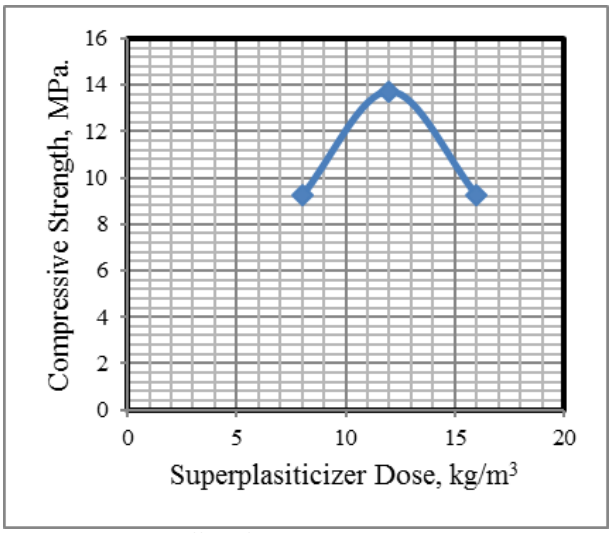

Figure 1: Effect of superplasticizer dose on compressive strength of Geopolymer concrete.

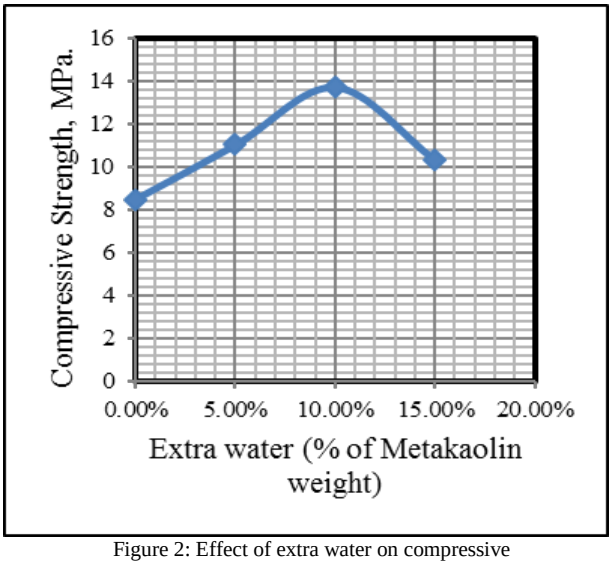

Figure 2: Effect of extra water on compressive strength of concrete

6. Metakaolin content:
The results also show that increasing the content of Metakaolin from 400 to 500 kg/m^3 leads to a slight increment in compressive strength. There is only an 8 percent difference in strength between mixes M10 and M11. Increasing the Metakaolin content could lead to an increase in Si and Al content and may increase Geopolymer gel that binds aggregate and increase compressive strength [9].

VII. CONCLUSIONS

1. Geopolymer could be considered as significant alternative for Portland cement because it has a good early compressive strength. The optimum mix proportion of Geopolymer concrete based on compressive strength at 7 days are 400 kg Metakaolin,

180 kg alkaline solution (sodium hydroxide and sodium silicate), 40 kg extra water, 1100 kg coarse aggregate and 720 kg fine aggregate.

2. The specific surface area (fineness) of Metakaolin has a great role in strength development, i.e. the relationship is positive between Metakaolin fineness and the Geopolymer concrete strength.
3. Geopolymer cured outside the laboratory under sunlight shows the highest strength. This case is very convenient and economical in hot weather countries such as Iraq in summer season.
4. In cold weather it is recommended to adopt another system of curing such as curing with halogen lights.
5. The extra added water has the same effect on Geopolymer concrete strength as that on Portland cement concrete strength.

REFERENCES

[1] D.M. Roy, "Alkali – Activated Cement, Opportunities and Challenges." Cement and Concrete Research, Vol. 29, Issue 2, 1999, pp. 249-254.

[2] E. Worrell, L. Price, N. Martin, C. Hendriks, and L.O. Meida, "Carbon Dioxide Emissions from the Global Cement Industry" Annual Review of Energy and the Environment Journal, USA, Vol. 26, 2001, pp. 303 – 329.

[3] J. Davidovits, "Properties of Geopolymer Cement" 1st International Conference on Alkaline Cement and Concrete, Kiev, Ukraine, 1994.

[4] M. I. Abdul Aleem and P. D. Arumairaj, "Optimum Mix for the Geopolymer Concrete" Indian Journal of Science and Technology, Vol. 5, No. 3, 2012, pp. 2299 – 2301.

[5] A.M. Ibrahem and A.A. Wahab, "Effect of Temperature on the Pozzolanic Properties of Metakaolin Produced from Iraqi Kaolin Clay". AL- Fatih Journal, Diyala University, Iraq, Vol. 4, Issue 32, 2008, pp. 268-285.

[6] D. Hardjito and S.E. Wallah, "On the Development of Fly Ash-Based Geopolymer Concrete" ACI Materials Journal, Vol. 101, Issue 6, 2004, pp. 467-472.

[7] G.S. Patil and K. Manojkumar, "Factors Influencing Compressive Strength of Geopolymer Concrete" International Journal of Research in Engineering and Technology, 2013, pp. 372 – 375.

[8] H. Kamarudin and A.K. Omar, "The Effect of Alkaline Activator Ratio on the Compressive Strength of Fly Ash-Based Geopolymer" Australian Journal of Basic and Applied Sciences, ISSN 1991-8178, 2011, pp. 1916-1922.

[9] J.N.Y. Djobo and H.K. Tchakoute, "Synthesis of Geopolymer Composites from a Mixture of Volcanic Scoria and Metakaolin" Journal of Asian Ceramic Societies, Vol. 2, Issue 4, 2014, pp. 387–398.

Dr. Basil S. Al-Shathr was born in Baghdad, Iraq in 1958. He received his BSc in building & construction engineering in 1979, his MSc in concrete design and technology in 1983, and he received his PhD in building materials engineering in 2009. These degrees were all from the University of Technology, Iraq.

At present, he is an assistant professor of civil engineering in the Building and Construction Engineering Department, University of Technology, Iraq. His research interests include sustainability of building materials, in addition to the behavior of high performance and ultra-high performance concrete. He published more than 20 research paper and has three patents in the field of building materials.

Dr. Al-Shathr is a member of the Iraqi Union of Engineers since 1979.

Corrosion of Steel Reinforcement in High Performance Concrete Containing Local Slag

Dr. Suhair K. Al-Hubboubi **Prof. Dr. Hana A. Yousif**

***Abstract*— In this research, high performance concretes (HPC) were prepared using high range water reducing admixture (HRWR) with 10% local grounded slag as a partial replacement by weight of cement individually or synergistically with 50% granulated slag by weight of fine aggregate. The electrochemical behavior of reinforcing steel in these concretes was studied. The specimens were partially submerged in saline solution at concentrations similar to those present in the soil and underground water of the south of Iraq. Corrosion of embedded steel was monitored electrochemically by measuring the half –cell potentials and corrosion currents. The result indicated that, under the action of aggressive solution, HPCs showed considerable improvement in compressive strength and reduction in corrosion activity over those of HRWR-concrete.**

***Index Terms*— Corrosion, Granulated slag, High performance Concrete, Reinforcing Steel.**

I. Introduction

Concrete construction in Arabian Gulf seaboard countries shows an alarming degree of deterioration within the duration of 10 to 15 years. The deterioration is accentuated by the geomorphic and climatic environmental conditions, which are characterized by reactive and marginal aggregates, high temperature –humidity regimes, and severe ground and ambient salinity [1]. Several solutions to this problem have been proposed and tested, though to date no ideal solution has been found. Protection against penetration of salts is affected by the permeability of concrete .The use of low water/cementitious blended cement concrete is most likely to ensure optimized performance of concrete. Using superplasticizer as a high range water reducing admixture (HRWR) in concrete becomes of great and beneficial importance due to its performance in reducing the water to cement ratio (w/c) of concrete, in turn increasing its strength with significant reduction in permeability [2].The use of mineral admixtures in concrete has the advantages of reducing the micro-craking and strengthening the transition zone through the processes of pore size and grain size refinement.

Blast furnace slag is a byproduct obtained while melting iron ore in a blast furnace. By melting the iron ore at 1400-1600°C pig iron is produced and the floating impurities, containing mainly lime, silica and alumina from the blast furnace slag. By slow cooling of the slag crystalline material is produced which is used as aggregate and has no cementing properties. Glassy pellets (> 4 mm) produced on rapid cooling form excellent lightweight aggregate and granules (> 4 mm) on grinding possess hydraulic properties. This granulated ground blast furnace slag (GBFS) is used for the production of blast-furnace cement. The ground blast furnace slag exhibits hydraulic action in the presence of calcium hydroxide liberated by Portland cement when hydrated. The ground slag is blended with Portland cement to produce Portland blast furnace slag cement, the proportion of the former not exceeding 65 per cent. The early strength of the cement so produced might be less, but the ultimate strength is comparable. Because of a low heat of hydration the ground blast furnace slag cement finds its application in mass concreting. The other advantages of the addition of blast furnace slag to cement are improved workability, resistance to chemical attack and the protection provided to reinforcement that makes it suitable for reinforced concrete and pre-stressed concrete [3].

The local slag is produced from the Electric Arc Furnace (EAF) by slow cooling. Damping away this by product represents a waste of the material and causes serious environmental pollution problems. Least amount of work was conducted to investigate the local slag.

Al-Mulla [4] investigated the use of Iraqi slag as cementing material. He used Iraqi slag to produce different kinds of blended cement by replacing part of ordinary Portland cement with slag in different percentages (10, 20...90 and 100). Results indicated that the replacement should not be more than 40% due to the low activity of Iraqi slag. Also, he added the slag to the concrete as an admixture, without

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Suhair Kadhem Abed Alhubboubi is with the Building Research Directorate / Ministry of Const. and Housing, Iraq; e-mail: suhairkah@yahoo.com.

Prof. Dr. Hana Abid. Yousif was a professor of Building and Construction Engineering at University of Technology, Baghdad, Iraq. Dr. Yousif is currently a professor of Civil Engineering at University of Akron, OH, USA. Her research interests include durability of ultra-high strength cementitious composite, self-compacting concretes, cement replacement materials, and electrochemical corrosion of steel in high strength concrete, reactive powder concrete, and high performance lightweight concrete, and concrete durability and repair.

replacement at percentages (10, 20...90 and 100) by weight of cement, the results confirmed that when the slag was increased, there was a reduction in workability and an improvement in both strength and initial surface absorption. Considerable research work has been conducted to examine the influence of different types of mineral admixtures such as Slag, Fly Ash and Silica Fume on the durability of reinforcing steel embedded in concrete [5, 6]. Other investigators examined the corrosion performance of steel in high-strength self-compacting concrete exposed to chloride solution [7]. In this work, the electrochemical behavior of steel in high performance concretes (HPCs) containing 10% local grounded slag as a partial replacement by weight of cement individually or combined with 50% granulated slag by weight of fine aggregate was investigated. The specimens were partially submerged in saline solution at concentrations identical to those exist in the soil and underground water of the south of Iraq.

II. Experimental work

A. Materials

Ordinary Portland cement produced at the Kubaysa cement factory, with a fineness of 2880 cm^2/gm. was conformed to the Iraqi specification No. 5/1984 was used. The fine aggregate was Al-Ukider natural sand with grading limits in zone 2. The coarse aggregate was crushed gravel of 12.5 mm maximum size. Melamine-based super-plasticizer was used throughout this research work as HRWA. Sulphonated melamine formaldehyde condensate (Melment L_{10}) is usually available as a 20% aqueous solution with a density of 1.1 gm. $/cm^3$ and is clear to turbid or milky in appearance. The local slag was brought from Al-Basrah factory .Table 1 gives the chemical composition of this slag .It was used as a partial replacement by weight of cement and/or as a partial replacement by weight of sand. In order to use slag as a fine aggregate, large rocks of slag were ground to have a grading similar to that of sand namely zone 2. Slag was also finally crushed using (ball mill) to obtain pulverized slag used as a partial replacement of cement. The specific gravity of such slag was 4.1 using Le chatelier flask in accordance with ASTM C188-86. Blaine method was used to control the fineness of slag and was 5200 cm^2/gm.

B. Concrete mixes

Concrete mixes were designed in accordance with building research establishment method to have a minimum compressive strength of 40 N/mm^2 at 28 days. Ordinary Portland cement concretes with cement: sand: gravel ratio of 1:1.335:1.447 was prepared, cement content was 550 kg/m^3, and the water/cement ratio of reference mix to obtain slump 100±5 mm was 0.39. Three types of high performance concrete mixes were investigated; HRWA-concrete, concrete containing 10% slag as a partial replacement by weight of cement with normal dosage of super-plasticizer (SC-HRWA), and concrete mix containing 50% slag as a partial replacement by weight of sand with 10% slag as a partial replacement of cement as well as with normal dosage of super-plasticizer (SSC-HRWA). The water/cement or water/cementitious materials ratios were adjusted to maintain equal workability to those reference concrete.

TABLE 1
CHEMICAL ANALYSIS OF SLAG

Oxide composition	Oxide content for tested slag*	ACI 226, % blast furnace slag
SiO_2	26.01	32-40
CaO	26.6	7-17
Al_2O_3	8.28	29.42
MgO	8.00	8-19
SO_3	0.07	0.7-2.2
L.O.I	Nil	-
Total iron as (FeO or Fe_2O_3)	24.07	0.1-1.5
Na_2O	0.2	
K_2O	0.04	

*chemical analysis was carried out by the state company of geological survey and mining

C. Specimen preparation and exposure conditions

150*150*530 mm concrete columns reinforced with four, 12 mm, diameter longitudinal deformed bars and 6 mm lateral ties were used as a clear cover of 40 mm. All the specimens were cured in tap water for a period of 27 days after de-molding. At the end of the curing period, the specimens were taken out of the water and left in the laboratory for a period of 3 days. The exposure conditions used throughout this research work were the partial submersion in severe aggressive solution containing $Cl^- + SO_4^{--}$ ions. The curing solution was prepared on the basis of water analysis report made by the state enterprise for geological survey and mining of the southern part of Iraq. The data presented in this report showed that the chloride ion concentration ranges between 20000 and 40000 ppm, while the sulfate ion concentration lies between 5000 to 7000 ppm. The cations concentrations were 10000 to 20000 ppm of sodium, 1500 to 2000 ppm for magnesium and 1000 to 1500 ppm for calcium. The salts used in preparing the solution were pure NaCl, $CaCl_2.2H_2O$, and $MgSO_4.7H_2O$. Tap water was used as a solvent for these salts. Table 2 illustrates the types and concentrations of salts used in curing solution and the actual anions and cation provided by such salts.

III TEST RESULTS AND DISCUSSION

A. Effect of slag on compressive strength

Compressive strength was measured on 100 mm cube partially submerged in $Cl^- + SO_4^{--}$solution. HRWA- concrete showed continued strength development up to 180 days. This increment in strength is attributed to the reduced water/cement ratio and to the excellent dispersion of cement particles throughout the mixes. Since the attack of aggressive ions takes place, only when water or solution can penetrate into the concrete, the impermeability of concrete is considered its most important attribute.

TABLE 2
TYPES AND CONCENTRATIONS OF SALTS AND IONS USED IN CURING SOLUTION

Type of salt	Concentration Ppm	Concentration gm/L	Salt content % by Wt. of curing solution
NaCl	45100	45.1	4.51
$CaCl_2.2H_2O$	55125	5.5	0.55
$MgSO_4.7H_2O$	17967	17.97	1.79
$Cl^- + SO_4^{--}$ solution			
Anions		Cations	
Type	ppm	Type	ppm
Cl^-	30019.5	Mg^{++}	1773
SO_4^{--}	7002.7	Na^+	17750
		Ca^{++}	1500

SC-HRWA concrete exhibited a progressive strength gain up to 180 days. The use of highly active pozzolans as a partial replacement by weight of cement has been reported to increase the resistance of concrete to deterioration by aggressive chemicals such as chlorides and sulfates [8-10]. This increase in durability has been attributed to reduced permeability, thereby, reduced ion diffusion, resulting from a finer pore structure, in addition to reduced contents of easily leached or reacted calcium hydroxide in the hardened paste fraction [11]. SSC-HRWA concrete showed superior performance to those of the other mixes exposed to similar environments. This superiority over the SC-HRWA concretes is related to better cement-aggregate bond associated with improved mechanical interlocking, thereby, strengthening the transition zone and reducing the micro cracking at the interface**.** Fig.1

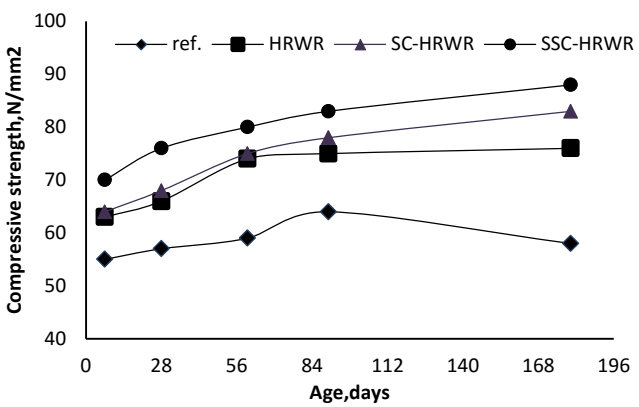


Fig. 1 Compressive strength of reference and HPCs

B. *Electrochemical measurements*

Half-cell potentials

The corrosion activity was monitored using high-impedance voltmeter and noting the half-cell potentials with respect to a reference Copper-Copper Sulfate electrode (CSE). According to ASTM Test method (C876) for half-cell potentials of uncoated reinforcing steel in concrete, if the half–cell potentials are in the range of -200 to -350 mV against (CSE), corrosion activity is uncertain. If the potentials are numerically more negative than -350 mV, there is a greater than 90% probability that reinforcing steel corrosion is occurring. The results of half-cell potentials of steel are shown in Fig. 2.Specimens incorporating HRWA only, showed lower steel potential after about 140 days of exposure compared with the corresponding reference concrete, indicating that passivity was achieved and maintained. This improvement may be attributed to the reduction in the water content of super-plasticized concretes leading to low permeability, thereby, reducing the penetrability of the aggressive ions.

The results also demonstrated that the combined action of pulverized slag and HRWA in SC-HRWA concrete provide an excellent reduction in steel potentials over those obtained for steel in all other types of concrete. The superior performance of SC-HRWA concrete over the other types is attributed to the reduced permeability, which limits the intrusions of extraneous ions. This is mainly caused by pore size and grain size refinement processes suggested by Mehta [12]. These processes strengthen the transition zone and reduce the micro cracking. Reinforcing steel in SSC-HRWA concretes exhibited lower potential than those of steel in HRWA concrete as well as in the reference concrete up to the end of the testing period. This is mainly linked to a lower water content of the mix in addition to the formation of the denser gel.

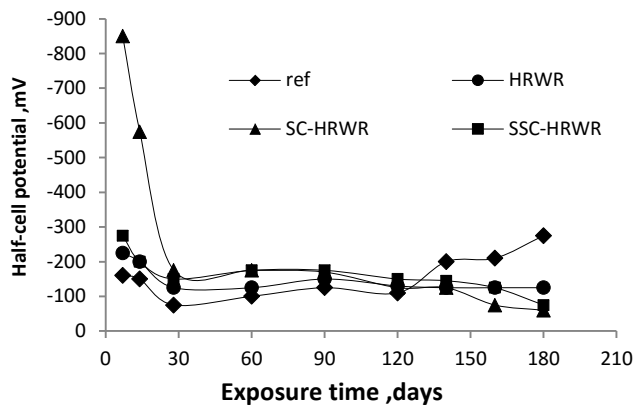


Fig.2 Half-cell potentials of reinforcing bars in reference and HPCs

C. *Polarization measurements*

Tafel plot technique

In this technique, a potential scan was applied to the specimen starting from $Ecorr$ and extending cathodically to -120 mV and anodically to +120 mV. The polarization rate of 10 mV/min was used and the flowing current was recorded at the end of each 30 seconds for every 5 mV potential shift. The potential is plotted against the measured current using logarithmic plots. To restore the original value of the electrode potential $Ecorr$., a certain waiting time is needed between the cathodic and anodic measurements. A potentiostat type "PRT 10-0.5" which is capable to maintain a signal potential of the sample at preferred value relative to the suitable referenced electrode was used. Tafel constants (anodic Tafel constant (Ba) and cathodic Tafel constant (Bc)) can be determined

from the slopes of the straight line portions of a polarization curve, which usually occur at more than 30 mV shift from $Ecorr$..The intersection of the linear region of the plot with $Ecor_{r.}$ gives $icorr$.. Fig 3 presents the experimental setup for electrochemical tests and Fig.4 shows Tafel plots of steel reinforcement in various types of concretes.

Linear polarization technique

In this technique, a controlled potential scan is applied to the specimen in a range smaller than that used in the Tafel plot technique. It was from $(Ecorr. - 30\ mV)$ to $(Ecorr. + 30\ mV)$ using a polarization rate of 10 mV/min. the flowing current was recorded each 30 seconds for every 5 mV potential shift. The resulting current is linearly plotted against the potential. The polarization resistance $Rp\ (\Delta E/\Delta I)$ is obtained from the slope of the potential-current curve near $Ecorr.$ in the cathodic and anodic direction. The polarization resistance Rp is related to the corrosion rate $Icorr.$ through the following Stern-Geary relationship [13].

$$Icorr. = B/\ Rp \quad (1)$$

$$B = Ba \times Bc/\ 2.3\ (Ba + Bc) \quad (2)$$

Fig. 5 shows the variations in the corrosion current, $Icorr,$ with time. At 180 and 240 days of exposure, reinforcing steel in HRWR-concrete showed 48%, 58% reduction in corrosion current compared to steel in the reference concrete. The corresponding reductions in corrosion currents when the slag was used as a partial replacement by weight of cement in conjunction with HRWR were 43%, 54%, whereas the steel in SSC-HRWR exhibited 34% and 38% reduction. The superior performance of steel in SC-HRWR concrete over that of steel in SSC-HRWR is mainly associated with pore size and grain size refinement processes which manage the micro-cracking as well as strengthen the contact zone between aggregate and cement paste. This transition zone may be negatively influenced when 50 % of sand was replaced by porous granulated slag in SSC-HRWR concrete as it may facilitate the penetrability of aggressive solution, thereby reducing the embedded steel passivity.

Corrosion current data determined by Stern-Geary equation is represented in Fig. 5. Table 3 presents Tafel data ($Ba, Bc, B, icorr.$), RP and $I\ Corr$ for steel reinforcements in reference and HPCs measured at 240 days of exposure.

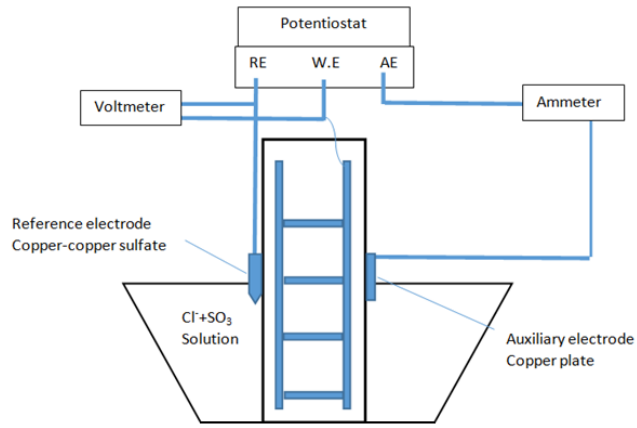


Fig.3. experimental setup for electrochemical tests

Fig.4 Tafel plots for steel reinforcement in various types of concretes

Fig. 5 Corrosion current variation with exposure time

TABLE 3
TAFEL CONSTANTS AND CORROSION CURRENTS FOR STEEL IN REFERENCE AND HPCS AT 240 DAYS

Concrete mixes	Quasi potentio dynamic –static $RP-\Omega$	Tafel constant mV/decade Ba	Bc	B	$icorr.$ Tafel 10mv/ min	$Icorr.\mu A$ Stern quasi potentio-dynamic static= B/RP
Reference	358	326	184	51	110	142
HRWR	1088	298	293	64	34	58
SC-HRWR	1022	312	300	66	40	65
SSC-HRWR	800	328	317	70	45	87.5

IV CONCLUSIONS

Based on the results of this investigation, the following conclusions can be drawn:

1- At the optimum dosage of HRWR (5.5% by weight of cement),it was possible to attain high strength concrete with an average 28 day compressive strength of 61 N/mm^2 compared to 49 N/mm2 for reference concrete. The corresponding values for SC-HRWR and SSC-HRWR concretes were 61 N/mm^2 and 65 N/mm^2 respectively.

2- The combined action of pulverized slag and HRWA in SC-HRWA concrete provides excellent reduction in steel potentials over those obtained for steel in all other types of concrete.

3- At 180 and 240 days of exposure, reinforcing steel in HRWR-concrete showed 48%, 58% reduction in corrosion current compared to steel in the reference concrete. The steel in SC-HRWA showed considerably better performance compared to steel in SSC-HRWA concrete. This may be attributed to the fact that the processes produced by the pozzolanic reaction between ground slag and cement can lead to the pore size and the grain size refinement. These processes have the influence of minimizing the micro-cracking in the transition zone between aggregate and cement paste and reducing the intrusion of extraneous ions into the concrete thus, can control the corrosion process. This contact zone may be adversely influenced when 50 % of sand was replaced by porous granulated slag in SSC-HRWR concrete as it may facilitate the permeation of aggressive solution, thereby reducing the embedded steel passivity.

REFERENCE

[1]Rasheeduzzafar,Dakhil,F.H.,AL-Gahtani,A.S.,AL-Saadoun,S.S.andBader,M.A., "Influence of Cement Composition on the Corrosion of Reinforcement and Sulfate Resistance of Concrete," ACI Journal ,Mar-Apr.1990,pp.114-122.

[2] Neville, A.M., "Properties of Concrete," Pearson Education Limited. Fourth and final edition, 2005, 843 pp.

[3] S.K. Duggal," Building materials", New Age International (P) Ltd, New Delhi, Third revised edition, 2008,p.517.

[4] Al-Mulla,J.A,"Effect of using Iraqi slag in concrete manufacturing",Msc.Thesis,Baghdad University ,1992,p.88.

[5] Amir Poursaee, Carolyn M. Hansson. The influence of longitudinal cracks on the corrosion protection afforded reinforcing steel in high performance concrete. Cement and Concrete Research, Vol. 38, 2008, pp. 1098-1105.

[6] Ha, T.-H, Muralidharan, S., Bae, J.-H., Ha, Y. - C., Lee, H.-G., Park, K.-W., and Kim, D.-K., “Accelerated Short Term Techniques to Evaluate the Corrosion Performance of Steel in Fly Ash Blended Concrete,” Building and Environment, V. 42, No. 1, 2007, pp. 78-85.

[7] Hana A. Y., Farqad F. Al-H., Bashar Al-N., and Amani A. A. Corrosion of steel in high-strength self-compacting concrete exposed to saline environment. International Journal of Corrosion, V. 2014, Article ID 564163, pp.1-11

[8] Yousif,H,A,"Corrosion of reinforcement in concrete containing high range water reducing agent and rice husk ash,"Ph.D.Thesis ,University of technology .Jan.1997.pp.193.

[9] Abdulla,N,A.,"High performance reinforced concrete in view of its permeability and corrosion resistance,"Ph.D Thesis, University of technology .Feb1999.pp.205.

[10] Mutar ,A,A,"Durability of high performance concrete exposed to aggressive ions,"M.Sc Thesis University of technology .2000.pp.150.

[11] Hooton,R.D,"Permeability and pore structure of cement pastes containing fly ash,slag,and silica fume ,"Blended cement ,ASTM STP 897,1984.

[12]Mehta,P.K.,"Concrete structure.properties ,and materials,"Prentice-hall,Inc.,New Jersy,1993,p.496

[13] Stern, M., and Geary, A. L. “Electrochemical Polarization-I a Theoretical Analysis of the Shape of Polarization Curves,” Journal of the Electrochemical Society, V. 104, No. 1, January, 1957, pp.56-63.

Improving Construction Safety Performance For Iraqi Contracting Companies

Ali Mohammed-Hasan Majeed Albaghdadi , Dr Sedki E.Rezouki

***Abstract*— The construction industry has long been regarded as one of the most dangerous industries, as it has a history of poor safety performance.**
A literature survey has been done, and the methods and techniques for measuring safety performance in construction projects have been identified. Therefore, in this study, from the field survey, the Iraqi construction contracting companies are investigated for the safety measures of their projects. A questionnaire survey has been conducted for a selected sample of the Iraqi contracting companies consisting of a significant number of questions focusing on each company safety policies and practices.
Statistical analysis of the questionnaire shows that there are great weaknesses in Iraqi construction companies in controlling the safety for all employees in the projects in all parts of the questionnaire.
As a result of what has been concluded, solutions and recommendations and proposals given in the research to obtain the ideal condition of safety for the Iraqi companies .

***Index Terms*— system of safety; safety performance; safety evaluation**

I. INTRODUCTION

The construction industry plays a vital role in the social and economic development of all countries; the construction industry has historically experienced a disproportionately high rate of disabling injuries and fatalities for its size. This industry alone produces 30 % of all fatal industrial accidents across the European Union (EU), yet it employs only 10 % of the working population in the United States (US) it accounts for 20 % of all fatal accidents and only 5 % of the employed. In Japan, construction accidents account for 30 % - 40 % of the overall total of industrial accidents, with the totals being 50 % in Ireland and 25 % in the United Kingdom **Tauha, (2006).**

II. RESEARCH METHODLOGY

In order to achieve the research objectives, the following methodology has been adopted:

A. Literature Review

In this survey, the researcher study numbers of researches and past studies concerning with the safety in construction projects including (Books, Researches, Engineering Magazines, and the conferences concerning the safety in construction), in order to get information about the necessary complements for a successful methods for controlling safety in construction, as well as the actual status of safety in some construction companies in the world.

B. Field Work

This work has been divided into;

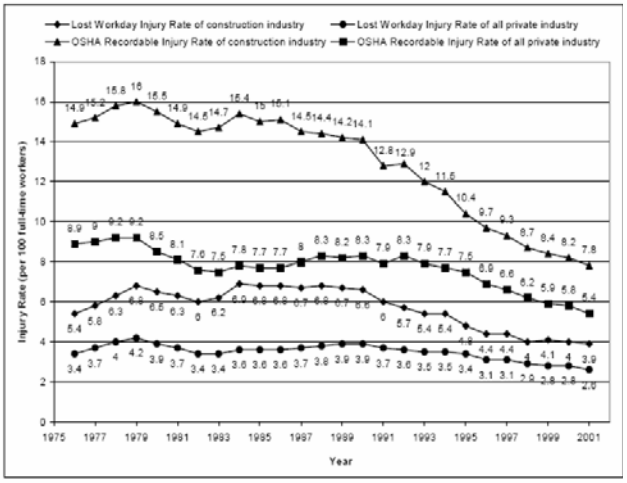
1. Field Survey; as information concerning the actual status of safety in construction industry in Iraq has been collected through personal interviews.
2. A closed questionnaire process to 50 fifty engineers in the construction companies .
3. Analysis of the closed questionnaire then give the result ,conclusions and recommendations.

ACCIDENT FACTS IN CONSTRUCTION

Accident data prepared by the Bureau of Labor Statistics**, Huang (2003),** shows that the construction industry has performed much worse than the average of all industries Fig(1), accident rates in the construction industry are still 50% higher than that of all industries, lagging all industries by about 10 years. An alarming fact is that the number of fatalities in the construction industry has increased in the past decade Fig (2).

SAFETY DEFINATIONS

According to the **Oxford Dictionary, (2006),** Safety is the state of being protected from or guarded against hurt or injury; freedom from danger.
According to **OHSAS 18001:1999,** Safety is freedom from unacceptable risk of harm.
The Researcher define the Safety as avoiding the risk of causing accident or incident by taking all protecting procedures to be safe form hazards risk.



Figure(1). Injury rate of construction and all private industry, (Huang2003)

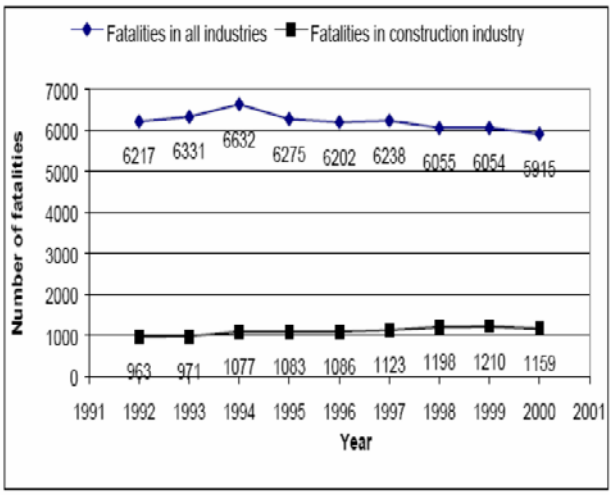


Figure (2) Fatalities in construction industry and all industries, (Huang 2003)

OCCUPATIONAL SAFETY AND HEALTH ADMINASTRATION (OSHA):

Introduction to OSHA

Until 1970 there are no laws that required for occupational safety and health in united state and in the year 1970 the American congress depend the laws of the occupational safety and health osh act, and in the 1971 the occupational safety and health administration was founded in the American work ministry to protected almost 90 million workers from the risk of the work and the accident of the work and to provide a safe work place. **Arab, eng, (2007).**

TECHNIQES FOR MEASURING CONTRACTOR SAFETY PERFORMANCE

One of the primary duty for safety professionals is to direct the management on safety performance measures. Contractors exploit these measures to monitor their own firm, to evaluate the safety procedures and to identify problem areas. Safety performance can be measured in different method and they are:

- Experience Modification Rating (EMR)

The Business Roundtable, (1982) in his report improving construction safety performance states that:

The insurance industry has developed experience rating systems as an equitable means of determining premiums for workers' compensation insurance. These rating systems consider the average workers' compensation losses for a given firm's type of work and amount of payroll and predict the dollar amount of expected losses to be paid by that

Employee in a designated rating period, usually three years. Rating is based on comparison of firms doing similar types of work, and the employer is rated against the average expected performance in each work classification. Losses incurred by the employer for the rating period are then compared to the expected losses to develop an experience rating.

Workers' compensation insurance premiums for a contractor are adjusted by this rate, which is called the experience modification rate adjusted by this rate, which is called the experience modification rate (EMR). Lower rates, meaning that fewer or less severe accidents had occurred than were expected, result in lower insurance costs. A contractor's EMR is adjusted annually by using the rate for the first three of the last four years.

Dave Smith, (2007) updates his paper and takes this formula to measure (EMR). The basic formula for a workers compensation experience modification factor (EMF) is:

EMF = Actual Losses/ Expected Losses (1)

EMF= Actual Losses/ (Payroll) x (Expected Loss Rate) (2)

The actual formula is much more complex. However, this simplified formula demonstrates that if actual losses exceed the expected losses the EMF is greater than 1 or 100%. If the actual losses are less than expected losses then the EMF is less than 1 or 100%. This leads many to conclude that the EMF is a good measure of safety performance.

- OSHA Recordable Incidence Rate:

Occupational safety and health administration (OSHA) recordable incidence rate is one of the effective safety performance measures which is generally available for comparisons between different contractors and companies. This rate is based on company's yearly total for fatalities, injuries, and illnesses with lost workdays and injuries and illnesses without lost work days used. The other information form time records, the number of hours all employees actually worked during the year. **Juaind, ackhm, (2004)**.

The formula used for this evaluation by the U.S. Bureau of Labor Statistics, Department of labor is:

INCIDENCE RATE = (NUMBER OF INJURIES AND ILLNESSES *200000) / EMPLOYEE HOURS WORKED (3)

(The 200,000 hours in the formula represents the equivalent of 100 employees working 40 hours per week, 50 weeks per year, and is the standard base for incidence rates).

III. FIELD QUESTIONNAIRE

The field questioner represents the first section of the field part of this research by which the necessary information has been gathered from a selected sample.

The object behind the field questionnaire is to identify the actual safety state for the Iraqi contracting companies, the means and methods used for this purpose and determining the strength and weakness points in their control process in safety in order to set boundaries of the proposed safety system which will satisfy the requirements of the construction sector in Iraq in this regard.

STATISTICAL ANALYSIS FOR QUESTIONNAIRE ANSWERS:

The questionnaire questions divided in to five type each of them represent evaluation limit and it is (always , often ,some time ,seldom , no)and degree of evaluation of each one be evaluate by Table(1) **Dilshad , (2006):**

Table1.
Evaluation range and degree of evaluation of each question

Answer	no	seldm	Some time	Often	Always
Evaluation limit	0-2	2-4	4-6	6-8	8-10
Evaluation degree	1	3	5	7	9

These Statistical standers used to evaluate and analysis the answers:

First: Mean: is the average of the evaluation answers and it used to analyze each item from parts items. **Dilshad, (2006).**

$$M = \sum_{i=1}^{i=n} XiFi / N \quad \text{..... (4)}$$

M: is the mean of the answers, for questioner item.

Xi: the degree of evaluation of answer for function (i) for questioner item

Fi: frequency answers for faction (i) for questioner item

N: number of answers

Mean analysis for each item from questioner items due to the:

If the (M < 5)……. The evaluation of item is (weakness) …. Must Development

If the (5<M<7) ……. The evaluation of item is (acceptable) …..Wanted development

If the (M> 7) ……..the evaluation of item is (good – very good) …. Desired development

Second :(Conformance ratio) which is used to evaluate each part of questionnaire parts which represented how conformance range prerequisite for each part for ideal condition which is calculate by this question **.Dilshad,(2006):**

Cr = M/Xmax (5)

Cr: modulus of conformance for coordinate

M: the mean for (average) for coordinate items

Xmax: the higher degree of value and be center of function for answers evaluation = 9

The analyses and evaluation of each part of the questionnaire depend on conformance ratio and the range of it between (1-9) and median value can be evaluated like this:

M =5/9 = 0.55 (6)

And the first Quarter value (Qu) can be evaluated as:

Qu = 7/9 = 0.77 (7)

The evaluation for questionnaire parts due to this:

If (cr < 0.55) …the evaluation of part is weakness… (Must developed)

If (0.55<cr <0.77)….the evaluation of part is …. (Wanted development)

If (cr > 0.77) …. The evaluation of part is (good – very good) Desired development

IV. EVALUATION AND ANALYSIS OF QUESTIONNER PARTS

1.0) Part One: Safety Policy

No.	Question	no	seldom	some time	often	always	mean frequency
1	Does the policy clearly state that decisions on other priorities should give due regard to health and safety requirements?	12	3	12	6	7	4.32
2	Does the policy commit the organization to full compliance with all relevant health and safety legislation?	11	6	20	6	7	4.6
3	Does the policy set targets for health and safety performance, including a commitment to progressive improvement?	11	9	16	9	5	3.9

4	Does the policy identify key senior personnel for overall coordination and implementation of the policy?	27	7	8	7	1	2.92
5	Is the policy explained to new employees as part of their training and orientation prior to the commencement of work?	19	12	8	6	5	3.64
6	Are there effective arrangements for reviewing the health and safety policy at least once a year?	18	8	19	3	2	3.52
7	Does the review Arrangement include feedback from employees at all levels?	22	8	9	5	1	3
8	Are the revisions, where elevant, promptly brought to the attention of all employees?	20	12	8	5	5	3.52

Analysis of Part One Safety Policy :

Conformance ratio for this part was (cr =0.409), and it is less than what it required, and it is mean the safety policy of the companies does not have all the axis of the safety, and to improve it we need to make safety in the first priority when making decision and set targets of health and safety performance include progressive improvement, and explain safety policy to all new employee.

2.0) Part Two: Safety Organization

No.	Question	no	seldom	some time	often	always	mean frequency
1	Is there an organizational chart showing the names and positions with responsibility lines for safety performance management?	27	10	8	2	3	2.76
2	Have the individual health and safety responsibilities of all employees been clearly defined?		9	8	7	6	3.8
3	Are there arrangements to collect and review feedback on health and safety matters?	25	12	7	4	2	2.84
4	Have sufficient competent safety officers and safety supervisors been appointed and engaged for the site?	30	12	2	2	4	2.52
5	Is someone in charge of updating health and safety information, including changes to regulations, new codes of practice, newly identified hazards, and new work practices?	30	11	2	2	5	2.64

Analysis of Part Two Safety Organization

The conformance ratio for this part was (cr = 0.323) ,and it is less than the acceptable ratio which equal to (0.55) , and the rate of the conformance ratio lead us to know that safety organization is weakness , and to improve it we need to make an organization chart show the names and positions and with responsibility lines for safety performance and make a person responsible for updating health and safety information ,including new codes of practice ,new identified hazards ,and new work practices

3.0) Part three: Safety Training:

No.	Question	no	seldom	some time	often	always	mean frequency
1	Is there a health and safety training plan and is it reviewed regularly?	28	6	10	4	2	2.84
2	Have all workers received basic general safety training?	32	4	6	8	0	2.6
3	Have all workers received site-specific safety training?	30	8	11	1	0	2.32
4	Have all workers received tool-box training related to the tasks?	25	4	9	7	5	3.52
5	Is the effectiveness of health and safety training monitored by checking the new skills applied?	30	7	8	5	0	2.52

Analysis of Part Three Safety Training

The conformance ratio for this part was (cr =0.306), and it is less than what is required ,this ratio lead us to know that safety training is very weak and poor , so to develop it we need to make health and safety training plan and implemented it ,and make safety training for all employees ,engineers, foremen ,worker ,and other level of employees in the companies .

4.0)Part Four: Program for Inspecting Hazardous Conditions:

No.	question	no	seldom	some time	often	always	mean frequency
1	Are there appropriate arrangements to monitor the effectiveness and thoroughness of the inspection?	18	15	11	3	3	3.32
2	Do safety officers and safety supervisors carry out safety inspections at regular intervals?	22	7	16	4	1	3.2
3	Are there appropriate arrangements to ensure that action is taken as a result of the findings of safety inspections?	24	10	9	5	2	3.04
4	Are there appropriate arrangements to collate and analyze the results of safety inspections?	26	12	7	4	1	2.68

Analysis of Part Four Program for Inspecting Hazardous Conditions

The conformance ratio for this part was (cr =0.34), and it is less than the acceptable ratio, and this ratio lead us to know that the program for inspection hazards conditions is poor and weakness, and to improve it we need to make a safety inspection at regular intervals by the safety officer and safety supervisor, and to analyze the result of inspection to know where is the leak and to improve it.

5.0) Part Five: Safety Coordination of the Owner

1. What is the major priority (value) of your company?	chose
on time delivery	29
Low cost	12
Conformance	8
Safety	1

2.Does your firm include safety as part of the project performance review?	no	seldom	some time	often	always	mean frequency
	15	15	11	7	2	3.64

3. When, at the earliest, does your firm begin to emphasize safety on projects?	
During the concept and feasibility phase.	1
During the design phase.	0
Before the start of bidding.	0
During the bidding phase.	3
After bidding, before starting site work.	32
After the start of site work.	5
Other (please specify):	9

4. Does your firm assign full-time safety representative(s) to the construction Projects being built ?	No	seldom	some time	often	always	mean frequency
	32	7	5	3	3	2.5

5. Which of the following does the owner's site representative/manager attend?	
Safety orientations.	5
Site safety meetings.	6
Site safety audits.	25
None of them.	14

6. As the owner, does your firm make any allocation (personnel or	No	seldom	some time	often	always	mean frequency

financial) for the training of the contractor's employees						
	35	9	5	0	1	1.92

Analysis of Part Five Safety Coordination of the Owner

The conformance ratio for this part was (cr = 0.299),and it is the less one of all coordinate in the questioner ,and this ratio make us to know that the safety coordination of the owner is very weak and poor ,and this condition need to improve it by ,include safety as part of the project performance review ,and make the owner and the firm to emphasize safety on projects on all the phases of the work, and to make safety representative for all projects and making him to attend all the safety orientation and site safety meetings and site safety audits .

V. CONCLUSIONS, RECOMMENDATIONS

1. The construction industry has performed much worse than the average of all industries.
2. The project manager must pay more attention to the factors influence on construction safety including the historical, economical, psychological, technical, organizational and environmental issues are considered in terms of how these factors are linked with the level of site safety.
3. Techniques for measuring contractor safety performance are including; Experience modification rating (EMR), Occupational safety and health administration (OSHA) recordable incidence rate, the frequency measure, and a severity measure.
4.The absence of clear understanding of the meaning, principle objective, requirements and methodology of safety performance in construction projects ,both by the construction managers and the main and sub contractors
5. The Conformance ratio for safety policy part was (40 %), and that's mean absence of safety policy for most of the Iraqi contracting companies.
6. The Conformance ratio for safety organization part was (32 %), so this ratio applying that the Iraqi contracting companies does not implement the items for this part.
7. The Conformance ratio for safety training part was (30 %), and that's mean absence of safety training or safety training is very weak and poor for Iraqi contracting companies.
8. The Conformance ratio for program for inspection hazardous conditions part was (34 %), this ratio explains the weakness of program for inspection hazardous conditions if it found for Iraqi contracting companies.
9. The Conformance ratio for safety coordination of the owner was (29 %), this ratio is the less one of all parts of questioner, and this ratio makes us know that the safety coordination of the owner is very weak and poor.
10. There is no motivation for the contractors by Iraqi contracting companies to follow the scientific and a methodic procedure in managing the work with the safeties methods to get a good workplace environment.
11. The absence of safety plan by the contractors in the construction projects.
12. The absence of safety department in most Iraqi contracting companies.

VI . THE RECOMMENDATIONS

1. The construction industry in Iraq should have uniform safety regulations and codes .There should be specific procedures applicable to industrial construction in practice and construction industry in general.
2. The safety department is headed by Safety Director. The Construction Safety Manager, Safety Supervisor, foremen and technicians are the other key personnel in the safety department in all Iraqi contracting companies should be established.
3. Contractor should maintain his own safety management program; it should describe in detail the safety organization, hazard control details, management support and directions, environmental rules & regulations, worker education, and the company medical and first-aid plans.

REFERENCES

[1] Business Roundtable, "Improving Construction Safety Performance: A Construction Industry Cost Effectiveness Project Report," Report A-3, January 1982, New York, NY.

[2] Dilshad 2006, "Development of Management System for Engineering Insurance for Construction Projects Execution in Kurdistan – Iraq", A thesis Submitted to the Department of Civil Engineering at Al- Mustansirya University in partial of fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Construction Management).

[3] Dave Smith & Company. 2007 Update: Experience Modification Still A Poor Current Safety Measurement ("Experience Modification: Problems and Pitfalls") PROFESSIONAL SAFETY, September 1997: 30-31.

[4] Juaind, ackhm. May, 2004. a model for benchmarking contractors project management elements in Saudi Arabia .a thesis presented to the deanship of graduate studies king fahd university of petroleum& minerals Dhahran ,Saudi Arabia In Partial Fulfillment of the Requirements for the degree master of science in construction engineering &management .

[5] Oxford dictionary, oxford university press, second edition 2006, printed in china.

[6] Tauha Hussain Ali, **2006**," Influence of National Culture on Construction Safety Climate in Pakistan" A thesis submitted to School of Engineering Faculty of Engineering and Information Technology Griffith University .Gold Coast Campus.

Causes of Delay on Large Building Projects in Qatar

Hassan Emam, Peter Farrell, and Mohamed Abdelaal

Abstract— **The construction industry is of high importance to the economy of most countries. However, it is notorious for projects overrunning time and cost. Several studies have been conducted to define causes of delay in completing construction projects. This research has employed a wide variety of analytical methods to conclude the most precise statistical ranking of causes of delay. Moreover, the delays for construction projects differ from one country to another and even between types of project within the same geographic location. The aim of this study is focused on identifying the causes of delays in large building projects in Qatar. A comprehensive literature review was carried out in neighbouring Gulf countries. The causes of delays are identified from literature and assessed in exploratory interviews with industry experts in Qatar to investigate the relevance of each cause. A survey questionnaire was prepared and was subject to pilot interviews prior to issuing it to practitioners, including clients, consultants, and contractor organisations. Results reveal that the top five factors causing delay to large building projects are: slow decision-making; discrepancies between specifications and drawings; major changes in design during construction; delay in the settlement of contractor claims; and unreasonable project time frames.**

Index Terms— **Construction Management, Delay Causes, Project Management.**

I. Introduction

PROJECTS in many industries fail to achieve their goals. The construction sector is one business that is notorious for continuous failures to meet time and budget constraints. Whilst, construction is a major contributor to economies across the globe, it can be a source of waste of scarce resources. The effect of construction contribution to economies depends on levels of development in countries; where there are higher levels of development, the lower the construction contribution to economic growth. Some parts of the world are currently undertaking massive developments such as Gulf Cooperation Council (GCC) countries, where lots of construction activity is progressing and planned for the future in fields such as infrastructure, housing and oil and gas. Qatar is a good example of building projects booming due to several factors such as the award of the FIFA World Cup and the National Development Strategy contained in Qatar's National Vision 2030. Construction for the World Cup involves massive projects related to stadia, hotels, infrastructure, and transport systems. It has been announced that $205 billion investments are planned for five years i.e. between 2015 and 2020. This considerable investment in construction projects requires that there is an understanding of challenges by analysing historical reasons for delay and by investigating earlier projects in order to plan to avoid such causes.

In this study, it is intended to investigate the causes of delay in building projects which are a considerable share of construction activities within Qatar in particular and GCC countries in general. The objectives of this research are to: (1) understand the factors that contribute to building project delays in GCC countries and in particular to those related to Qatar; (2) examine and validate the importance of these factors in Qatar's context; (3) propose changes that can reduce the effect of the most significant factors to delay on future projects; and (4) recommend a future research roadmap to enhance project management of construction projects.

II. Literature Review

The systematic quantitative literature review results showing a number of studies in each country are mapped with project types in Table 1. It was found that four studies investigated contributors to building project delay. In this section relevant studies are going to be explored; for a comprehensive literature survey of delay causes in GCC countries, refer to the work be Emam *et al.*[1].

Building projects have been subject to numerous studies within GCC countries due to the increase in volume of work. It was found that four studies with a particular focus on building projects in Kuwait, Qatar and Saudi Arabia were conducted. The country with the most studies in relation to building projects is Kuwait with two studies (Al-Tabtabi[2]; Koushki et al.[3]). Al-Tabtabai[2] dedicated his study to governmental building and housing projects in Kuwait. The study identified 53 causes of delays; the significances of causes were investigated by conducting an on-line survey questionnaire with 48 participants. Surveyed results were analysed using

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Hassan Emam is with the Department of Construction Management, University of Bolton, Deane Road, Bolton, BL3 5AB, UK (phone: +974-7725-2522; e-mail: he2mpo@bolton.ac.uk).

Peter Farrell is a Reader in Construction Management with the Department of Construction Management, University of Bolton, Deane Road, Bolton, BL3 5AB, UK (e-mail: P.farrell@bolton.ac.uk).

Mohamed Abdelaal is with the Department of Construction Management, University of Bolton, Deane Road, Bolton, BL3 5AB, UK (e-mail: maa4mpo@bolton.ac.uk)..

relative importance which suggested that delays are attributable to poor project management, client administration and bad site supervision practices. Koushki et al.[3] studied reasons for delay on private residential buildings in Kuwait. The study used a qualitative approach where 450 personal interviews were conducted with owners of private housing projects. The study concluded that three major factors contribute to delay on this particular type of project thus: lack of owner experience, financial constraints, and changes to design requirements and specifications.

In the context of Qatar, Jurf and Beheiry[4] studied delay contributors to residential compound projects. These complex projects have a specific nature of repetitive typical units. The study was conducted by surveying 20 industry experts and the collected data were analysed using relative importance to identify factors influencing schedule and cost at different project phases. The conclusions on causes of time over-run were delay in material deliveries, design changes, labour unavailability, and inaccurate estimates at tenders.

TABLE 1 RESULTS OF QUANTITATIVE LITERATURE REVIEW (EMAM ET AL.[1])

	Bahrain	Kuwait	Oman	Qatar	Saudi Arabia	UAE	Total
Buildings	-	2	-	1	1	-	4
Construction	-	-	1	-	4	3	8
Oil & Gas	-	-	1	-	1	1	3
Pipeline	-	-	-	-	1	-	1
Road	1	-	-	-	-	-	1
Infrastructure	-	-	-	-	-	1	1
Total	1	2	2	1	7	5	18

Assaf et al.[5] explored causes of delays on large building projects in the Eastern Province of Saudi Arabia. The study identified 53 causes of delay and used them to administer a survey to 48 participants representing owners, consultants and contractors. Responses were analysed using relative importance and rank agreement. The results showed a high degree of agreement between contractors and consultants and lower level of agreement with owners. The most significant factors influencing project delays were found to be: shop drawings preparation and approval processes, poor progress of contractors, late payment by owners, and negative cash flow during construction phases.

III. METHODOLOGY

This study follows a mixed research method; it blends qualitative and quantitative techniques to validate outcomes. The first stage in achieving the objectives of the study comprised an in-depth literature review for neighbouring countries that have similar characteristics to Qatar. In order to ensure a comprehensive survey of literature, it was decided to adopt the systematic literature review methodology developed by Pickering and Jason[6]. The methods commence by identifying search keywords that are then executed in numerous scientific databases. The search results are filtered for their relevance to the study. Related studies are used to determine additional resources that were not found by keywords combinations from the first search. Other keywords are identified from the found papers and applied to search databases. This process iterates until no further studies are found. Relevant papers are reviewed and critically evaluated; process illustration is shown in Figure 1.

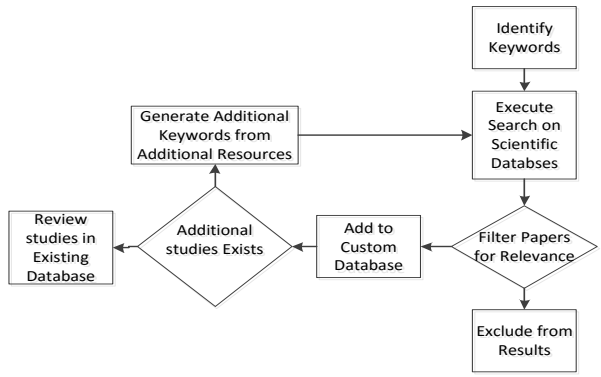


Fig. 1. Systematic Literature review inspired from Pickering and Jason[6].

The literature review identified causes of delays to construction projects in similar studies conducted in neighbouring countries. These defined reasons for delay were used for investigating their relevance to Qatar by means of semi-structured interviews with four construction professionals. The qualitative method was selected to validate defined causes of delay and their importance to Qatar to gain more in-depth explanation and additional factors that are limited to Qatar. Upon the validation of contributors to construction delays, an on-line survey questionnaire was administered to construction professionals involved in building projects within Qatar. The survey was sent to a random sample of 190 construction professionals; 31 people completed the survey with a response rate of 16.3%. It is acknowledged that client representation within the sample is limited, and does not allow divergence to population means.

TABLE 2 PARTICIPANTS' DISTRIBUTION

	Client	Consultant	Contractor	Overall
Participants	2	9	20	31
Percentage returned	6.5%	29.0%	64.5%	16.3%

A. *Survey Design*

The survey was designed in two main parts. The first part collected personal, professional and project information about participants. The gathered information provides an understanding of the participants as the location of the organisation in relation to the supply chain, the seniority level within the organisation, and their experience. The project related section collects data on the size of the project, procurement arrangement, contractual agreement, and project

type. The second part of the survey targets obtaining frequency and severity information from participants about 88 identified causes of delay in Qatar. Factors were clustered into four different groups to corresponding sources of delay i.e. contractors, consultants, clients, and external factors. The frequency is measured in the second part by means of a Likert-scale with the possible selections: always coded as 4, often 3, sometimes 3, rarely 1 and never 0. Meanwhile, the severity information is obtained by impact scale with the following choices; very high coded as 4, high 3, moderate 2, low 1, and very low 0.

B. Data Analysis

The statistical survey analysis technique and indices used in this study were severity index, frequency index and importance index as follows:

Frequency and severity indices formulas are used for the purpose of ranking causes of delays based on their frequency as selected by participants

$$(F.I.)(\%) = \sum a\left(\frac{n}{N}\right) * \left(\frac{100}{4}\right) \quad (1)$$

$$(S.I.)(\%) = \sum a\left(\frac{n}{N}\right) * \left(\frac{100}{4}\right) \quad (2)$$

where ‘a’ is, the constant expressing weighting given to each response (ranges from 0 to 4), ‘n’ is the frequency of the responses, and ‘N’ is total number of responses. The relative importance index of each individual cause is calculated from the result of multiplying the frequency index (1) and severity index (2) as shown in formula (3).

$$(IMP.I)(\%) = \frac{(F.I.)(\%) * (S.I.)(\%)}{100} \quad (3)$$

C. Rank Correlation

The Spearman’s rank correlation is used to measure the level of agreement or disagreement of each two parties based on the importance index. The used formula Eq. (4) shows the calculation method of the correlation factor.

$$r_s = 1 - \left[6 * \sum \frac{d^2}{(n^3 - n)}\right] \quad (4)$$

where ‘rs’ is Spearman’s rank correlation coefficient between two parties; ‘d’ is the difference in rank assigned to variables of each cause; n is the number of ranks.

IV. Results and Discussion

The on-line questionnaire responses were analysed and results are reported in this section. Causes of delay are grouped into four categories according to attributable source i.e. clients, consultants, contractors, and external factors.

The primary factors due to contractors are improper technical study during bidding, delay in materials delivery and changes to material specifications. These factors were discussed with construction experts during interviews to gain in-depth understanding. It was stated that due to time constraints during tender stages, contractors tend to carry out rough estimates based on previous experiences within Qatar. This might lead to unexpected conditions that were overlooked during tender stages such as ground conditions, underground water levels, and unconsidered complexity in executing tasks that were considered typical during the estimation phase. The consequences of inadequate technical studies are under-estimated schedule provisions and unavailability of required resources for performing works. Subsequently, this will contribute to interrupted project activities, failure to arrange proper resources or will result in elongated durations for scheduled tasks. Materials delivery delays were argued to be attributable to two main causes thus; limited port capacity in Doha, and lack of local manufacturing facilities which compels suppliers to purchase materials from abroad. The last factor of delay associated with contractors is deviating from materials specification where alternatives are proposed for several reasons including cost savings, value engineering and immediate availability of substitutes. According to experts, often these deviations involve extended periods of approvals that lead to losing time even in case of local availability. Whilst these factors are of high importance, they are ranked of lower urgency than those of consultants and clients.

TABLE 3: TOP CONTRACTOR FACTORS CAUSING DELAY

Description	Frequency %	Impact %	RII %	Overall Rank
Improper technical study during the bidding stage	0.70	0.81	0.57	7
Delay in materials delivery	0.69	0.81	0.56	10
Changes in materials specifications	0.67	0.81	0.55	12

Consultants were criticised for causing project delays with a considerable emphasis on their contributions to design components such as discrepancies between specifications and drawings, changes to design during construction stages, and delay in solving design problems. Lack of coordinated designs was considered the factor of highest influence to project delay by designers. These discrepancies were argued to be resultant of geographic spread of designers in different offices around the world. In addition, there was slow adoption of new collaboration tools such as building information modelling (BIM) as reported by Ahmed et al.[7] in a survey conducted on barriers and usages of this technology in Qatar. Interviewees stressed that changes in construction projects are inevitable but the timing of these changes cause undesirable impacts such as schedule over-runs and budget excesses. There is a correlation between discrepancies and design changes along with owners’ limited knowledge of construction. The third cause of delay by designers is slow resolution of design problems. Interviewed professionals emphasised the possibility to mitigate effects of design coordination by using the state-of-art technologies for communication and collaboration such as BIM.

TABLE 4: TOP CONSULTANT FACTORS CAUSING DELAY

Description	Frequency %	Impact %	RII %	Overall Rank
Discrepancies between specifications and drawings prepared	0.76	0.83	0.63	2
Major change of design during construction	0.71	0.88	0.62	3
Delay in solving design problems	0.71	0.80	0.57	8

Survey participants ranked three factors attributable to clients from the top ten factors causing delay to construction project completion. These factors are commercial and planning aspects of projects. The top clients' contributor is slow decision-making; this is linked to bureaucratic processes of decision-making which is due to lack of delegation and evading decision-making and associated accountabilities to individuals. Interview participants emphasised the consequence of the deliberate postponement of claims resolutions to the back-end of projects. These determinations are usually subject to amicable settlement as a part of a 'deal' at project completion. Unreasonable project durations was reinforced as a re-occurring issue across different developers in Qatar. The time frames are set to be considerably less than reasonable length to complete works. Contractors participate in tenders and submit offers knowing that completion dates will not be met and depend upon extension of time and completion claims to avoid penalties associated with late project handovers.

TABLE 5: TOP CLIENT FACTORS CAUSING DELAY

Description	Frequency %	Impact %	RII %	Overall Rank
Slow decision-making	0.73	0.87	0.63	1
Delay in the settlement of contractor claims	0.73	0.82	0.60	4
Unreasonable project time frame	0.69	0.85	0.59	5

A. *Groups Ranking*

The four main groups were ranked to identify influences of groups to delaying projects. It was observed that consultants related causes were ranked as the highest influence to delaying projects completion. This is attributable to dependencies on design activities and their subsequent knock-on effect on succeeding tasks such as procurement and construction.

TABLE 6: GROUPS RANKING

Group	RII	Rank
Consultant Delays	0.44	1
Client Causes	0.43	2
Contractors Delay	0.36	3
External	0.28	4

Client related causes group was ranked second due to the significance of financing, cash flow and decision-making. These are inter-related to procuring materials and ability to paying suppliers and subcontractors to ensure works' continuity.

The second section of the survey was divided into 15 categories of questions. These categories were ranked and results support design importance discussion. The categories and their ranking are reported in Table 7.

TABLE 7: CATEGORIES RANKS

Categories	RII	Rank
Consultant Design	0.54	1
Government Related	0.48	2
Client Finance	0.43	3
Client Contract	0.43	4
Reviews and Approvals	0.43	5
Client Management	0.42	6
Contractor Material	0.39	7
Contractor's Management	0.39	8
Contractor Manpower	0.38	9
Consultant Management	0.36	10
Consultant Personnel	0.36	11
Contractor Finance	0.34	12
Site Conditions	0.25	13
General	0.19	14
Equipment	0.17	15

B. *Rank Agreement*

Rank correlation using Spearman's correlation test was conducted to explore the level of agreements between stakeholders on categories of delay causes. The reported results in Table 8, shows a high level of agreement between clients and consultants. However, investigating correlations between contractors and both consultant and clients revealed a lower level of agreement on ranking categories. These findings are in line with Assaf et al.[5] that concluded the same and supports the argument about adversarial relationships between clients or consultants and contractors on the other side.

TABLE 8: SPEARMAN'S RANK AGREEMENT

	Spearman's Coefficient
Clients - Consultants	0.70
Clients - Contractors	0.15
Consultants - Contractors	0.39

C. *Comparative Study*

A comparative study with the only published paper on causes of delay in Qatar was conducted. The results of the analysis are reported in Table 8. The variances in ranks are explainable due to different factors between residential complexes and large buildings projects such as: owner types, value and complexities associated with projects, and specification of required materials. The owners of large projects are more sophisticated organisations with more complex and bureaucratic structures that affect decision-making swiftness. However, compound developers are either small companies or even individuals with full decision-making

authority. This explains the variance between the two studies with regard to slow decision-making variances. Discrepancies between specification and drawings are of higher importance in large building than compounds due to the difference in complexity. Housing complexes are usually typical houses with repeating construction activities which reduces risks of contradictory information. Variances related to delays to claims settlement and issuing change orders late are consequences of decision-making swiftness. The larger effect of materials late delivery in large projects is attributable to their specifications which often require overseas purchasing to fulfil requirements. It was observed that changes in scope has higher importance to compound projects; this was further investigated and results revealed that compounds that are usually possessed by individuals are subject to continuous changes based on their personal preferences. In addition, owners of such compounds often have limited or no construction knowledge which causes contradictory and continuous changes to project scope without acknowledgement to time or associated cost.

TABLE 8: COMPARATIVE STUDY WITH JURF AND BEHEIRY[4]

Description	Rank	Jurf and Beheiry[4]	Rank Change
Slow decision-making	1	15	14
Discrepancies between specifications and drawings prepared	2	9	7
Major change of design during construction	3	3	-
Delay in the settlement of contractor claims	4	10	6
Unreasonable project time frame	5	8	3
Delay in issuance of change orders	6	11	5
Improper technical study during the bidding stage	7	8	1
Delay in solving design problems	8	9	1
Changes in the scope of the project	9	3	6
Delay in materials delivery	10	5	5
Delay in issuing the drawings	10	12	2

V. CONCLUSION

The aim of this study was to investigate significant factors contributing to delays of large building projects in Qatar. The earlier literature studying neighbouring countries were reviewed to identify common reasons for delay. A quantitative systematic literature review was adopted to ensure inclusion of all studies on GCC countries and identify gaps. The process concluded by defining 120 causes of delay that were filtered to 88 relevant to Qatar. The identified factors were administered in an on-line survey that was responded to by 31 participants from various stakeholders. The results were analysed using statistical tools for ranking, rank agreement, and compared with an earlier study in Qatar. The top five factors were: slow decision-making; discrepancies between specifications and drawings prepared; major change of designs during construction; delay in the settlement of contractor claims; and unreasonable project time frames. The study also concluded that there are significant differences between contractors' views and both clients and consultants which reinforce interview outcomes of the adversarial relationship between these stakeholders. On the contrary, clients and consultants reflected an acceptable level of agreement. It is acknowledged that number of participants from client organisations does not allow results to be generalised.

Future research can potentially focus on other projects types such as utilities and oil and gas. Also, focus on study factors that cause variations of causes of delay such as client organisation types, contract arrangements, project types, and geographical locations. These research areas will help decision-makers to realise root causes and instigate corrective action to mitigate their impact.

REFERENCES

[1] Emam, H., Farrell, P. and Abdelaal, M.: Causes of Delay in GCC Construction Projects: A Critical Review. In: Okeil, A (Ed.) The 1st International Conference of CIB Middle East & North Africa Conference, Abu Dhabi, UAE. 2014, pp. 607^621.

[2] Al-Tabtabi, H.: Causes for Delays in Construction Projects in Kuwait. "Engineering Journal of the University of Qatar", 2002, 15, pp. 19^37.

[3] Koushki, P, Al-Rashid, K, and Kartam, N.: Delays and Cost Increases in the Construction of Private Residential Projects in Kuwait. Construction Management and Economics, 2005, 23, No. 3, pp. 285^294.

[4] Jurf, N and Beheiry, S.: Qatar Residential Construction Projects. In: Mohamed, S (Ed.) 2nd International Conference on Engineering System Management and Applications, 30 Mar - 01 Apr 2010, Sharjah, UAE. IEEE, 2010, pp. 1^6.

[5] Assaf, S., Al-Khalil, M. and Al-Hazmi, M.: Causes of Delay in Large Building Construction Projects. Journal of Management in Engineering, 1995, 11, No. 2, pp. 45^50.

[6] Pickering, C. and Jason, B.: The Benefits of Publishing Systematic Quantitative Literature Reviews for PhD Candidates and Other Early-Career Researchers. Higher Education Research & Development, 2014, 33, No. 3, pp. 534^548.

[7] Ahmed, S., Emam, H., and Farrell, P.: Barriers to BIM/4D Implementation in Qatar. In: Okeil, A (Ed.) The 1st International Conference of CIB Middle East & North Africa Conference, Abu Dhabi, UAE. 2014, pp. 533^547.

Hassan Emam is a planning and scheduling professional with a civil engineering background and Master of Business Administration in Construction and currently he is a PhD candidate at the University of Bolton. He has diversified experience in the construction industry on a variety of projects such as residential, commercial, airports, bridges, marine, and railways.

Peter Farrell is a Reader in Construction and Commercial Management at the University of Bolton. He has industry experience in planning and quantity surveying, whilst most of his post-qualification work was as a construction site manager.

Mohamed Abdelaal is a planning and scheduling professional with a civil engineering background and certified project management profession. He has various experience in the construction industry on a variety of projects such as residential, commercial, airports, bridges, road works, marine, treatment plants and infrastructure.

Sorptivity index of self-compacting concretes with nano-silica and fly ash

Asraa Al-Goody, Erhan Güneyisi, Mehmet Gesoğlu, and Süleyman İpek

***Abstract*—This experimental study covers the investigation of effect of nano-silica and fly ash utilization on the sorptivity index of self-compacting concrete. Therefore, the total binder content and water-to-binder ratio were kept constant in all self-compacting concrete mixtures. Four different self-compacting concrete series were designed with a total binder ratio of 570 kg/m^3 and at a water-to-binder ratio of 0.33. Both nano-silica and fly ash were replaced with cement by weight. In all self-compacting concrete series, the nano-silica was incorporated to concrete with replacement levels of 0%, 2%, 4%, and 6%. First self-compacting concrete series were produced without fly ash whereas second, third, and forth self-compacting concrete series were produced at fly ash replacement levels of 25%, 50%, and 75%, respectively. In the total, 16 self-compacting concrete mixtures blended with nano-silica and fly ash were produced and they were tested for the sorptivity index at the age of 28 and 90 days. The results indicated that utilization of nano-silica enhanced the sorptivity index of self-compacting concrete mixtures at both ages. The sorptivity indexes of mixtures at both ages were improved by fly ash replacement level at 25% and 50%.**

***Index Terms*— Fly ash, Nano-silica, Permeability, Self-compacting concrete.**

I. Introduction

THE concrete is the man-made construction material that has the largest utilization all over the world. This fact leads to important problems regarding its design and preparation to finally obtain an economical cost of the product on short and long time periods. As a result of this, high performance concretes developed having a superior mechanical and durability characteristics. Self-compacting concrete (SCC) is an innovative concrete that does not requires any vibration for placing and compaction. It has ability to flow under its own weight, completely filling formwork and achieving full compaction, even in the presence of congested reinforcement. The hardened concrete is dense, homogeneous and has superior mechanical properties and durability characteristic than traditional concrete. In order to achieve the SCC with high fluidity and to prevent the segregation and bleeding during transportation and placing, the formulators have employed a high portland cement content, low water-to-binder ratio and used superplasticizer and viscosity modifying additives [1–4]. However, cost of such concretes remarkably increased associated with the use of high amount of portland cement and chemical admixtures. In some cases, the savings in labor cost might offset the increased cost. But the use of mineral admixtures such as fly ash, blast furnace slag, and/or limestone filler reduces the cost of the SCC and also improved fresh and hardened properties of the SCC [5,6]. A number of studies have been reported in the literature concerning the use of mineral admixtures to enhance the self-compatibility and durability characteristics of the SCCs [7].

In addition to the increasing in the cost of the SCC due to using a high amount of cement in the production, some problems such as high hydration heat and high autogenously shrinkage can be introduced. Moreover, serious environmental impacts can be occurred due to the natural resources consumption and carbon dioxide emissions associated with cement production [8]. The purpose of this paper is to investigate the effects of using nano-silica and mineral admixtures such as fly ash on the sorptivity index of self-compacting concretes. The addition of suitable cementitious materials, mostly siliceous or aluminous, with cement which will react with excess calcium hydroxide ($Ca(OH)_2$) and produce additional calcium silicate hydrate (C-S-H) with the replacement of porous $Ca(OH)_2$ and refines the pore structure and reduces permeability in concrete [9]. The supplementary cementitious materials such as pulverized fly ash, ground granulated blast furnace slag, condensed micro silica fume, rice husk ash, metakaoline etc. have been studied extensively in concrete as pozzolanic materials to enact the $Ca(OH)_2$ and get the additional C-S-H. The addition of supplementary cementitious materials in the concrete will not only improve the workability and mechanical properties of concrete but also its durability characteristics.

At this juncture, the researchers are capitalizing on nano technology to innovate a new generation of concrete materials that overcome the above drawbacks and trying to achieve the sustainable concrete structures. Evolution of materials is need of the day for improved or better performance for special engineering applications and modifying the bulk state of materials in terms of composition or microstructure or nanostructure has been the established route for synthesizing new materials. With the advancement of nano technology, nano materials have been developed that can be applied to concrete mix designs to study the physical, chemical and enhanced mechanical properties of concrete. Among the various developed or manufactured nano materials such as nano-silica, nano-alumina, nano-titania, nano-zirconia, carbon

nano tubes or wires etc., the addition of nano-silica enhances the possibility for the reaction with $Ca(OH)_2$ to develop more strength carrying structure of cement: C-S-H and also pore filling effect of nano silica in the concrete [10–12].

Permeability is the most important aspect of concrete durability. The concrete must be relatively impervious if the durable concrete is aimed. In general, lower permeability means greater durability [13]. Permeability of concrete is governed by many factors such as the amount of cementitious material, water content, aggregate grading, consolidation, and curing. Through its pozzolanic properties, fly ash chemically reacts with $Ca(OH)_2$ and water to produce secondary C-S-H gel [14]. The $Ca(OH)_2$ is consumed in the pozzolanic reaction and is converted into a water-insoluble hydration product [15]. The transformation of large pores to fine pores, as a result of the pozzolanic reaction between portland cement paste and fly ash, substantially reduces permeability in cementitious systems [16]. The reduced permeability by the use of fly ash in the concrete production can decrease the rate of ingress of water, corrosive chemicals, and oxygen [14]. This leads to enhanced durability.

Sorption is the water movement driven by capillary action in short-term exposure in partially dry concrete. The rate of water uptake by a porous material is defined as sorptivity. It has been considered as an important criterion to assess the durability of concrete [17]. The pore system of the paste and the interfacial zone has a great influence on sorptivity. The interfacial zone is porous but it is the hardened paste, the only continuous phase in concrete that controls the ingress and transportation of water [18]. The sorptivity of SCCs with up to 40% fly ash were unchanged but were significantly higher in mixes with 60% fly ash [19].

The nano-silica concrete has better permeability resistance than the normal concrete. With advent of supplementary cementitious materials and other siliceous and aluminous materials, today's concrete technology has achieved enormous potential applications, by the way of reduction in cement consumption, enhanced properties and reduced carbon foot print. In concrete, for example, the micro silica fume works in the form of chemical reaction with $Ca(OH)_2$ form more C-S-H gel at final stage and also fill the voids and pores in the fresh and hardened cement paste, thereby increasing the concrete's density.

This study aims to investigate the combined effects of nano-silica and fly ash on the sorptivity characteristic of the SCCs. For this reason, four SCC series were designed with FA contents of 0%, 25%, 50%, and 75%. Portland cement was substituted with NS at levels of 0%, 2%, 4%, and 6%. Moreover, PC was replaced with both FA and NS by weight. Totally, 16 SCC mixtures were designed with a total binder content of 570 kg/m^3 and at a constant w/b ratio of 0.33. The sorptivity index of each mixture was determined and also the results were evaluated statistically and it was noticed that the age of the concrete, nano-silica and fly ash contents had remarkable effect on the sorptivity index of the SCC mixtures.

II. Experimental Study

A. Materials

In this study, portland cement (CEM I 42.5R), class F fly ash, and nano-silica were used to produce the self -compacting concrete. They had the specific gravity of 3.15, 2.25, and 2.20, and the specific surface area of 326, 379, and 150000 m^2/kg, respectively. Physical properties and chemical compositions of the portland cement, fly ash, and nano-silica are presented in Table 1.

TABLE I
PHYSICAL PROPERTIES AND CHEMICAL COMPOSITIONS OF PORTLAND CEMENT, FLY ASH AND NANO-SILICA

Analysis Report (%)	Cement	Fly ash	Nano-silica
CaO	62.58	4.24	-
SiO_2	20.25	56.20	≥99.8
Al_2O_3	5.31	20.17	-
Fe_2O_3	4.04	6.69	-
MgO	2.82	1.92	-
SO_3	2.73	0.49	-
K_2O	0.92	1.89	-
Na_2O	0.22	0.58	-
Loss on ignition	3.02	1.78	≤1.00
Specific gravity	3.15	2.25	
Specific surface area (m^2/kg)	326	379	150,000

The coarse aggregate used in this study was river gravel with a nominal maximum size of 16 mm and the fine aggregate with a maximum size of 4 mm, a mixture of natural sand and crushed sand, was used in the production. The particle size gradation of the aggregate mixture with A16, B16, and C16 curves is presented in Fig 1. A polycarboxylic ether type of superplasticizer with specific gravity of 1.07 was used to achieve the desired workability in all concrete mixtures.

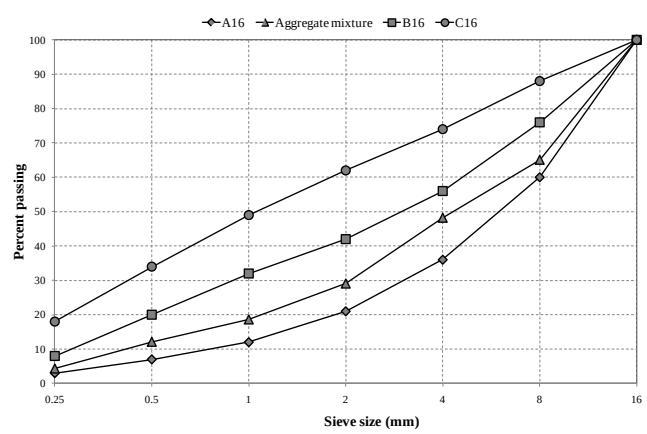


Fig. 1. Sieve analysis of aggregate mixture and A16, B16, and C16 curves

B. Mixture Design

The SCC mixtures were designed with total binder content of 570 kg/m3 and at w/b ratio of 0.33. Four SCC series were produced during this study and in each SCC series; the PC was substituted with NS at 0%, 2%, 4%, and 6% replacement levels by weight. The SCC mixtures in first series were produced without FA while in second, third, and forth series the Portland cement was also substituted with FA at 25%, 50%, and 75% replacement levels by weight, respectively. Totally 16 SCC mixtures were produced according to above variables. The detailed mix proportions for SCCs are presented in Table 2. In mix ID, fly ash is represented by FA while nano-silica is denoted by NS. For example, FA25NS2 indicates that the SCC mixture is designed with fly ash content of 25% and nano-silica content of 2%. The SCC mixtures were designed according to slump flow diameter of 750 ± 50 mm which was achieved by using the superplasticizer at varying amounts. Typical slump flow diameter of the mixtures is illustrated in Fig. 2.

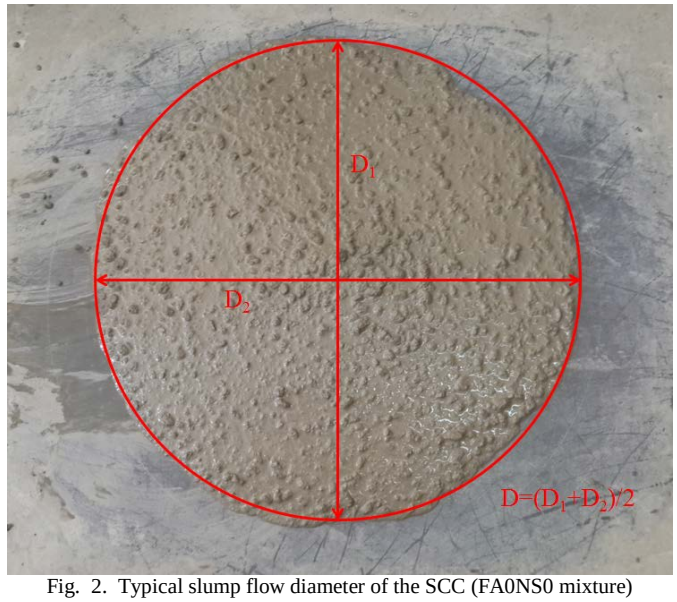


Fig. 2. Typical slump flow diameter of the SCC (FA0NS0 mixture)

TABLE II
MIX PROPORTIONS FOR SELF-COMPACTING CONCRETE (kg/m^3)

Mix ID	water-to-binder ratio (w/b)	Portland cement	Fly ash	Nano-silica	Water	SP*	Coarse aggregate	Fine aggregate	
								Natural sand	Crushed sand
FA0NS0	0.33	570.0	0	0	188.1	3.42	835.5	668.4	167.1
FA0NS2	0.33	558.6	0	11.4	188.1	9.98	825.2	660.2	165.0
FA0NS4	0.33	547.2	0	22.8	188.1	15.68	816.1	652.8	163.2
FA0NS6	0.33	535.8	0	34.2	188.1	21.38	806.9	645.5	161.4
FA25NS0	0.33	427.5	142.5	0	188.1	2.85	812.1	649.7	162.4
FA25NS2	0.33	416.1	142.5	11.4	188.1	7.13	804.7	643.7	160.9
FA25NS4	0.33	404.7	142.5	22.8	188.1	14.25	793.7	635.0	158.7
FA25NS6	0.33	393.3	142.5	34.2	188.1	19.95	784.6	627.7	156.9
FA50NS0	0.33	285.0	285.0	0	188.1	2.28	788.7	631.0	157.7
FA50NS2	0.33	273.6	285.0	11.4	188.1	5.70	782.4	625.9	156.5
FA50NS4	0.33	262.2	285.0	22.8	188.1	12.83	771.4	617.1	154.3
FA50NS6	0.33	250.8	285.0	34.2	188.1	17.10	764.0	611.2	152.8
FA75NS0	0.33	142.5	427.5	0	188.1	1.71	765.3	612.3	153.1
FA75NS2	0.33	131.1	427.5	11.4	188.1	4.28	760.1	608.1	152.0
FA75NS4	0.33	119.7	427.5	22.8	188.1	11.40	749.1	599.3	149.8
FA75NS6	0.33	108.3	427.5	34.2	188.1	14.25	743.5	594.8	148.7

*** SP: Superplasticizer**

C. Concrete Casting

The mixing sequence and duration are very important in the SCC production. For this reason, mixing and batching procedure recommended by Khayat et al [20] was followed in this study in order to obtain the same homogeneity and uniformity in all SCCs. Regarding to this procedure, the fine and coarse aggregates were poured in a power-driven revolving pan mixer and allowed to mix homogeneously for 30 seconds. After that about half of the mixing water was added into the mixer and it was allowed to proceed the mixing for one more minute. The aggregates, then, were left to absorb the water for 1 minute. Afterwards, the powder materials (cement and/or fly ash and/or nano-silica) were added to the wetted aggregate mixture for mixing another minute. After SP

with remaining water was poured into the mixer, the concrete was mixed for 3 min and then left to rest for a 2 min. Finally, the concrete was mixed for additional 2 min to complete the production. All specimens were casted without any compaction and vibration. After the concrete casting, all molded specimens Φ100x200-mm cylinders were wrapped with plastic sheet and left in the casting room for 24 h at 20±2 °C and then they were demoulded and 28 and 90-day water curing period was applied. Afterwards, they were tested based on the testing procedures below.

D. Testing Procedure

Sorptivity test which measures the rate of water drawn into the capillary pores of concrete was carried out after the specimens were dried in an oven at 100±5 °C till they reached the constant mass. The sides of the 50-mm disk specimens cut from 100-mm cylinders were coated by paraffin to prevent water suction from the sides of the specimen. The test was conducted on the surface of concrete that was in contact with water as shown in Fig. 3. The specimens were removed from the tray and weighed at different time intervals up to 1 h to evaluate mass gain. The volume of absorbed water was calculated by dividing the mass gained by nominal surface area of the specimen and by the density of water. Then, the square root of time versus these values was plotted and the sorptivity coefficient (index) of concretes was determined by the slope of the line of the best fit.

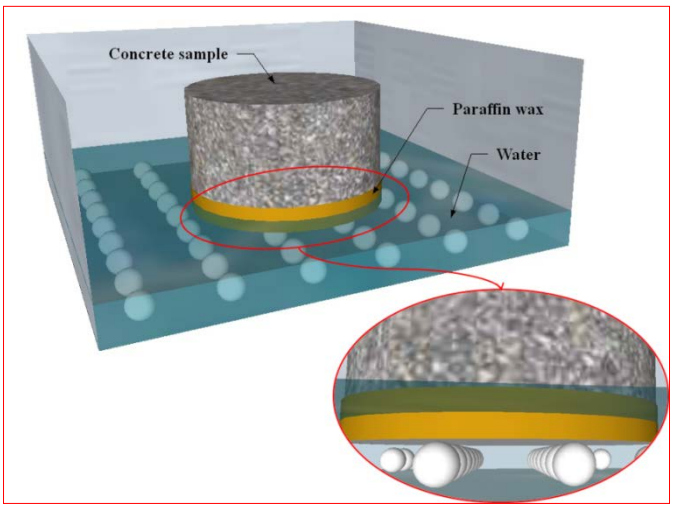
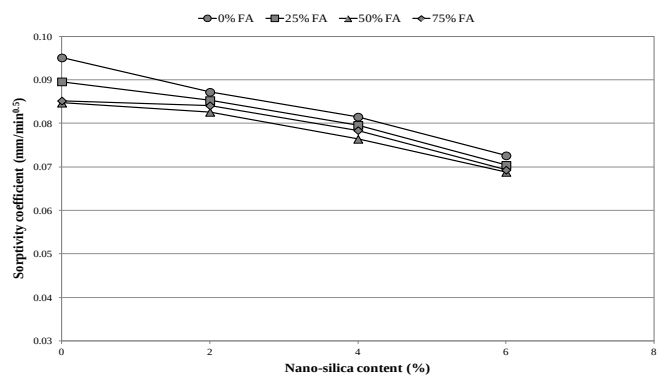


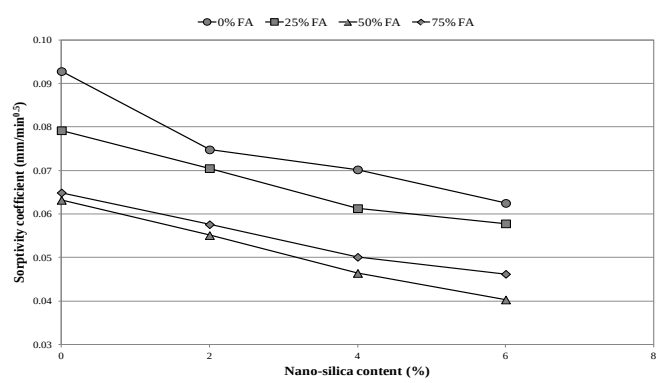
Fig. 3. Detail of sorptivity measurement

III. Results and Discussion

Fig. 4a and 4b presents the sorptivity test results of the SCCs measured at 28 and 90 days, respectively. The sorptivity index value change between 0.0688 and 0.0951 mm/min$^{0.5}$, and 0.0403 and 0.0958 mm/min$^{0.5}$ was measured at 28-day and 90-day, respectively. The results indicated that utilization of nano-silica in the SCC production systematically decreased the sorptivity index of the mixtures at both ages. However, the effect of the nano-silica incorporating on the 90-day sorptivity index was more than than the 28-day index. Namely, the increasing the nano-silica content from 0 to 6% resulted in decreasing of 28-day sorptivity index values as much as 23.7, 21.4, 18.8, and 18.7% in the first, second, third, and fourth SCC series, respecetively, while the decreasing amounts of 90-day sorptivity index values were about 32.7, 27.0, 36.2, and 28.8% in the first, second, third, and fourth SCC series, respectively. This might be due to the secondary reaction between the calcium hydroxide of the cement paste and the silicon dioxide of the nano-silica. In addition to nano-silica enhancement, fly ash utilization was also improved permeability of the SCC regarding to sorptivity index. The sorptivity index values were decreased by increasing the fly ash content till 50%. At 75% replacement level, the small amount of increment was determined. Although there was an increasing in the sorptivity index values at 75% replacement level of fly ash, the values were lower than the values of the SCC that did not include fly ash. As seen from Figure 4a and 4b, the best sorptivity index values of 0.688 and 0.403 were achieved in the FA50NS4 mixture at 28 and 90 days, respectively. It was caused due to balling effect of nano-silica at high replacement level. The nano-silica utilization at high percentage such as more than 4% generally aggravates the permeability properties of concretes due to the fact that nano-silica had high surface area and contains nano scaled particle that had tendency to coagulate when it was used at high replacement level.



(a)



(b)

Fig. 4. Variations in the sorptivity coefficient of the SCCs at: a) 28 days and b) 90 days

In order to find out the statistical significance of the experimental test parameters in a quantitative manner, general linear model analysis of variance GLM-ANOVA was performed at 0.05 level of significance. GLM-ANOVA is an important statistical analysis and diagnostic tool which helps in reducing the error variance and quantifies the dominance of a control factor. In the analysis, the measured sorptivity index for SCCs were assigned as the dependent variables while the concrete age, fly ash content, nano-silica content was selected as the independent factors. GLM-ANOVA was performed and the effective test parameters and their percent contributions on the sorptivity index results were tabulated in Table 3. If the P-value is less than 0.05, the independent variable is accepted as significant factor on the test results. The percent contribution, which defines the degree of effectiveness of the independent factors on the measured property, was also determined. When the percent contribution of one parameter is higher, the effectiveness of that parameter on the measured property is higher. Likewise, if the percent contribution is low, the contribution of the factor to that particular response is less. The statistical evaluation of the test results indicated that all independent variables had remarkable effect on the sorptivity indexes of the mixtures when P-values of the independent variables were considered. Moreover, the statistical analysis revealed that the dependent parameter, sorptivity index, was more influenced by the concrete age when the percent contribution values of independent variables were regarded.

IV. Conclusions

The following conclusions can be drawn according to the aforementioned findings:

- The sorptivity index of the SCCs was affected by both nano-silica and fly ash contents. Especially, the increasing the nano-silica content resulted in systematical reduction in the sorptivity index values. However, the increasing the fly ash content till certain level decreased the sorptivity index. In addition to effect of nano-silica and fly ash contents, the effect of age of the concrete was also investigated and it was revealed that the effect of nano-silica and fly ash was more at later age than at early age.
- The effect of nano-silica and fly ash contents and the concrete age was statistically evaluated and it was seen that all independent parameters, nano-silica and fly ash contents and the concrete age, had significant effect on the sorptivity index of the SCC regarding to P-values. However, the most effective independent parameter was the concrete age according to percent contribution values.

TABLE III
STATISTICAL EVALUATION OF SORPTIVITY INDEX OF THE SCCS

Dependent variable	Independent variable	Sequential sum of squares	Computed F	R-square (%)	P value	Significance	Contribution (%)
Sorptivity index	Concrete age	0.0027733	113.66		0.00	YES	43.4
	NS content	0.0019165	26.18		0.00	YES	30.0
	FA content	0.0011166	15.25	90.84	0.00	YES	17.5
	Error	0.0005856	-		-	-	9.1
	Total	0.0063920	-		-	-	-

References

[1] Sari M, Prat E, Labastire JF. High strength self-compacting concrete: original solutions associating organic and inorganic admixtures. Cement Concr Res 1999;29(6):813–8.

[2] Sakata S, Maruyama K, Minami M. Basic properties and effects of welan gum on self-compacting concrete. In: Bartos PJM, Marrs DL, Cleland DJ, editors. Proceedings of RILEM international conference on production methods and workability of concrete. London: E&FN Spon; 1996. p. 1–24.

[3] Lachemi M, Hossain KMA, Lambros V, Nkinamubanzi PC, Bouzoubaa N. Performance of new viscosity modifying admixtures in enhancing the rheological properties of cement paste. Cement Concr Res 2004;34(2):185–93.

[4] Saric-Coric M, Khayat KH, Tagnit-Hamou A. Performance characteristics of cement grouts made with various combinations of high-range water reducer and cellulose-based viscosity modifier. Cement Concr Res 2003;33(12):1999–2008.

[5] Sahmaran M, Christianto HA, Yaman _IO. The effect of chemical admixtures and mineral additives on the properties of self-compacting mortars. Cement Concr Compos 2006;28(5):432–40.

[6] Bouzoubaa N, Lachemi M. Self-compacting concrete incorporating high volumes of class F fly ash preliminary results. Cement Concr Res 2001;31(3):413–20.

[7] EFNARC, “Specification and Guidelines for Self-Compacting Concrete”, www.efnarce.org , February 2002, pp.32

[8] Li H, Gang H, Jie X, Yuan J, Ou J. Microstructure of cement mortar with nanoparticles.Compos Part B: Eng 2004;35(2):185–9.

[9] Maheswaran S.1, Bhuvaneshwari B.1, Palani G.S.1, Nagesh R Iyer1 and Kalaiselvam S., An Overview on the Influence of Nano Silica in Concrete and a Research Initiative , Available online at: www.isca.in , INDIA 2012

[10] Hou P, Kawashima S, Kong D, Corr David J, Qian J, Shah SP. Modification effects of colloidal nanoSiO2 on cement hydration and its gel property. Composites: Part B 2013;45:440–8.

[11] Zapata LE, Portela G, Suárez OM, Carrasquillo O. Rheological performance and compressive strength of superplasticized cementitious mixtures with micro/ nano-SiO2 additions. Constr Build Mater 2013;41:708–16.

[12] Choolaei M, Rashidi AM, Ardjmanda M, Yadegari A, Soltanian H. The effect of nanosilica on the physical properties of oil well cement. Mater Sci Eng A 2012;538:288–94.

[13] Donald Burden "The Durability of Concrete Containing High Levels of Fly Ash", B.Sc. E., University of New Brunswick, 2003.

[14] ACI Committee 232. (2003). “Use of Fly Ash in Concrete”. ACI Manual of Concrete Practice 232.2R-03.

[15] Joshi, R.C., and Lohtia, R.P. (1997). “Fly Ash in Concrete – Production, Properties and Uses”. Advances in Concrete Technology Volume 2. Gordon and Breach Science Publishers, Printed in India.

[16] Manmohan, D., and Mehta, P.K. (1981). "Influence of Pozzolzinc, Slag, and Chemical Admixtures on Pore Size Distribution and Permeability of Hardened Cement Pastes". Cement, Concrete, and Aggregates, V. 3, No. 1, pp. 63-67.
[17] Ho DWS, Chirgwin GJ. 1996 Jul. A performance specification for durable concrete. Construction and Building Materials 10(5):375-379.
[18] Sabir BB, Wild S, O'Farrel M. 1998. A water sorptivity test for mortar and concrete. Materials and structures 31:568-574.
[19] Miao Liu , Wider Application of Additions in Self-compacting Concrete , A thesis submitted to University College London for the degree of Doctor of Philosophy , Department of Civil, Environmental and Geomatic Engineering, July 2009.
[20] Khayat KH, Bickley J, Lessard M (2000) Performance of self-consolidating concrete for casting basement and foundation walls. ACI Mater J, 97(3):374–80.

Asra Al-Goody, Building and Construction Department, University of Technology, Baghdad, Iraq, e-mail: israayounis@yahoo.com

Erhan Güneyisi, Department of Civil Engineering, Gaziantep University, 27310 Gaziantep, Turkey, e-mail: guneyisi@gantep.edu.tr

Mehmet Gesoğlu, Department of Civil Engineering, Gaziantep University, 27310 Gaziantep, Turkey, e-mail: mgesoglu@gantep.edu.tr

Süleyman İpek, Department of Civil Engineering, Gaziantep University, 27310 Gaziantep, Turkey, e-mail: sipek@gantep.edu.tr

Durability of Different Types of Concrete Exposed to Kerosene

Asst. Prof. Laith Sh. Rasheed University of Kerbala,Civil Eng. Department
laith.alqarawee@uokerbala.edu.iq

***ABSTRACT*,Concrete has a good durability in different natural environment, however, concrete durability drops under effect of petroleum products and the deterioration of concrete tanks of petroleum products is still a problem. So, it's necessary to protect concrete against this effect to increase its service life. The present research is adopted to study mechnical properties and durability of normal weight concrete, self-compacted concrete (SCC), and concrete coated with epoxy, under effect of kerosene with period of subjecting of four months and comparing the test result with concrete cured for 28 days in water as reference mix to show the impact of kerosene. The experimental program include, test of fresh concrete, compressive strength, splitting tensile strength, density, absorption and percentage of voids. The result show bad effect of kerosene on all types of mixes but with less percentage on SCC than other two mixes.**

Key Words: Oil Products, Kerosene, Self-compacted Concrete, Durability, Epoxy.

1.Introduction

Concrete is the most commonly material used in building industry. This is due to high compressive strength, ease of making, availability and low cost of raw materials, good compatibility with the other materials, like reinforcing steel and good durability during service life. Durable concrete has the ability to withstand the effects of environmental conditions, which will affected it, such as weathering, chemical attack, fire and abrasion. [1]. However, when it used to keep petroleum products, concrete should protected against their effect compared with concrete used in conventional environment, because the capability of some petroleum products to penetrate it and destroyed the bonding between cement paste and aggregate, also between concrete and reinforcing steel, this deterioration depended on the quality of the concrete and its impermeability [2]. Oil has become one of the most vital energy resources from the beginning of the previous century for its unique economic and operative characteristics. This has enabled it to exceed the other available power resources, and its importance has increased rapidly with its wide spread use and the discovery of huge oil reserves in different parts of the world [3]. Petroleum products may be store in above ground or underground steel. In some areas of worlds, reinforcing concrete tanks are used instead of steel tanks to overcome the shortage in steels materials. The using of concrete tanks to for petroleum products.

storage has many benefits such as low cost of repairing and high resistance of concrete to fire and explosive [4].

Many researches have studied SCC its properties. SCC or high- fluidity concrete can be easily placed and consolidated by its own gravity in a formwork, even with highly congested reinforcements, without external compacting by vibration [5]. It is characterized by its high filling capacity caused by high visco- plastic deformability, resistance to segregation, and an ability to maintain a stable composition throughout transportation and placing. SCC has the advantages of fast construction, noise reduction, and good formability [6].

2. Research Objective

Kerosene has great importance in many fields of life. Its storage in reinforcing concrete tanks offered many advantages relative to steel tank, such as safety, service life, and repairing cost. Therefore, the main goal of this research is to study the mechanical properties and durability of ordinary concrete, ordinary concrete coated with epoxy and SCC after exposing to kerosene for 120 days after cured in water for 28 days, in order to clarify the effect of kerosene on these different types of concrete.

Laith Mohammed-Ridha Mahmmod
University of Kerbala
Civil Eng. Department
laith_m87@yahoo.com

3. Experimental Works

Materials

Cement

Ordinary Portland cement type **I** is used in this research for preparing all mixtures, which conforms to the Iraqi Specification No.5/1984 [7], Table (1) illustrates the oxide compositions and physical properties of it.

Table (1): Physical and Chemical Properties of Cement

Chemical Analysis Composition %

Oxide composition	% Percer	Limits of Iraqi Specification No 1984[19]
SiO2	20.2	-----
Fe2O3	3.02	-----
Al2O3	3.98	-----
CaO	61.3	-----
MgO	3.74	5.0%(max)
SO3	2.3	2.8%(max)
Loss on Ignition (L.O.I)	3.23	4.0%(max)
Insoluble Residue (I.R)	0.9	1.5%(max)
Lime Saturation Factor (L.S.F)	0.94	0.66%-1.02%
Main Compounds (Bogue's Equation)		
C3S	58.42	-----
C2S	13.84	-----
C3A	5.44	-----
C4AF	9.19	-----

Physical Properties

Properties		Test Result	Limits of Iraqi Specification No 5 / 1984 [19]
Time of Setti (Vicat test)	Initial Se (min)	176	45 (Minimum)
	Final Set (min)	326	600 (Maximum)
Compressive strength (MPa	3 Days	18.2	15 (Minimum)
	7 Days	26.1	23 (Maximum)

Fine Aggregate

Al-Ekhadier sand is used as fine aggregate and it is conforms to the Iraqi Specification No.45/1984 [8] as shown in Table (2).

Table(2):Physical and Chemical Properties Sand

Sieve size (mm)	%Passing	Limits of specification
4.75	97	90 - 100
2.36	88	75 - 100
1.18	78	55 - 90
0.60	59	35 - 59
0.30	24	8 - 30
0.15	7	0 - 10

Table (3): Physical and Chemical Propertie of Gravel

Sieve size (mm)	passing %	Limits of specification No. 45/1984
37.5	100	100
20	100	95-100
14	/	/
10	50	30-60
5	6	0-10
Material Finer than 75-µm (sieve No. 200)	1.2	3 % (Maximum
SO3	0.083	0.1 (Maximum
Material Fir than 75-µm	3.2	5% (Maximum)
SO3	0.17	0.5 (Maximum)

Coarse Aggregate

Local crushed gravel is used as coarse aggregate, Table (3) shows the properties of it which conform to the Iraqi Specification No.45/1984 [8].

Silica Fume

Silica fume is a highly reactive material used in this research as pozzolanic materials to produce SCC. Silica fume is in compatible with the requirements of ASTM 1240 [9]. Physical and chemical properties are listed in Table (4), as provided by the manufacturer.

Table (4): Physical and Chemical Properties of Silica Fume (According to manufacturer)

Chemical Analysis Composition %		
SiO2	91.2	85% (Maximun
Fe2O3	1.07	Not available
Al2O3	0.31	Not available
CaO	1.19	Not available
MgO	1.96	Not available
SO3	0.88	Not available
Na2O	1.27	Not available
Loss on Ignition (L.O.I	1.36	6% (Maximum)
Physical Properties		
Color	Gray	Not available
Specific Gravity	2.18	Not available
Fineness (m2/kg)	20000	Not available

Table (7): Mixture Proportion

Mixture Compositio	Materials (Kg/m3)		
	MIX#R	MIX#EP	MIX#SCC
Portland Cement	190	190	386
Water	350	350	130
Gravel	992	992	971
Sand	815	815	708
super plasticizer(G5 (Liter)	---	---	6.5
Silica Fume	---	---	70

SUPERPLASTICIZER

A superplasticizer, which is known commercially as (Glenium 54) is used to produce SCC, it is conforms the requirements of ASTM 494 [10]. Properties as listed in Table (5), as provided by the manufacturer.

Table (5): Properties of superplasticizer (According to manufacturer)

Main action	Concrete superplasticizer
Appearance/Colures	Light brownish liquid
Specific gravity	1.1
Chloride content	nil
Compatibility with cemer	all types of Portland cemen
PH	7

Epoxy

Strong coat 400 used in this research to coat ordinary concrete, it is a solvent free, non-toxic; high build epoxy resin protective coating with outstanding chemical and mechanical properties .Strong coat 400 is supplied by (DCP) company, as a two component product in pre-weighted base and hardener packs, ready for site mixing. Table (6) shows technical properties of epoxy and they are conform the requirements of ASTM 881 [11].

Table (6): Properties of Epoxy (According to manufacturer)

Specific gravity	1.60
Solid content	100%
Abrasion resistance	Excellent
Bond strength	> 2 MPa
Pot life	100 min @ 25°C 45 min @ 35°C
Taber abrasion resistance: (1000 g, 1000)	70 milligram

Mixing Proportion

In this research, all mixes were designed to give minimum compressive strength of (30MPa) at age of 28 days, the mix of ordinary concrete was designed according to ACI-211.1 [12]. Moreover, mix proportions of SCC depend on trial mixes with two mixes prepared with constant content of constituent and different dosage of super plasticizer; in order to find the optimum dosage of superplasticizer that give the characteristic compressive strength. Slump flow test was used to measure the flowability of SCC with different dosage of superplasticizer according to ASTM 1611 [13]. Table (7) shows all details of three mixtures that used in this research. Whereas MIX#R refers to ordinary concrete, MIX#EP refers to ordinary concrete coated with epoxy, and MIX#SCC to self-compacted concrete.

4. Results and Discussion

After 28 days of curing in water, all specimen were immersed in kerosene for (30,60,90, and 120 days). All tests were done each 30 days from the day of immersing in kerosene. For each age of test, three samples were tested and the average was reported.

Density

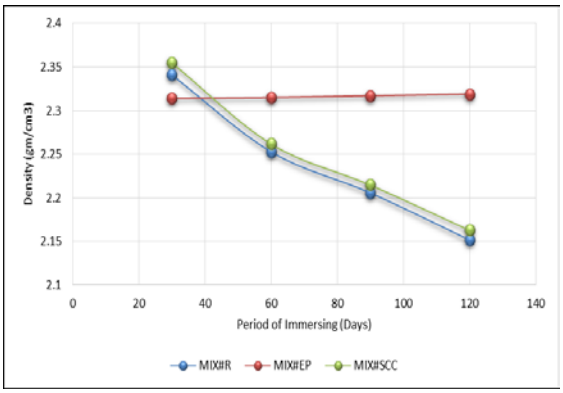
The density was conducted according to ASTM 642 [14]. Table (8) shows all test results that plotted in Fig. (1). The results showed decreasing in the density of both ordinary concrete and SCC with the increasing in the period of immersing in kerosene. This behavior can be attributed to the harmful effect of petroleum products on the bonding between cement paste and aggregate; So this leads to increase porosity and decrease density [4]. However, this reduction is higher in case of ordinary concrete than SCC with percentages of 5.9%, 0.4%, 0.41% and 0.51% at 30, 60,

90 and 120 days of immersing in kerosene, respectively.

The results also show that, the density of ordinary concrete coated with epoxy increaseed with slight percentage, as epoxy seal the voids of concrete, prevent kerosene from entering it, and make the harmful effect that mentioned above.

Table (8): Results of Density Test(gm/cm^3)

Mix Type	Age			
	30 Days	60Days	90 Days	120Days
MIX#R	2.341	2.253	2.206	2.152
MIX#EP	2.314	2.315	2.317	2.319
MIX#SCC	2.355	2.262	2.215	2.163



Figure, (1): Density vs. Period of Immersing in Kerosene

Absorption

Absorption of concrete was determined according to ASTM 642. Test results are listed in Table (8) and plotted in Fig. (2).

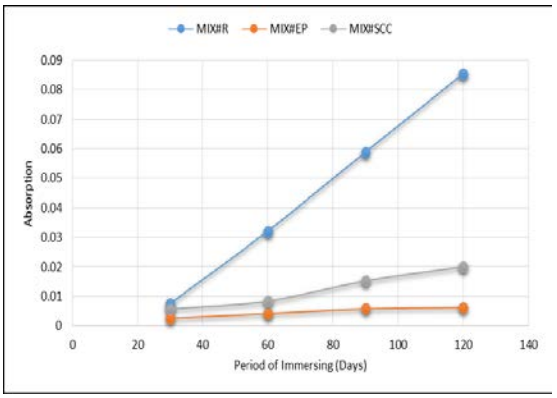
As indicated by results, absorption of all mixture increases with continues immersing in kerosene at all ages as a result of low viscosity.
The highest percentage of absorption was recorded for ordinary concrete, and the lowest percentage of ordinary concrete coated with epoxy, because seal activity of epoxy in seal concrete voids, SCC has lower absorption relative than ordinary concrete. This can be attributed to pozzolanic-reaction process through the hydration of cement that reduce the permeability of concrete [16].

Table (9): percentage of Voids

Mix Type	Age			
	30 Days	60 Days	90 Days	120Days
MIX#R	0.0277	0.0629	0.1231	0.1423
MIX#EP	0.0036	0.0068	0.0102	0.0142
MIX#SCC	0.0102	0.0320	0.0645	0.0821

Table (8): Results of Absorption Test (%)

Mix Type	Age			
	30 Days	60 Days	90 Days	120Days
MIX#R	0.00754	0.0321	0.0588	0.0853
MIX#EP	0.00265	0.00421	0.00585	0.00631
MIX#SCC	0.00561	0.00831	0.0153	0.0201

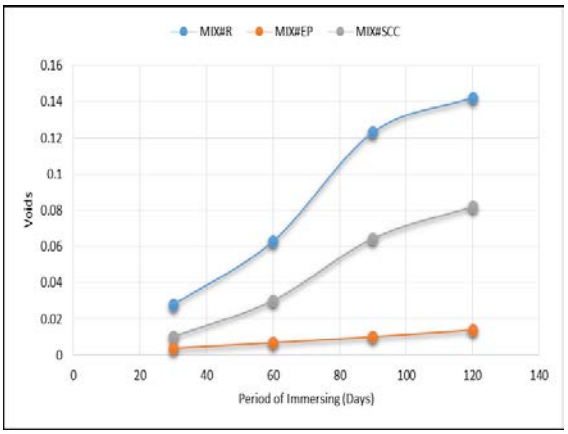


Figure, (2): Absorption (%) vs. Period of Immersing in Kerosene

Voids

As shown in Table (9) and Fig. (3), percentage of voids in all mixture increases with increasing period of subjecting to kerosene. Ordinary concrete has higher percentage of voids compared with SCC, which has higher percentage of voids relative to
concrete coated with epoxy, this is due to filling of voids of concrete with epoxy
and less affected by harmful effect of kerosene. In addition, SCC has percentage of voids lower than ordinary concrete as silica fume reaction with calcium hydroxide liberated on hydration, to form compounds possessing cementitious properties that fill pores of

concrete and increase its water tightness and decrease susceptibility to dissolution and leaching [16].



Figure, (3): Voids (%) vs. Period of Immersing in Kerosene

Compressive Strength

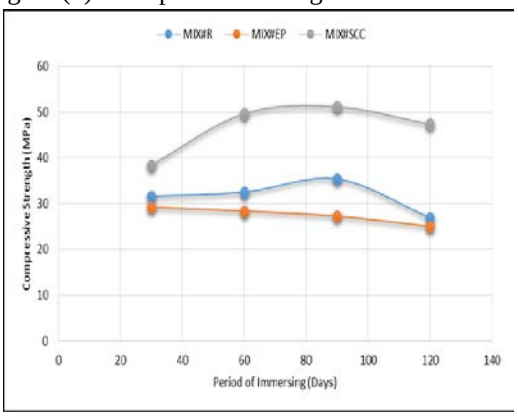
This test was carried out according to B.S.1881: part 116 [18], on 150mm cubes by using a compression testing machine. All compressive strength results for different immersing periods are represented in Table (10) and Fig. (4).while the compressive strength of ordinary concrete and SCC at 28 days was 28.9MPa and 37.4MPa respectively. The results indicated that increasing compressive strength of ordinary concrete and SCC with increasing the period of immersing in kerosene until 90 days, after that compressive strength of both mixtures decrease. This behavior is due to the pores inside the concrete, which was still partially filled with water and leads to further hydration that delay the deterioration of concrete [4]. In addition, the decreasing after 90 days because penetrate kerosene into structure of cement and caused extending gel pores and spreading solid hydration components which caused weakly adhesion and cohesion forces in cement and easily slip of internal grains [19].The compressive strength of concrete coated with epoxy is lower than that of ordinary concrete and SCC at all ages of testing. The results also indicated a continuous decreasing of compressive strength of concrete coated with epoxy with continues immersing in kerosene. This may be attributed to water held in concrete pores during hydration for 28 days and still inside it after coating with epoxy that rise the internal hydraulic pressure, and reduction in surface power

as a result to absorption water caused reduction in compressive strength.

Table (10): Results of Compressive Strength Test(MPa)

Mix Type	Age			
	30 Days	60 Days	90 Days	120 Days
MIX#R	31.6	32.5	35.4	26.9
MIX#EP	29.2	28.4	27.3	25.1
MIX#SCC	38.4	49.7	51.2	47.4

Figure (4): Compressive Strength vs. Period of Immersing



in Kerosene

Splitting Tensile Strength

This test method consists of applying a diametric compressive force along the length of a cylindrical concrete specimen at a rate that is within a prescribed range until failure occurs. This loading induces tensile stresses on the plane

Containing the applied load and relatively high compressive stresses in the area immediately around the applied load [20]. This test was conducted according to ASTM 496. All results are

listed in Table (11) and plotted in Fig. (5). The results showed reduction in splitting tensile strength with increasing in immersion period in kerosene as a result of its harmful effect on concrete, ordinary concrete gives lowest splitting tensile strength values compared with coated concrete and SCC which gives the highest values.

Table (11): Results of Splitting Tensile Strength Test(MPa)

Mix Type	Age			
	30 Days	60 Days	90 Days	120 Days
MIX#R	2.9	2.1	1.8	1.7
MIX#EP	3.0	2.4	2.2	2.1
MIX#SCC	4.1	4.0	3.8	3.5

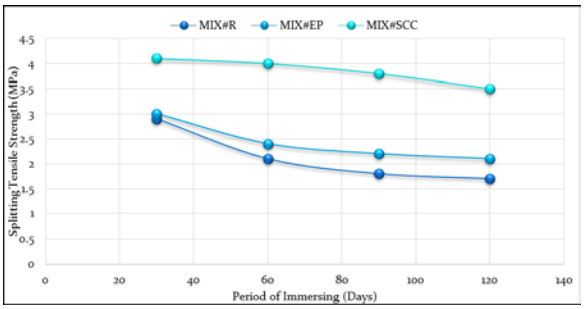


Figure (5): Splitting tensile Strength vs. Period of Immersing in Kerosene

5. Conclusion

1- A good behavior in concrete was shown for the SCC for different properties.

2- High density reduction takes place due to the higher amount of absorbed oil
products relative to their viscosity for all types of mixes.

3- The best behavior of absorption and voids content was recorded for ordinary concrete coated with epoxy of all samples exposed to kerosene.

4- The compressive strength of different concrete mixes exposed to kerosene decreases with time compared with that of control mixes at 28 days. The maximum reduction was found in ordinary concrete coated with epoxy. It was up to (13.1%) after 120 days immersing in kerosene.

5- The splitting tensile strength of different types of concrete exposed to kerosene decrease with increasing period of immersing in kerosene.

REFERENCE

1- Neville A.M. and Brooks, J.J., "Concrete Technology", Second Edition, Longman Group U.K. Limited, 2010.
2- Faiyadth, F.I., "Bond Characteristics of Oil Saturated
Concrete", The International Journal of cement and lightweight concrete, Vol.7, No.2, May 1985.
3- World Economic Forum, " Energy Vision: Energy Transition, Paste and Future", 2013.
4- Fawzi, N.M. and Abed Al-Ammer, S.A.,"Effect of Petroleum Products on Steel Fiber Reinforced Concrete", Ms.c. Thesis, University of Baghdad, 2010.
5- Efnarce, "Specification and Guidelines for Self-Compacting Concrete", Efnarce, UK, 2002.
6- Self-Compacting Concrete European Project Group," The European Guidelines for Self Compacting Concrete", 2005.
7- Iraqi Standard Specification No. (5), "Portland Cement", Central Organization for Standardization and Quality Control, 1984.
8- Iraqi Standard Specification No. (45), "Aggregate from Natural Sources for Concrete and Building Construction" .Central Organization for Standardization and Quality Control, 1984.
9- ASTM C1240-, "Standard Specification for Silica Fume Used in Cementitious Mixtures", Annual Book of ASTM Standard, 2003.
10-ASTM 494, "Standard Specification for Chemical Admixtures for
Concrete", Annual Book of ASTM Standard, 1999.
11- ASTM 881, "Standard Specification for Epoxy-Resin-Base Bonding Systems

for Concrete", Annual Book of ASTM Standard, 2002.
12- American Concrete Institute Committee 211.1-21, "Proportions for Normal, Heavyweight and Mass Concrete", Reported by ACI committee 211, ACI Manual of Concrete Practice.
13- ASTM 1611, "Standard Test Method for Slump Flow of Self-Consolidating
Concrete", Annual Book of ASTM Standard, 2013.
14-ASTM 642, "Standard Test Method for Density, Absorption and Voids in
Hardened Concrete", Annual Book of ASTM Standard, 1997.
15- Clute, D.," Fundamentals of Concrete Mix Design", National Design, Construction, and Soil
Mechanics Center; Fort Worth, Texas, 2003.
16- Kosmatks, S.H., Kerkhoff, B. and Panarese, W.C., "Design and Control of Concrete Mixtures", Fourteenth Edition, Portland Cement Association, United State of America, 2003.
17- Stratigic Highway Research Program, "Concrete Microstructure; Porosity and Permeability", National Research Council, Washington, 1993.
18- British Standards Institution. B.S 1881, Part 116," Method for Determination of Compressive Strength of Concrete Cubes", 1983.
19- Fawzi, N.M. and Atwan, D.S., "Effect of Kerosene and Gasoline on Some
Properties of High Performance Concrete", Journal of Engineering, Vol.17, No. 6, 2011.
20-ASTM 496, "Splitting Tensile Strength of Cylindrical Concrete Specimens", Annual Book of ASTM Standard, 1996

Production of high performance lightweight ferrocement plates

Prof. Dr. Nada Mahdi Fawzi A Jalawi Baghdad University/civil engineering Department

Telephone No.: 009647902216613, E-mail :naljalawi@yahoo.com

Abstract, **The research is concerned with a new type of ferrocement characterized by its lower density and enhanced physical properties such as compressive strength and impact strength.**
A compressive strength of 43 MPa was obtained using suitable proportion of cement and lightweight porcelenite aggregate.
The compressive strength of the mortar mixes using 10%feldspar was increased by (91.1, 66.6) % at 28 and 90 days age respectively compared to (82.2,60)% at the same ages respectively for microsilica.
Using calcined feldspar powder caused a clear drop in water absorption indicating a decrease in volume of open pores.
Include, the impact resistance which was deduced from the free falling weight experiments was found to increase as the volume fraction of reinforcement increased.

Key word: ferrocement, impact strength, lower density, microsilica, porcelenite

Introduction

The use of ferrocement returns back to the beginning of the use of Portland cement, In the year 1848 **(Ramaswamy1977**) Jean Louis Lamboat created a structure of a boat using reinforced mortar cement with many layers of steel wire, and named this composite materials (Ferricement). In the year 1943 the architect (Nervi) **(ACI Committe 549-1997)** revive the idea of ferrocement when he noticed that the layers of reinforced concrete become a durable material, possesses high mechanical properties in terms of resistance and performance.

The uses of ferrocement has expanded all over the world, and the International Ferrocement Information Center (IFIC) was established at the Asian Institute of Technology in Bangkok, Thailand. The center issuing ferrocement magazine, which publishes researches for scientists and specialists in ferrocement and its applications. And in 1979 . The International Union of laboratories testing materials and research which was named RILEM Committee 48FC was founded to evaluate methods of examination of ferrocement.

Review of previous research

Ferrocement can be defined as a composite material, composed of two or more materials to form a new substance with properties that is different from its constituents in behavior and properties.

Ferrocement has mechanical properties better than conventional concrete such that it has a high value of flexibility index and resistance to cracking and it reduces the width of cracks due to the use of wire meshes with small diameters. It also has an improved ductility and high Resilience (**ACI Committe549**).

RILEM defines ferrocement as a function describing a construction material or building blocks (components) .

Neville 2010 defines high-performance concrete as a concrete with high resistance and low permeability noting that the high performance concrete need to reduce the size of voids and this is done by using micro mineral additives like types of pozolana active microsilica. Neville considered the high-performance concrete as a development and extension of the conventional concrete.

Al Badri 2001 presented a research about high performance concrete using ground furnace slag after the addition of small amounts of microsilica, as well as ground slag with the addition of the rice husks ash to increase the activity of slag. He said that the addition of slag lead to increase compressive strength and that the addition of microsilica 2% of cement weight lead to increase the activity of slag which is also increase by adding rice husks ash and that the best rate of addition is 5% by of cement weight and besides to the increment in compressive strength there will be increment in modulus of elasticity.

Naji 2001 made several experiments on several ferrocement panels where he studied the effect of several variables such as the number of layers of steel meshes, plate thickness, the effect of additives and super plasticizers and the effect of rice husks ash. He

concluded that the number of layers of steel meshes and cement content do not affect the maximum load. The results also showed that the use of ashes of rice husks with super plasticizer is of significant effect on the flexural strength compared to the reference mix.

Ahmed 1999 Studied the behavior of four domes made of ferrocement with several variables such as thickness , ratio of steel skeleton. He analyzed the results using a flexible-plasticity relationship with strain hardening and the results of the experiment of the load –deflection had approached to the theoretical results.

There are previous researches **(Matti 1985, Raouf 1975, Nwokoye 1974)** proved that the modulus of elasticity of concrete E depends on the pulse velocity and density (P) and the form of wave propagation (longitudinal, transverse, surface or plate). They further explained that in general it can be applied on composite materials when they are small sized compared to the length of the vibrating wave.

Impact Strength test methods

Mahjoub 1992 had categoried impact tests to two main types:

1. tests that end by breaking the sample test as a result of one blow and are measuring the energy absorbed in the course of the test.

2. tests that lead to breaking the sample test by repeated strikes, which are of known energy.

Shah 1972 had explained that the volume ratio of the steel meshes and their types that are used in construction parts manufactured of ferrocement plays a main role in determining its behavior when exposed to impact loads. In general, impact strength of ferrocement increases with the increment in specific surface and increases almost linearly with the increase in the volume ratio.

The materials used

cement:

The ordinary portland cement has been used and after physical and chemical tests we founded identical to the Iraqi Specification **(IQS) No. (5) 1984**.

Porcilinite Aggregate

Broken porcilinite Aggregate was used, which is rocks of light weight silica present in west desert Iraq in plated deposits with montmoronite clays and they were identical to (**ASTM C330**) sieve analysis, rodded dry density = 10.1 kN / m 3, porosity of 25.3%, specific gravity 1.41 , absorbed 36.6%.

Water

Ordinary tap water is used in the experimental process as mixing water. Curing was done by putting samples in a closed polyethylene bags.

Steel wire mesh

Only one type of steel wire mesh was used with a diameter of 1 mm and space of 10 mm in the form of square.

Additives

Two types of mineral additives were used :, burned feldspar at a temperature of 650 ° C and microsilica. Also super plasticizer type G was used according to (**ASTM C 494**).

Feldspar

It is considered one of the main minerals of igneous rocks. The chemical formula for feldspar is (Al2O3.ySiO2.nMO) where (M) may be K, Ca, or .Na it can change into kaoline metal through geological ages and by physical and chemical factors. It can be used as pozzolan active, After grinding to high fineness (**Laith 2004**).

For this research it has been brought from the sea of Najaf, where it was crushed (using a ceramic balls grinder) to the smoothness (measured by Blaine device) of 1200 m 2 / kg. And then burned until 650 ° C in an electric furnace laboratory, in order to activate it.

Microsilica

Microsilica was used, which is a soft material with high fineness. It has been chemically tested and proven that it is a sort of amorphous silica containing 95% of silica which is a by- product of the production of iron furnaces and ferrous metals containing silicon where quartz turns into silicon at 2,000 C and produces silicon dioxide fumes SiO. This in turn, gather as spherical particals at lower

temperatures. The Diameter of these particals ranges between (0.1 -0.12) micrometer and the surface area ranges between (15-25) m 2 / g.

Super plasticizer

Its properties are as follow : 1-super plasticizer type G was used according ASTM C494 and British BS 5075 2-A dark brown liquid 3- specific weight ratio of 1.1 4- chlorides do not exist, 5- the recommended dosageto be used is (0.8 - 1.5)% of cement weight in the concrete mix.

Trial mixes of mortar ferrocement

The trial mixes of**(Aljalawi 1997**) was used and in order to achieve the required properties. A number of experimental mixtures was used depending on different variables such as, water cement ratio, compressive strength. Through the results of this experimental concrete mixes we reached to set a series of mixtures to study the variables mentioned above.

The ratio of cement to aggregate mixtures is 1: 1.1: 1.5, 1: 2 by vol.. Then water cement ratio was changed to get the required properties, it was noted that the percentage of water cement ratio of 0.4 is appropriate to the requirements of this research.

Mixing and casting concrete

The mixing process was conducted using a manual mixing and according to the American Standard **ASTM C-192**. Thus we obtained a mix concrete, elastic and cohesive without excessive smoothness. After the complete of mixing, concrete is casted into metal molds and then compacted using a vibrating table.

Curing

After completion of the casting process templates were covered directly by polyethylene foil to prevent evaporation of water from fresh concrete and was left for 24 hours. They were then taken out from mold and saved in ethylene closed plastic bags for 28 days. This method has been followed in accordance with the recommendations of the American Standard **ASTM C-192.**

Laboratory tests

Strength activity index

We found it for each of the feldspar and microsilica according to specifications of (**ASTM C-311, ASTM C 1240**).

Flow test

Flow test is done in the lab. to identify the extent of their tendency for segregation and workability. This test expresses, the extent cohesion of concrete components together. This test was according to **ASTMC-230** which is calculated according to formula No. (1)

Flow = {(D1-D)} / D1} * 100. …….(1)

Where D1=original dia. D=scattered dia.

Compressive strength test

Concrete samples had been used with mold measuring 50 mm to find a compressive strength of light weighted concrete according to American Standard ASTM **C-109** and with an average of three samples for each test.

Density test

We found density for mix of concrete in the dry state air according to American Standard ASTM C-567.

Flexural Strength

It was done according to the American standard (**ASTM C-78**) where loading was done by hydraulic device and by using proving ring with measuring deflection by dial gauge accuracy 0.01 mm and plates were used with measurements of panels (30 * 200 * 500 mm).

Absorption test

Absorption test was done to assess the permeability of the mortar. Each mix used in this research was according to **ASTM C642** .

Impact strength

An iron block was used thrown from a meter high on the plates to measure the impact strength of ferrocement plates dimensions (500 * 200 * 30) mm reinforced by number of layers of meshes and unreinforced at different curing days (28-90)days .

5-Results and Discussion

The influence of the proportion of feldspar and microsilica

from table (1) we can notice that the increase in the proportion of the partial replacement of feldspar leads to increase the compressive strength at 28 days and 90 days and that the increase is by (77.7%, 56.6%) at a partial replacement for cement by (5%) at 28 and 90 days respectively, and by an increase of (66.6,91.1)% when the partial replacement of 10% at 28 and 90 days respectively. Where as for the microsilica shall be the rate of increase (73.3-50%) when the partial replacement of cement is by 5% at 28 and 90 days, respectively, and increase (82.2-60%) when the partial replacement of 10% at 28 and 90 days respectively. This confirms what was presented by (**Nassif et al 2005**)

Behavior ferrocement plates in flexural strength

The flexural strength for reinforced plate is high if compared with unreinforced plate, because the increment volumetric ratio of reinforced plates lead to a strengthening the link between mortar and reinforce (**Shuxin Wang 2000**, **Abo El-Wafa 2010**), as well as high modulus of elasticity for reinforced plates lead to high modulus of elasticity for ferrocement light weighted plates, the results are shown in table (2).

Effect of partial replacement feldspar or microsilica on the absorption

The usage of feldspar or microsilica leads to improve the properties of mortar lightweighted porous and reduce the gaps where these materials react with Ca (OH) 2 of cement hydration composed materials stable and is working to fill in the blanks and thus reduce the porosity(**Boshehrian,A.2011, Sumadi, S.R2008**).

Effect of volumetric ratio of reinforced on the impact properties

Increase volumetric ratio of reinforced plates in light weighted ferrocement plate results in high resistance to impact, and lead to good mechanical properties to prevent cracking and absorb the high-energy, making ferrocement plates with higher durability and flexibility to make it withstand external shocks better than light weighted plate without reinforcement (**Dhaher1993, Husein, N. R. 2013**). the results are shown in table (2).

Conclusions

In this paper , a new type of high-performance and lightweighted ferrocement was produced using local raw materials and studying physical properties.

1- Trial mixes: through the experiences of several variables on the same experimental mixtures it was found that the optimum mix is by using cement with the fine aggregate porcilinite ratio of 1: 1 and the ratio of water to cement is 0.4 with the replacement of 10% feldspar.

2- Compressive strength of the optimum mixture 43 MPa. The compressive strength for the mixture with local feldspar to reference mix was increased (91.1,66.6)% at 28 days and 90 days respectively in the case of the use of microsilica. The increase of compressive strength (82.2,60)% at 28 days and 90 days respectively showed that the use of the local feldspar gives a higher compressive resistance of microsilica.

3- Flexural strength: The addition of reinforcement ratio 2.08 % lead to an increase in flexural strength compared with unreinforced mortar and this increment reached (77.3, 61.7)% at 28 days and 90 days respectively for the mortar containing 10% feldspar. While when it contained the microsilica, the increment was (67.3,59.3)% at 28 days and 90 days respectively.

4- It was observed that the replacement feldspar or microsilica reduce the absorption ratio of mortar because it has fine material that works on reducing the porosity of the mortar due to fineness.

5- Impact strength test results showed that the impact of ferrocement plates increase with the volumetric ratio of reinforced plates. When panels are unreinforced they fail with three hits, compared to 39 hits for plates that are reinforced with volumetric ratio amount of (2.1 %).

References

1. Abo El-Wafa, M. :Flexural behavior of lightweight ferrocement sandwich composite beams, Journal of Science & Technology Vol. (15) No.(1) 2010 JST 3

2.ACI Committee 549,:State-of-the Art Report on Ferrocement, 2009

3. American Socity For Testing and Material, C-78: Flexural strength of concrete, 2005
4. American Socity For Testing and Material, C-330 :Lightweight aggregates for structural concrete, 2005.

5. American Socity For Testing and Material, C-494/C494M: Standard specification for chemical admixtures for concrete,2006
6. American Socity For Testing and Material, C-311: Sampling and testing Fly ash or natural pozzolans for use in Portland – cement concrete, 2006

7. American Socity For Testing and Material, C-1240: Use of Silica Fume as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout1,2006.
8. American Socity For Testing and Material, C-642:Density, Absorption, and Voids in Hardened Concrete, 2006.
9. American Socity For Testing and Material, C-567:Determining Density of Structural Lightweight Concrete, 2006

10. Ahmed Azad M., :Thin ferrocement dome structures, Ph.D Technology University 1999.

11. Al-Badri, Firas Fadhil :Evaluation of high-performance concrete by nondestructive test, M.Sc thesis, University of Baghdad at 2001.

12.Al - Jalawi, Nada Mahdi :properties of lightweight concrete reference to the thermal insulation and acoustic impedance, M.Sc. thesis, University of Baghdad in 1997.

13. Boshehrian,A., Hosseini,P. :Effect of nano-SiO2 particles on properties of
cement mortar applicable for ferrocement elements, *A*.Booshehrian & P.Hosseini CRL Letters Vol. 2(1) 2011
14. Dhaher, B.A.:The use of local porcelinite for the production of lightweight concrete units, M. Sc. Thesis. University of Technology 2001.

15.Husein, N. R., Agarwal, V.C.:An Experimental Study on Using Lightweight Web Sandwich Panel as a Floor and a Wall, International Journal of Innovative Technology and Exploring Engineering (IJITEE) ISSN: 2278-3075, Volume-3, Issue-7, December 2013.

16. Iraqi StandardSpecification No. 5 of 1984 "Portland cement".

17. Jensen and Bendt Aarup :Fire resistance of fiber reinforced silica fume basedconcrete, 4th International Symposium on Utilization of High – performance concrete , Paris 1996.

18. Mahjoub,M.H., Gorst, N.J.S. and Barr, B.I.G.,: Development of an instrumented impact testing apartus, Fiber Reinforced Cement and Concrete, Edited by Swamy ,R.N., Rilem,1992,London No.18.
19. Matti, N. :Assessment of dynamic properties of ferrocement plates by non destractive tests, M.Sc. Thesis University of Baghdad, 1985.

20. Nazar,L. :Flexural behavior of steel fibre reinforced high performance concrete manufactured using local material, M.Sc. Thesis of University of Baghdad, 2004.

21. Naji ,A., :Effect of superplasticizer and rice husk ash on ferrocement slabs under static load, M.Sc. thesis Technology University 2001.

22. Nassif H.H., Husam N.,Nakin S. :Effect of pozza: anic materials and curing methods on the elastic modulus of HPC, Cement and Concrete Composites 2005.

23. Neville ,A.M : Properties of concrete, Fifth edition ,Published long man group, 2010.

24. Nwokoye, D.N.:Assessment of elastic modulus of cement paste and mortar phases in concrete from pulse velocity tests, Cement and Concrete Research Vol.4, 1974 PP 641-655 .
25.Ramaswamy, G.S.:Ferrocement a new building material for long spans, The first Scientific Conference of Scientific Research Stablish, Baghdad 25-30 March 1972.

26. Raouf, Z.A., :Dynamic mechanical properties of fibre cement composites, Ph.D. thesis UMIST, 1975.

27.Shah, S.P., Key, W.H.,:Impact resistance of ferrocement, Journal of the structural division ASCE, Vol.98, No. ST1, January 1972.

28.Shuxin W., Naaman, A.N. and Victor C.Li.,:Bending response of hybrid ferrocement plates with meshes and fibers, Journal of ferrocement Vol. 34, No. 1, 2004.

29.Sumadi, S.R.: Development of lightweight ferrocement sandwich panels for modular housing and industrialized building system, Universiti Teknologi Malaysia 2008

Table (1) physical properties of porcelinite mortar

Mix symbol	Flow %	Admixture ratio%	Comp. strength MPa(28 days)	Comp.strength MPa(90 days)	Absorption% (28 days)	Absorption % (90 days)
Ref.	87	0	22.5	30	7.3	7
Fel 1	70	5	40	47	6.1	5.9
Fel 2	77	10	43	50	5.8	5.6

Sf 1	68	5	39	45	6.3	6
Sf 2	75	10	41	48	6	5.8

Table (2) flexural strength for ferrocement plate

Mix symbol	description	Admixture ratio%	V_f %	Sr (cm^{-1})	Density Kg/m3	Flexural strength MPa(28days)	Flexural strength MPa(90 days)
Fel 2	Without mesh	0	0	0	1710	2.82	3.4
Fel 21	2 layer mesh	10	1.04	0.416	1730	3.97	4.2
Fel 22	4 layer mesh	10	2.1	0.832	1742	5.0	5.5
Sf 2	Without mesh	0	0	0	1700	2.6	3.2
Sf 21	2 layer mesh	10	1.04	0.416	1720	3.68	4.0
Sf 22	4 layer mesh	10	2.1	0.832	1745	4.35	5.1

Table (3) impact strength for ferrocement plate

Mix symbol	Impact strength at 28 days		Impact strength at 90 days	
	No of hits until first crack	No of hits until failure	No of hits until first crack	No of hits until failure
Fel 2	1	3	1	5
Fel 21	3	18	4	22
Fel 22	6	25	8	30
Sf 2	1	3	1	5
Sf 21	3	16	4	20
Sf 22	6	24	7	27

Strength characteristics of CO_2-cured synthetic fibers reinforced cementitious boards

Hussein Jerri Jasim, Shakir A. Salih, and Maan S. Hassan

***Abstract*—Diffusion of carbon dioxide through the pores structure in the cementitious matrix promote carbonation reaction with the calcium hydroxide $Ca(OH)_2$ produced from the cement hydration. Such manufacturing procedure which accelerates the setting time may adversely affect the higher pH value and decrease the alkalinity of the cement matrix leading to break down the steel fiber passive film and then corrosion process to initiate. The present work investigates the suitability of utilizing synthetic fibers (glass or carbon fibers) in manufacturing CO_2 cured fiber reinforced cementitious boards. Two types of synthetic fibers were implemented and evaluated: glass and carbon fibers with volume fractions up to 0.625%. Comparisons were made between the flexural strengths, stiffness and toughness of the produced boards which fabricated with conventional and different concentrations of CO_2 curing (i.e. 0%, 50%, and 100%). This paper is an attempt to fabricate sustainable eco-composites that re-integrate the pollutant CO_2 again with cementitious boards. Flexural performance results suggested the preferred fabrication conditions. In glass and carbon fibers boards, chamber duration, fiber content, fiber lengths and CO_2 concentration have significant effects on the board's flexural performance. Higher volume fractions (i.e. 0.26% to 0.25% for glass fiber and 0.375 to 0.635 for carbon fiber) and shorter fiber lengths resulted better boards flexural strengths when compared with boards reinforced with fibers of lower contents or longer lengths, even at half the CO_2 concentration.**

Index Terms*— *accelerated curing, carbon dioxide gas, carbon fiber, cement composite, glass fiber.

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F. A. Author is with the National Institute of Standards and Technology, Boulder, CO 80305 USA (corresponding author to provide phone: 303-555-5555; fax: 303-555-5555; e-mail: author@ boulder.nist.gov).

S. B. Author, Jr., was with Rice University, Houston, TX 77005 USA. He is now with the Department of Physics, Colorado State University, Fort Collins, CO 80523 USA (e-mail: author@lamar.colostate.edu).

T. C. Author is with the Electrical Engineering Department, University of Colorado, Boulder, CO 80309 USA, on leave from the National Research Institute for Metals, Tsukuba, Japan (e-mail: author@nrim.go.jp).

I. Introduction

Cement based mortar composite is well known as a brittle type of failure due to its weakness in tensile strength and low fracture toughness which required intensive attention throughout the design process to achieve long-term durability aspects in concrete structures. Incorporation of fibers is receiving growth attention to enhance the flexural performance of cement based composites such as toughness, modulus of elasticity and peak strength; as well as other mechanical properties like impact resistance, capacity of energy absorption and fatigue endurance. It is well recognized [1-6], that the ability of fibers to resist the development and propagation of micro-cracks occurs in the cement paste due to the external applied load or even due to other sources. They can bridging the opposite crack sides and then stress transfer to the nearby paste area, and finally crack tip plasticity mechanisms. Such advantages promote the implementation and usage of fibers in several applications such as hydraulic structures, lining tunnels, airfield pavements and highway, concrete beams and slabs, boards and thin sections [4-6].

Several types of fibers were being used to reinforced cementitious materials and extensive works have been done to investigate the final mechanical performance of these types of cementitious mortars reinforced with fibers like: cellulose, steel, polypropylene, glass, carbon; and with different aspect ratios or content. However, the non-corrodible characteristics, low weight and low cost properties may motivate structural engineers to implement some types such as glass or carbon fibers among the others. Furthermore, the availability of this kind of synthetic fibers promote more investigations to be used as an alternative reinforcing fibers to asbestos especially in the present of health restrictions.

Manufacturing process of fibers reinforced cement board may includes heat curing combined with pressing steps of the fresh mixes in the board molds. Such long time procedure may lead to increase initial costs and reduce production rates. However, these processing steps are considered essential to prevent swelling back to the original thickness after pressure release [7,8].

This study is, therefore, aimed to generate more information on the flexural response of cementitious composite reinforced with two different fiber types: carbon and glass fibers; two different lengths: 4mm and 9mm; and three percentage of fiber contains. It is also aimed to develop an efficient approach to processing of synthetic fiber-reinforced cement composites, which makes value-added use of carbon dioxide for early curing conditions. Flexural tests on cement composite boards

of size 300 × 150 mm and deferent processing aspects were implemented.

II. Experimental program

A. Materials and manufacturing procedures

The investigated carbon and glass fibers were taken from a fiber warp waste roll. They were cut into two lengths: 4 mm and 9 mm. Fig. 1 shows a picture of carbon and glass fibers.

Figure 1- Appearance of roll and prepared fibers, (A) carbon fibers and (B) glass fibers

TABLE 1-PROPERTIES OF CEMENT AND SILICA FUME

Composition (%)	Cement	Microsilica
Chemical compositions		
SiO_2	21.7	90.65
Al_2O_3	4.61	0.02
Fe_2O_3	3.35	0.01
CaO	61.89	1.22
MgO	3.05	0.01
SO_3	2.4	0.24
Lime saturation factor	0.87	-
Insoluble material	0.6	-
Loss on ignition	2.16	2.86
Physical properties		
Bulk density(kg/ m^3)	1180	500
Specific surface (m^2/kg)	310	16000

TABLE 2-THE COMPOSITION OF FIBER REINFORCED CEMENT COMPOSITE

Fiber Type	Carbon or Glass
Fiber Mass Fraction (%)	0.3, 0.9, or 1.5
Fiber lengths (mm)	4mm or 9mm
Sand/Cement ratio (by weight)	1.25
Superplasticizer (% by weight of cement)	1%
Microsilica/Cement ratio (by weight)	0.4

Type I Portland cement with chemical composition as given Table 1 was used in the mixtures of this investigation. The fiber mass fraction and matrix mix proportions used are shown in Table 2. The manufacturing process of a thin board carbon/or glass fiber reinforced cement composites was similar to that used by Soroushian et al [9,10] and others [11]. It involved mixing of the ingredients using mortar mixer then placing the blend into a 300 mm by 152.5 mm (12 in. by 6 in.) rectangular wooden box (made of plywood). The ratios of fiber/cement used in this study were 0.3%, 0.9% or 1.5% by weight, and water/ cement weight ratio of 0.29, targeting 12 mm thick boards. The box was first painted by thin oil layer to prevent any possible adhesion with hardened matrix, the mix was then spread in the box and carefully leveled with appropriate tool, and was then covered by nylon sheet to keep it in moist condition. After 24 hrs, the wooden box was removed and specimens were now ready for curing. Fig. 2 shows the cement-bonded fiberboard (CBFB) processing system for CO2 curing. To produce various concentrations of CO2 gas in air, as seen in Fig. 2, two gas cylinders (one CO2 and the other air) were used. To control the gas flow level and thus the CO2 concentration, a flow meter was connected for each cylinder.

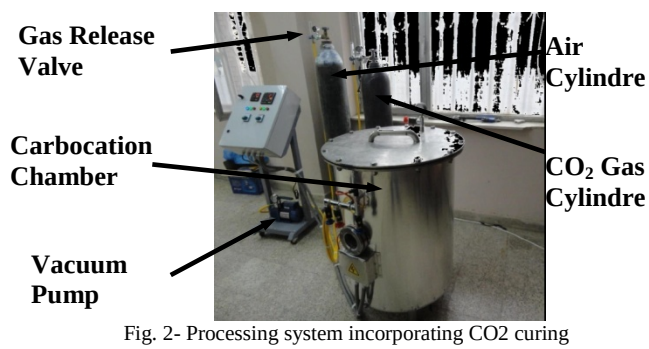

Fig. 2- Processing system incorporating CO2 curing

Fig. 3- Typical appearance of cement-bonded fiberboard (CBFB)

After the completion of processing, curing was started firstly by stacked flat in a pre-curing oven-drying for young sheet prior to CO2 curing. This step is essential to lower moisture contain of board to the point where CO2 penetration and reaction would be facilitated [12]. Typical appearance of the resulting cellulose fiber cement boards is shown in Fig. 3. The set-up of carbonation system is capable of applying any combination of CO2, air and vacuum on the board. Three different carbon dioxide (CO2) gas concentrations: 0%, 50%, or 100%, were used and oven durations of 0.5, 1.0, 2, or 4 hrs inside the chamber for each processing condition. Fig. 4 describes the details of the experimental works.

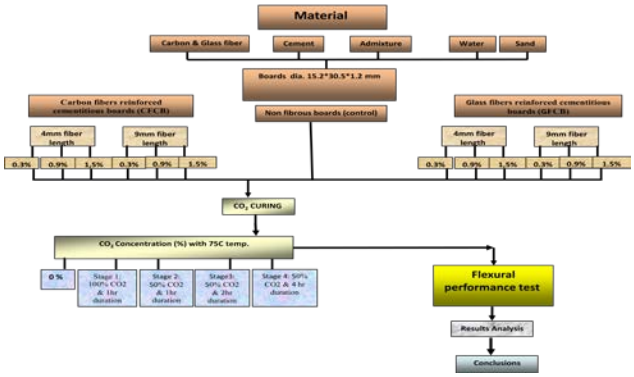

Fig. 4- Details of the experimental works in this investigation

B. *Specimens and test procedures*

Flexural tests were performed following ASTM C 1185-12 [13]. Samples have a clear span of 254 mm, a width of 152.4 mm, and a thickness 12 mm. For each condition at least three replicated specimens were tested for all mix designs considered. The flexural test set-up (3-point test) used for all boards was shown in Fig. 5. A displacement rate of 2.2 mm/ min was used in flexure tests (which were conducted in a displacement-controlled mode). A data acquisition system controlled by computer was used to record the test data. Data generated from this system were used to: (i) draw load–deflection curves: (ii) characterize flexural strength, toughness (which is the total area underneath the load–deflection curve); and (iii) initial stiffness (which is defined here as the stiffness obtained through linear regression analysis of the load–deflection points for loads below 15% of maximum load). A dry condition type was implemented throughout the evaluation of flexural performance.

Fig. 5- Photograph of flexural test setup for cement-bonded fiberboard (CBFB)

III. Results and Discussions

Figs. 6 and 7 present typical flexural load–deflection curves of cement-bonded carbon/ or glass fiberboards subjected to different CO2-curing concentrations.

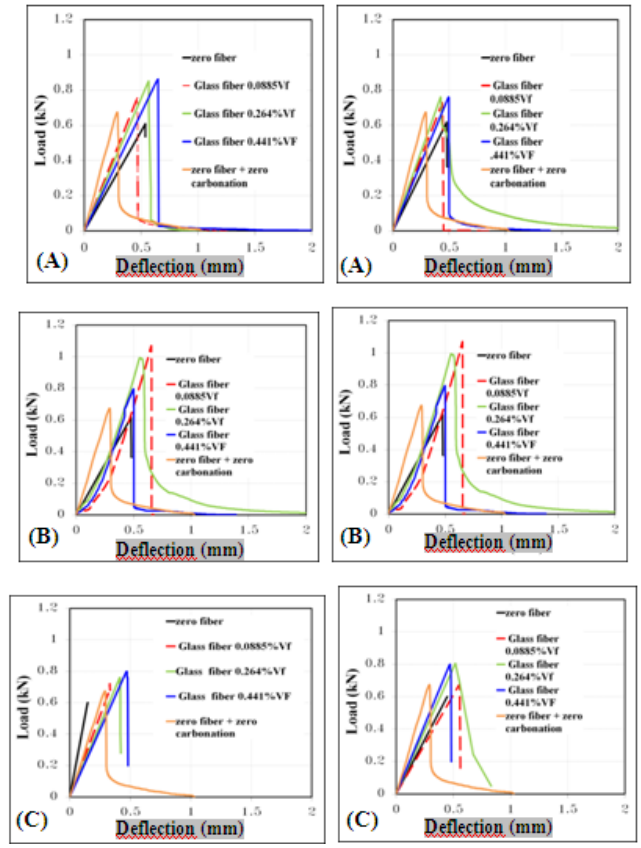

Figure 6- Typical Load deflection curve characteristics of cement-bonded glass fiberboard; (A) 100%carbonation 1 hr, (B) 50% carbonation 1 hr, (C) 50% carbonation 4hrs; Left side 4mm and Right side 9mm fiber length.

In general, the flexural performance of CO2 cured boards was improved particularly for higher fiber content for both carbon and glass fiber types. A higher concentration of CO2, 100% combined with minimum chamber duration (i.e. one hr), is noticed to have non- significant effect towards improving flexural performance. Instead, reducing CO2 concentration to 50% seemed to produce better flexural performance characteristics compared to those obtained with control specimens (i.e. 0% CO2 concentration) and also with the higher concentration, but with similar chamber duration, one hr. The long time effect of CO2 curing (i.e. 4 hrs chamber duration) seems to have the same effect on both carbon (see Fig. 6C), and glass fibers (see Fig. 7C). It did not add anything useful to the final performance, when compared with control specimens (Figs. 6A and 7A).

Furthermore, all tested specimens behaved elastically up until the peak flexural strength (Pmax). Beyond the Pmax the initiated cracking growth and leading to a separation of the board into two parts. It is also noted that for glass fiber type, the post peak part of the load deflection curve drops down while for carbon fibers it decreases relatively slowly providing some enhancement to resultant toughness. Figs. 7A,B provide good examples for this behavior.

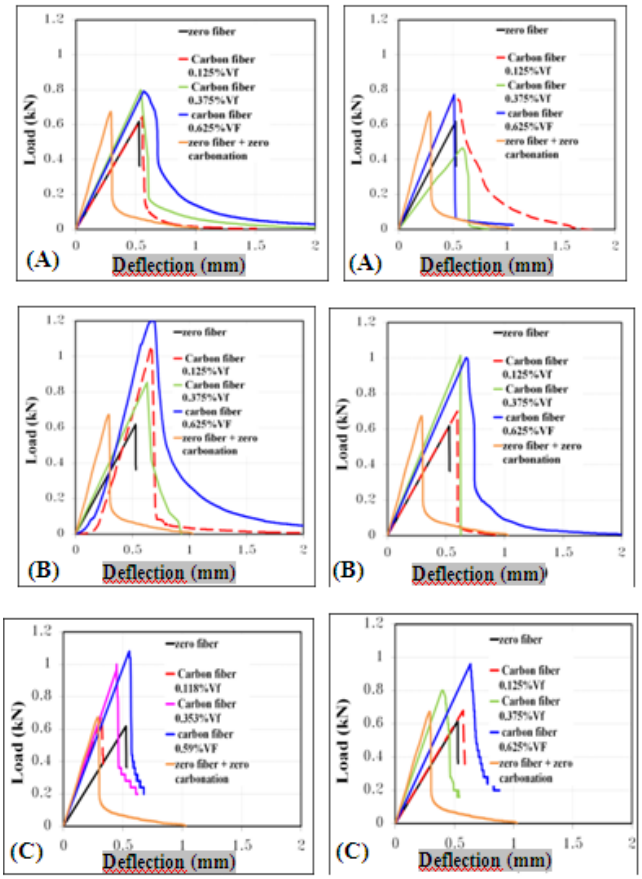

Fig. 7- Typical Load deflection curve characteristics of cement-bonded glass fiberboard; (A) 100%carbonation 1 hr, (B) 50% carbonation 1 hr, (C) 50% carbonation 4hrs; Left side 4mm and Right side 9mm fiber length.

The improvements in flexural performance were more pronounced in higher fiber/matrix ratio. Any improvements in the flexural properties (i.e. flexural strength and toughness) will depend on whether fibers bridging the cracks are able to

The strain in the composite at a given stress depends on the length of debonded fibers and, hence, a greater bond leads to raising the peak flexural force Pmax. Glass fibers due to the low tensile strength are expected to be cut off rather than pulled out only, while carbon fibers showed better performance under flexure test due to its relatively higher tensile strength (i.e. pulled out only). This behavior probably interprets the enhancement in flexural properties associated with CO2 curing combined with higher fiber content.

Fracture surface observations

Fig. 8 depicts the SEM images of the fractured surface of the fiber reinforced cementitious composites. Samples taken from the lower tension fracture zone of tested boards under flexural load. The SEM micrographs used here are typical images of the microstructure observed from around overall twenty images for each composite treatment. The analysis of these micrographs allows the observation of the cement phases developed after the exposition to accelerated carbonation, and their impact on the interface between cement past and other constituents.

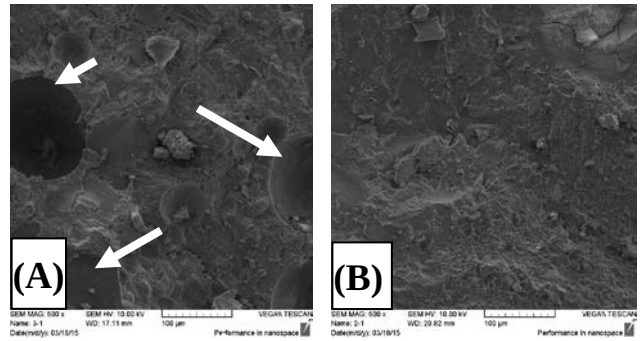

Figure 8- SEM micrographs of the fractured surface of fiber-cement composites, (A) non-carbonated, white arrows indicate poor and debonded areas, (B) CO2 cured, black arrows indicate denser areas.

Fig. 8 indicats that debonded areas (indicated by the white arrows) appear to occupy higher percentage in non-carbonated specimens compared to the tested specimens subjected to CO_2-curing. Differently from the non-carbonated composites, the microstructure in the accelerated carbonated composites, (**Fig. 8B**), is compact and formed by layered structures (black arrow), probably related to the $CaCO_3$ phases produced due to the carbonation chemical reactions.

IV. Conclusions

An experimental study was conducted to assess the processing aspects of CO2-cured cementitious boards on the flexural performance of 28-day age of carbon /or glass fiber-reinforced cement composites, and to develop an efficient processing approaches which makes value-added use of carbon dioxide and/ or synthetic fibers. Two fiber lengths and three fiber/matrix ratios were evaluated. The performance characteristics were evaluated through flexural testing of composites and compared with control specimens. The results indicate that:

- Tested manufacturing parameters (i.e. CO2-curing concentrations, chamber duration, fiber/matrix ratio, and fiber lengths) had statistically significant effects on the flexural performance of the end product.
- CO2-curing increases matrix densities and reduces the debonded areas in the carbonated specimens. The enhancement in microstructure features were more pronounced in the 50% gas concentration.
- Higher fiber/matrix ratio and lower CO2-curing concentration were chosen as the preferred conditions, regardless the fiber types.
- Longer chamber duration of CO2-curing did not benefit the final performance of tested specimens, when compared with control specimens.
- Carbon fibers type combined with shorter fiber length seem to have preferred flexural performance of cementitious boards, when compared with counter glass fibers type

REFERENCES

[1] ACI committee 544. "State of the art report on fiber reinforced concrete," Concr Int, vol. 5, pp. 9–30, 1982.

[2] ACI committee 544. "Guide for specifying, mixing, placing and finishing steel fiber reinforced concrete," ACI Mater J, vol. 90no. 1, pp. 94–101, 1993.

[3] ACI committee 544. "Measurement of the properties of fiber reinforced concrete," ACI Mater J, vol. 6, pp.583–9, 1985.

[4] Najamj T, Dwarakanath H. "Structural response of partially fibrous concrete beams," Struct Eng, vol. 111, no. 11, pp. 2798–812, 1984.

[5] Rahimi M, Kesler C. "Partially steel-fiber reinforced mortar," Struct Eng, vol. 105, no. 1, pp. 101–9, 1975.

[6] Swamy R, Ai-Taan S. "Deformation and ultimate strength in flexure of reinforced concrete beams made with steel fiber concrete," ACI Mater J, vol. 78, no. 5, pp. 395–405, 1981.

[7] Dinwoodie, J.M. "Wood–cement particleboards-a technical assessment. Building Research Establishment," Information Paper, United Kingdom, April 1983.

[8] Soroushian, P., Won, J.-P., and Hassan, M.S., "Sustainable Processing of Cellulose Fiber Cement Composites," ACI Materials Journal, vol. 110 , no. 3, pp. 305-314, 2013.

[9] Soroushian, P., Won, J.-P., Chowdhury, H., and Nossoni, A., "Development of Accelerated Processing Techniques for Cement-Bonded Wood Particleboard," Cement and Concrete Composites, vol. 25, pp. 721-727, 2003.

[10] Soroushian, P., Won, J.-P. and Hassan, M.S., "Durability and microstructure analysis Characteristics of CO2-Cured cement-bonded wood particleboard," Cement and Concrete Composites, vol. 41, pp. 34-44, 2013.

[11] Alanbari, Riyad H., Hassan, Maan S., and Fakhri, Ali H., "Manufacturing of sustainable cellulose date palm fiber reinforced cementitious boards in Iraq", Engineering and Technology Journal, Article in press, UOT, Baghdad, Iraq, 2015.

[12] Tonoli, G.H.D., Santos, S.F., Joaquim, A.P. and Savastano Jr., H., 'Effect of accelerated carbonation on cementitious roofing tiles reinforced with lignocellulosic fibre', Construction and Building Materials 24 (2010) 193-201.

[13] ASTM C1185-12 "Standard Test Methods for Sampling and Testing Non-Asbestos Fiber-Cement Flat Sheet, Roofing and Siding Shingles, and Clapboards," ASTM International, West Conshohocken, PA, 2012.

[14] Hannant, D. J. Fibre Cements and Fibre Concretes, (John Wiley & Sons, Inc., New York, 1978).

[15] Pizzol, V.D., Mendes, L.M., Frezzatti, L., Savastano Jr., H., and Tonoli, G.H.D. "Effect of accelerated carbonation on the microstructure and physical properties of hybrid fiber-cement composites," Minerals Engineering, vol. 59, pp. 101-106, 2014.

[16] Soroushian, P., Won, J.-P., and Hassan, M.S., "Durability Characteristics of CO2-Cured Cellulose Fiber Reinforced Cement Composites," Construction & Building Materials, vol. 34, pp. 44-53, 2012.

Biography:

ACI member Maan S. Hassan is an Assistant Professor of Building and Construction Engineering at University of Technology, (Baghdad, Iraq). He has conducted post-doctoral research at Leeds University (UK), and has also been a Fulbright researcher at Michigan State University (USA). He received his Ph.D. in 2007 from University of Technology (Iraq), and his specialty is in Concrete Materials and technology. He has published broadly in this area and currently serves as President of ACI Iraq Chapter.

The Influence of Coarse Aggregate Size, Type and Content on the Mechanical Properties of Modified Reactive Powder Concrete

Lect. Dr. Layth Al-Jaberi,

***Abstract*—The objective of this search is to study the impact of the maximum size, type and content of coarse aggregate (CA) on the mechanical properties of Modified Reactive Powder Concrete (MPRC) in terms of compressive (fcu) and flexural (fr) strengths. (32) Mixes of MRPC are designed, mixed, molded and tested. The results showed that high (fcu) can be achieved by involvingthe CA in the mixes of MRPC; this outcome is in contrast with the model offered to relate the high compressive strength level of Reactive Powder Concrete(RPC) to the absence of CA.Increasing the maximum size of CA lead to decrease the values of (fcu).The results of MPRC mixes with crushed CA is better than these mixes with rounded CA. The optimum CA/Total aggregate percentage in MRPC mixes is 45% for (fcu). In addition, it is observed that increase of CA content will increase (fr).**

***Index Terms*—MRPC, Compressive, Flexural, strength, Aggregate.**

I. Introduction

Many researches have been made to figure out the ultimate strength limit of the cement-based materials (1). Ultra High Performance Concrete (UHPC) is one of the most important kinds of concrete. UHPC mix contain cement, very fine sand (instead of ordinary aggregate which consist of fine aggregate (FA) and coarse aggregate (CA)), silica fume, fibers, High-Range Water-Reducing Admixture (HRWRA), with very low water-cement ratio (w/c) (2). The cement content in UHPC is as high as (900-1000 kg/m^3); this high content is because of involving very FA instead of ordinary aggregate (3). The loss of CA rated by the designers to be the main or most influential reasons for the microstructure and the performance of the UHPC in order to reduce heterogeneity between the cement paste and the aggregate (4).Nevertheless, the CA plays an important role in concrete. There are three main reasons for mixing aggregate with cement paste to form concrete, rather than using cement paste alone. Reducing the cost of the concrete mix is one of those reasons. Aggregate is cheaper than cement and its use lead to reduce costs. In addition, aggregate minimizes shrinkage and creep, affording better mass stability. Moreover, greater durability of concrete achieved due to aggregate (5).

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Lect. Dr. Layth Al-Jaberi is a faculty member of the Civil Engineering Department at College of Engineering-Al -Mustansiriya University.Ph. D., Civil Engineering Material from Baghdad University (Baghdad, Iraq), at 2012. M.Sc., Civil Engineering Materials, from Al-Mustansirya University, Baghdad, Iraq at June 2005.B.S., Civil Engineering from Military Engineering College, Baghdad, Iraq at June 1987. His expertise in Civil Engineering Materials. He published ten research. (e-mail: laithjabri@yahoo.com).

II. MODIFIED REACTIVE POWDER CONCRETE (MRPC)

Reactive Powder Concrete(RPC), which is one of the newest kinds of UHPC, is characterized by a very dense matrix thanks to the improvement of the granular packing of the dry fine powders and a firm microstructure. Thus, RPC is characterized with ultra-high strength and durability. It can be said that, RPC is not a concrete because there is no CA in the mix. In order to reduce its cost and to get other technical benefits through using CA in the mix, many successful researches have been made to involve CA in RPC mixes. This type of concrete mixes is termed as MRPC. Collepardi et al. (6) compared the mechanical properties of RPC with the same properties of MRPC where a graded CA (maximum size 8mm) was used to replace the fine sand and/or part of the cementitious binder. Different curing conditions were followed for both RPC and MRPC mixes. The important indication from the results of this investigation was that the inclusion of CA dose not reduce the compressive strength provided that the quality of the cement matrix, in terms of its water-cement ratio, is not changed.Wasan I. Khalil (7) studied some properties of MRPC where crushed graded natural aggregate (maximum size 12.5 mm) was used. A high compressive strength of 150 MPa was achieved. The results of this study was in contrast with the model proposed to relate the high level of compressive strength of RPC to the absence of CA. Sujatha and Basanthi (9) studied the mechanical properties of MRPC mixes that made with the introduction of graded aggregate (3-8mm) to RPC mixes. In this study compressive and flexural strengths of RPC with MRPC are compared by taking different proportions of aggregates by varying w/c ratios. From the results obtained of compressive and flexural strengths of the MRPC mixtures of 30%, 40% and 50% (by weight) replacement on quartz sand by graded aggregate are compared to the Original RPC. It was found through this study that optimum amount of the quartz sand to be replaced by the graded aggregate is around 40% for compressive strength. In addition, it was observed that increase in replacement of aggregates would increase the flexural strength.

III. RESEARCH SIGNIFICANCE

As illustrated above several researches had been made to investigate the effects of CA characteristics on the performance of MRPC. However, the circumstances of these researches such as the properties of the materials that used, their proportions in the mixes, the methods of curing and tests are not same for all. Thus, an accurate judgment on those effects cannot achieved. The present study aims to investigate the influences of maximum size, type and content of CA on the both compressive and flexural strength of MRPC. Except these factors, others are hold constants. Locally natural CA from one region is used for the all mixes. This research provides data for the researchers concerning with the main properties of MRPC processing using economical available materials.

IV. EXPERIMENTAL PROGRAM

A. Materials

The cement used in this study is Iraqi ordinary Portland cement (Taslogah) type (I). This cement is tested and checked according to IOS 5:1984. (10). To avoid any an undesirable feature in extra fine sand such as high content of salts, high percentage of excessive fines (clay, silt and/or dust) or others; Anti-Slip Sand No. 4, chemically inert, graded, hardwearing aggregate with size (300-600) µm is used in this work. This extra fine sand is one of Don Construction Products. Its grading satisfies the fine grading in accordance with the Iraqi Specification No.45/1984 zone 4 (11). Crushed and rounded gravel from AL-Nibaey region was used with maximum sizes of 5mm, 8mm, 10mm and 12mm. This gravel was washed, then stored in air for surface drying, and then stored in containers in a saturated surface dry condition before using. The test results show that the CA characteristics are within the limit specified by Iraqi standard specification No.45/1984 (11). Tap water is used for both mixing and curing of concrete. SikaFume is used as Silica fume. SikaFume is a concrete additive based on powered Sika Silica Fume Technology. It is a highly reactive pozzolanic material suitable for high quality concrete. It meets the requirements of BS EN 13263-1:2005. SikaFume contains extremely fine (0.15 µm; specific surface 20 m2/gm) latently reactive silicon dioxide. A copolymer-based HRWRA, designed for the production of HPC is used (Glenium 51). Ultra-fine steel fibers is used throughout the experimental program. The properties of the used MSF are presented in Table (1).

TABLE (1): PROPERTIES OF THE USED STEEL FIBERS

Property	Specifications	Property	Specifications
Type	SF 0213	Form	Straight
Surface	Brass coated	Average length	13 mm
Relative Density	0 Kg/m3	Diameter	0.2mm±0.05mm
Tensile Strength	Minimum 2300MPa	Aspect ratio (Lf/Df)	65

B. Concrete Mixes

In order to achieve the scope of this study, the work is divided into (2) sets of mixes include (32)MRPC mixes. The mixes are designed to differ among themselves as shown in Table (2). The details of the mix proportions is shown in Table (3).

TABLE 2 – SETS AND DESCRIPTION OF MIXES

Sets	Mix Symbol	Description of Mix	Sets	Mix Symbol	Description of Mix
Rounded	MRPC1	40% (by mass of aggregate) rounded CA max. size 5mm	Crushed	MRPC 17	40% (by mass of aggregate) crushed CA max. size 5mm
	MRPC2	40% (by mass of aggregate) rounded CA max. size 8mm		MRPC 18	40% (by mass of aggregate) crushed CA max. size 8mm
	MRPC3	40% (by mass of aggregate) rounded CA max. size 10mm		MRPC 19	40% (by mass of aggregate) crushed CA max. size 10mm
	MRPC4	40% (by mass of aggregate) rounded CA max. size 12mm		MRPC 20	40% (by mass of aggregate) crushed CA max. size 12mm
	MRPC5	45% (by mass of aggregate) rounded CA max. size 5mm		MRPC 21	45% (by mass of aggregate) crushed CA max. size 5mm
	MRPC6	45% (by mass of aggregate) rounded CA max. size 8mm		MRPC 22	45% (by mass of aggregate) crushed CA max. size 8mm
	MRPC7	45% (by mass of aggregate) rounded CA max. size 10mm		MRPC 23	45% (by mass of aggregate) crushed CA max. size 10mm
	MRPC8	45% (by mass of aggregate) rounded CA max. size 12mm		MRPC 24	45% (by mass of aggregate) crushed CA max. size 12mm
	MRPC9	47.5% (by mass of aggregate) rounded CA max. size 5mm		MPRC 25	47.5% (by mass of aggregate) rounded CA max. size 5mm
	MRPC10	47.5% (by mass of aggregate) rounded CA max. size 8mm		MRPC 26	47.5% (by mass of aggregate) crushed CA max. size 8mm
	MRPC11	47.5% (by mass of aggregate) rounded CA max. size 10mm		MRPC 27	47.5% (by mass of aggregate) crushed CA max. size 10mm
	MRPC12	47.5% (by mass of aggregate) rounded CA max. size 12mm		MRPC 28	47.5% (by mass of aggregate) crushed CA max. size 12mm
	MRPC13	50% (by mass of aggregate) rounded CA max. size 5mm		MRPC 29	50% (by mass of aggregate) crushed CA max. size 5mm
	MRPC14	50% (by mass of aggregate) rounded CA max. size 8mm		MRPC 30	50% (by mass of aggregate) crushed CA max. size 8mm
	MRPC15	50% (by mass of aggregate) rounded CA max. size 10mm		MRPC 31	50% (by mass of aggregate) crushed CA max. size 10mm
	MRPC16	50% (by mass of aggregate) rounded CA max. size 12mm		MRPC 32	50% (by mass of aggregate) crushed CA max. size 12mm

TABLE 3 – DETAILS OF MIX PROPORTIONS

w/c ratio	Cement Kg/m^3	Total Aggregate Kg/m^3	Silica FumeKg/m^3	Super-Plasticizer % of cement mass	Micro Steel Fibers V_f %
0.18	933	1030	234	5	1

C. Experimental Procedure

In this study, compressive and flexural strengths of MRPC are compared by taking different maximum sizes, types and contents of CA by keeping everything else constant. The strengths are tested by using at 7 and 28 days by ordinary curing.The size of cubes (Compressive Strength Test) is casted by the 100 X 100 X 100 mm. The tests are carried out by 3000 KN capacity machine. The average value of the three specimens for each mix and age is determined and recorded.

The flexural strength, expressed as the modulus of rupture, is calculated using the results obtained from a simple beam with third-point loading test according to ASTM C78-84 (14). Test prisms measuring 100×100×500 mm are prepared according to ASTM C 192-88 (15). ELE testing machine of 50KN capacity is used for testing prisms. Flexural strength (MPa) is obtained by averaging the very close two results.

V. RESULTS AND DISCUSSION

A. Compressive Strengths

The compressive strength, as one of the most important properties of hardened concrete, is in general the characteristic material value for the classification of concrete in national and international codes. For this reason, it is of interest to investigate whether the changes in the mixture composition and positive dissimilarities in the microstructure, as mentioned before, affect the early and later compressive strengths. Tables (4&5) and Figures (1-8) show the average results of the compressive strength (fcu) tests at 7 and 28 days gained from tests.

The results indicate that high levels of (fcu) can be achieved through using CA. This indication confirms the contradictionwith the model, which explain the high compressive strength grade of RPC byexcluding of CA from the mix. The tables and figures clarify that increasing the maximum size of CA lead to decrease the values of (fcu). However, the degree of decrement differs from size to size. Values of (fcu) for mixes with 8mm maximum size of CA show slightly decrement from those mixes with 5mm maximum size of CA. More decrement is exhibiting in the values of (fcu) for both mixes with 10mm maximum size of CA and those with 12mm maximum size of CA. The results, also, show that the optimum percentage of (CA content / the total content of aggregate) in the mix is 45% for (fcu). Increasing percentage more than 45% of CA content lead to decrease level of (fcu). Mixes contain crushed CA have better results from those contain rounded CA.

TABLE 4 – RESULTS OF COMPRESSIVE STRENGTH (FCU) FOR MRPC MIXES@ 7 DAYS

Mix	fcu	Mix	fcu	Mix	fcu	Mix	fcu
MRPC1	86.0	MRPC5	88.6	MRPC9	85.0	MRPC13	83.0
MRPC2	84.1	MRPC6	86.0	MRPC10	83.1	MRPC14	81.0
MRPC3	81.8	MRPC7	84.8	MRPC11	82.0	MRPC15	79.0
MRPC4	80.4	MRPC8	83.0	MRPC12	81.3	MRPC16	77.0
MRPC17	105.6	MRPC21	109.8	MPRC25	104.2	MRPC29	101.1
MRPC18	103.5	MRPC22	103.9	MRPC26	102.6	MRPC30	97.9
MRPC19	93.1	MRPC23	102.2	MRPC27	91.2	MRPC31	87.8
MRPC20	83.7	MRPC24	91.9	MRPC28	83.9	MRPC32	80.0

TABLE 5 – RESULTS OF COMPRESSIVE STRENGTH (FCU) FOR MRPC MIXES @ 28 DAYS

Mix	fcu	Mix	fcu	Mix	fcu	Mix	fcu
MRPC1	110.9	MRPC5	113.1	MRPC9	108.0	MRPC13	105.0
MRPC2	108.0	MRPC6	110.4	MRPC10	107.0	MRPC14	104.0
MRPC3	105.4	MRPC7	108.2	MRPC11	103.9	MRPC15	100.6
MRPC4	104.8	MRPC8	105.1	MRPC12	103.0	MRPC16	98.8
MRPC17	131.6	MRPC21	132.5	MPRC25	129.7	MRPC29	127.9
MRPC18	125.8	MRPC22	129.4	MRPC26	125.7	MRPC30	124.2
MRPC19	117.7	MRPC23	124.7	MRPC27	113.5	MRPC31	109.9
MRPC20	106.6	MRPC24	113.9	MRPC28	105.5	MRPC32	101.0

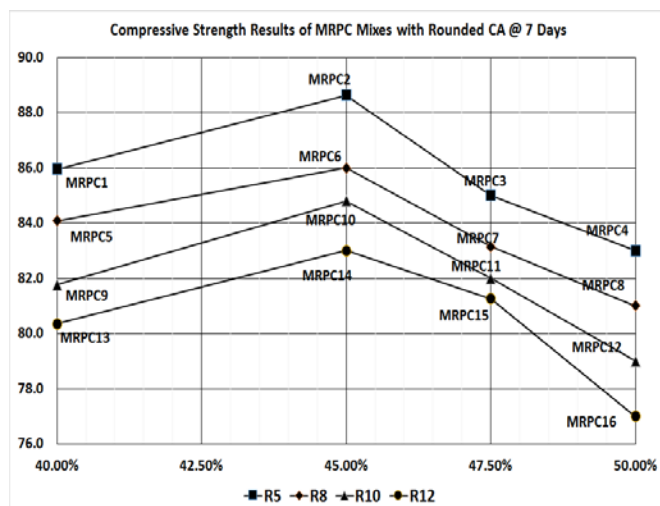


Fig. (1): Compressive Strength of MRPC Mixes with Rounded CA @ 7 Days

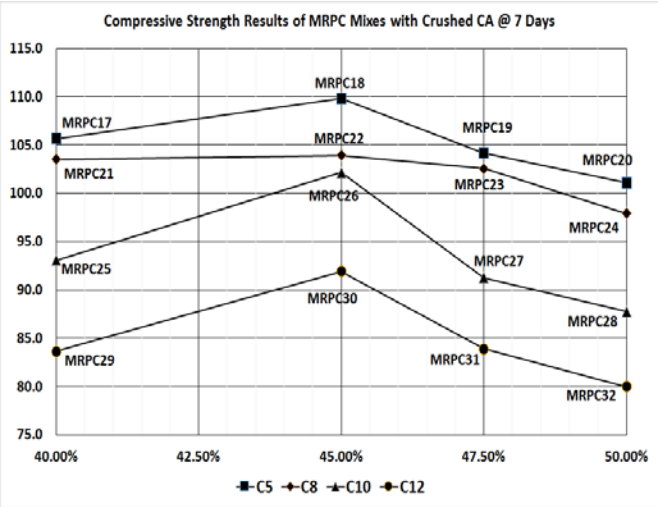


Fig. (2): Compressive Strength of MRPC Mixes with Crushed CA @ 7 Days

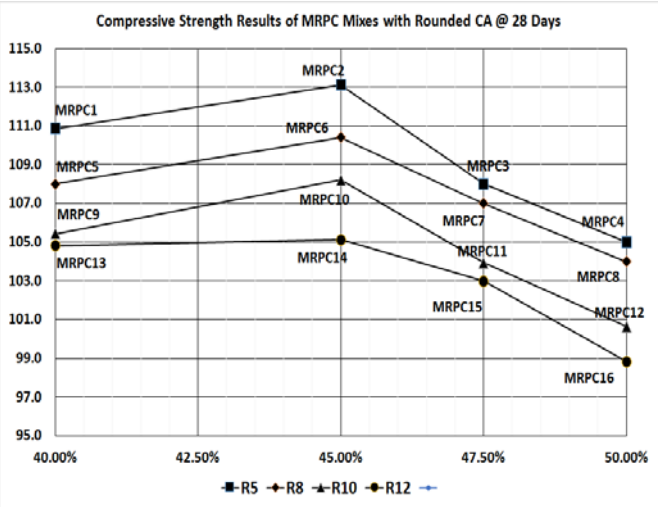


Fig. (3): Compressive Strength of MRPC Mixes with Rounded CA @ 28 Days

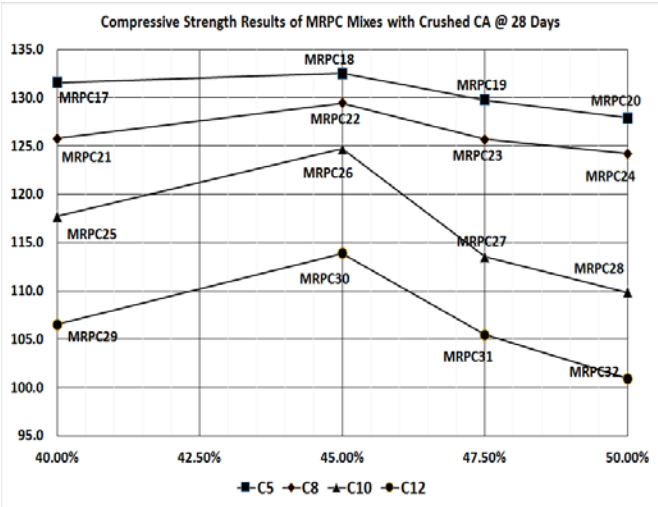


Fig. (4): Compressive Strength of MRPC Mixes with Crushed CA @ 28 Days

B. Flexural Strengths

The results of flexural strength (fr) tests are listed in Tables (9&10) and plotted in Figures(5-8). The results show that the values of (fr) increased with the increasing of the maximum size of CA. It is very clear that increase in replacement of fine aggregates (FA) with CA will increase the flexural strength. The effect of type of CA on the values of (fr) is same as its effect on the values of (fcu). Mixes with crushed CA has the better values of (fr).

TABLE 9 – RESULTS OF FLEXURAL STRENGTH (FR) FOR MRPC MIXES @ 7 DAYS

Mix	fcu	Mix	fcu	Mix	fcu	Mix	fcu
MRPC1	8.0	MRPC5	9.0	MRPC9	9.0	MRPC13	10.0
MRPC2	9.0	MRPC6	10.0	MRPC10	12.0	MRPC14	14.0
MRPC3	11.0	MRPC7	12.0	MRPC11	13.0	MRPC15	16.0
MRPC4	13.0	MRPC8	14.0	MRPC12	15.0	MRPC16	18.0
MRPC17	10.0	MRPC21	12.0	MPRC25	13.0	MRPC29	15.0
MRPC18	11.0	MRPC22	13.0	MRPC26	16.0	MRPC30	17.0
MRPC19	13.0	MRPC23	14.0	MRPC27	17.0	MRPC31	19.0
MRPC20	14.0	MRPC24	16.0	MRPC28	19.0	MRPC32	20.0

TABLE 10 – RESULTS OF FLEXURAL STRENGTH (FCU) FOR MRPC MIXES @ 28 DAYS

Mix	fcu	Mix	fcu	Mix	fcu	Mix	fcu
MRPC1	12	MRPC5	13	MRPC9	14	MRPC13	15
MRPC2	13	MRPC6	15	MRPC10	17	MRPC14	20
MRPC3	16	MRPC7	19	MRPC11	20	MRPC15	23
MRPC4	18	MRPC8	20	MRPC12	23	MRPC16	25
MRPC17	15	MRPC21	19	MPRC25	20	MRPC29	22
MRPC18	16	MRPC22	20	MRPC26	23	MRPC30	25
MRPC19	18	MRPC23	22	MRPC27	25	MRPC31	27
MRPC20	21	MRPC24	23	MRPC28	26	MRPC32	28

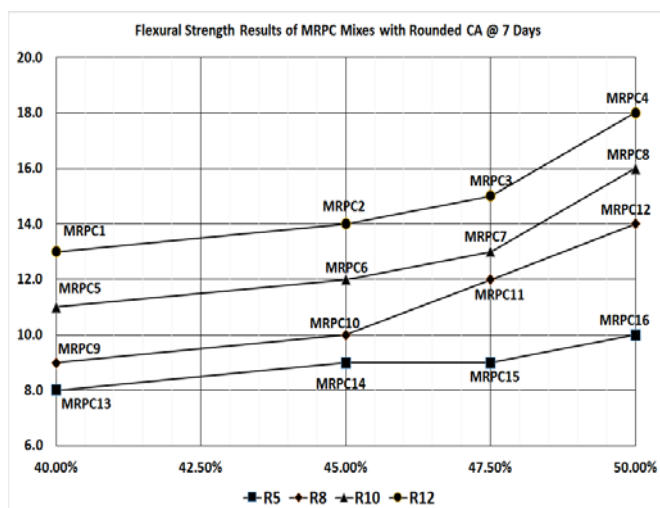


Fig. (5): Flexural Strength of MRPC Mixes with Rounded CA @ 7 Days

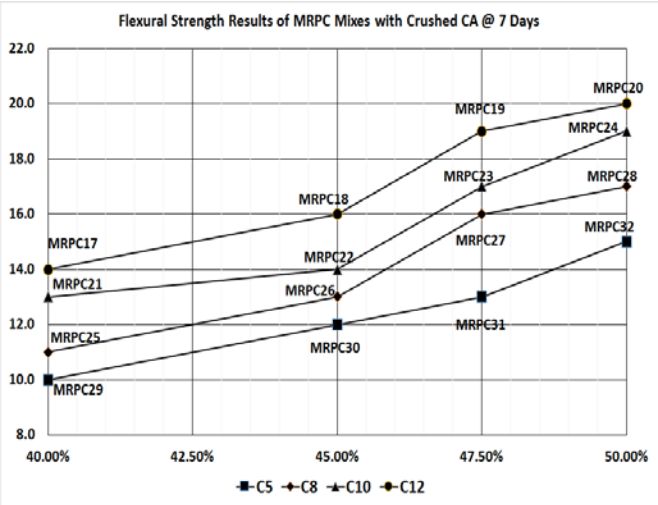


Fig. (6): Flexural Strength of MRPC Mixes with Crushed CA @ 7 Days

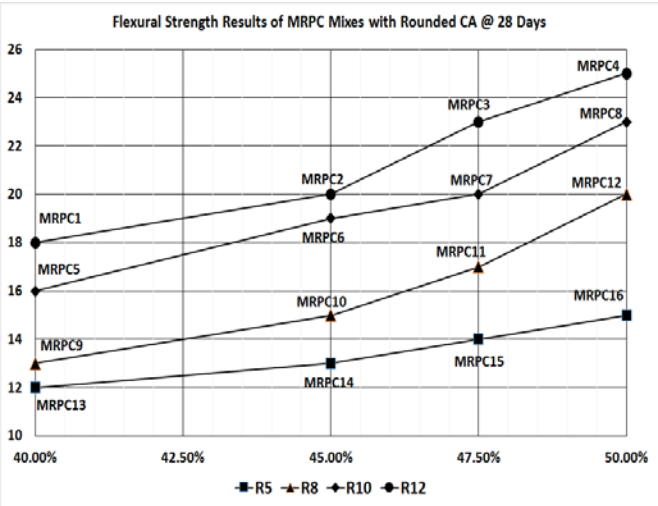


Fig. (8): Flexural Strength of MRPC Mixes with Crushed CA @ 28 Days

VI. Conclusions

According to the results of this research, the following conclusions can be drawn:

1- Inclusion of CA as a replacement of FA in the model of MRPC is satisfactory and the produced concrete can achieved high level of mechanical properties.
2- Increasing the maximum size of CA lead to decrease the values of (fcu).
3- The optimum CA/Total aggregate percentage in MRPC mixes is 45%.
4- Crushed CA has a better effect on both of the values of (fcu) and (fr).
5- A direct variation is noticed between the values of (fcu) and the maximum size of CA.

References

[1] Aitcin P., “High-Performance Concrete”, First Published (1998).
[2] A. R. Lubbers. 2003. "Bond Performance between Ultra-High Performance Concrete and Prestressing Strands". Faculty of Fritz, College of Engineering and Technology of Ohio University. pp. 155-165.
[3] Neville, A.M., “ Properties of Concrete ”, Pitman publishing, fourth and final edition, London, (2000).
[4] Richard P., Cheyrezy M., "Composition of Reactive Powder Concretes", Cement and Concrete Research, Vol. 25, No. 7, pp. 1501-1511, 1995.
[5] John Lay, "The effects of natural aggregates on the properties of concrete", Advanced Concrete Technology 1 p
[6] Collepardi, S. , Coppola, L., Troli, R., and Collepardi, M., "Mechanical Properties of Modified Reactive Powder Concrete", www.encoserl.it.
[7] Asst. Prof. Dr. Wasan I. Khalil, " Some Properties of Modified Reactive Powder Concrete", Journal of Engineering and Development, Vol. 16, No.4, Dec. 2012 ISSN 1813- 7822.
[8] T.Sujatha and D.Basanthi, " Modified Reactive Powder Concrete", IJEAR Vol. 4, Issue Spl-2, Jan - June 2014.
[9] IOS No. 5 : 1984, Iraqi Cement Standard for Portland Cement.
[10] Iraqi specification No.45/1984, Natural Aggregate for concrete and Buildings.
[11] ASTM C 1240-05, “Standard Specification Silica Fume Used in Cementitious Mixtures”, Manual of A04-02, October, (2007).
[12] ASTM C78-84, “Test Method for Flexural strength of Concrete (Using Simple Beam with Third-Point Loading),” ASTM International.
[13] ASTM C192-88, “Practice for Making and Curing Concrete Test Specimens in Laboratory”, ASTM International.

The Effect of Firing Temperatures on Some Physical & Mechanical Properties of Clay Bricks Containing Attapulgite

Ahmed A. Hussein, Waleed A. Abbas and Qais J. Frayyeh

***Abstract*—The present paper investigates the ability of using Iraqi Attapulgite clays of 10%, 20%, and 30% as partial replacement by weight of the clay used in the manufacturing local clay bricks, to produce the laboratory bricks with dimensions (75×38×25) mm & (175×38×25) mm laboratory extrusion device was used. Then they were dried in laboratory temperature and fired in electrical furnace with different degrees (750, 800, 850, 900, 950, &1000) ℃.**

Some of the mechanical properties of that clay bricks (longitudinal shrinkage, bulk density, compressive strength, efflorescence, water absorption, and modulus of rupture) has been studied. The results showed that the addition of Attapulgite clays with 20% by weight of clay noteworthy improvement in the mechanical & physical properties of laboratory bricks.

***Index Terms*—A Clay bricks, firing temp. , Attapulgite, mechanical properties of clay bricks.**

I. Introduction

The current addressing in the world & in our country especially towards keeping & economy in power and decreasing of its consumption, common building materials of concrete or bricks or others are considered good as durability or strength to raise of its concentration [1]. And used it in Mesopotamia in the ancient times, where bricks are considered one of most important of basic materials to construction purposes, where its uses are common in Mesopotamia since old ages till our current days [2].

From all these information many researches have made on bricks, more of researchers who adopted in their researches clay bricks. In our current research some of mechanical & physical properties of clay bricks for lab samples with different firing temperatures using clay additives (Attapulgite Clays) & comparison them with reference soil to state possibility of gaining clay bricks with same specifications or better but with low firing temperatures to secure saving energy.

II EXPERIMENTAL

A. Grain Size distribution of soil:

In this research we used soil from Al-Nahrawan city in Baghdad, dry sieving and hydrometer method used according to British specifications [3], as shown in table (1).

Table (1): Grain size distribution of reference soil

No.	Grain Size(mm)	Content in reference soil %
1	sand particles (≥ 0.02)	20
2	silt particles (0.002-0.02)	46
3	Clay particles (≤ 0.002)	32

The results showed that reference soil used was alluvial, it is an appropriate type for the production of clay bricks.

B. Chemical Analysis of soil

Table (2) show the chemical analysis for the reference soil & table (3) show the chemical analysis for the Attapulgite clay after grinding.

Table (2): Chemical analysis of the reference soil

N o	Chemical symbol	Content %
1	SiO_2	41.64
2	Fe_2O_3	5.7
3	Al_2O_3	12.38
4	CaO	13.37
5	MgO	5.39
6	SO_3	0.56
7	CL	0.41
8	NaO_2	1.04
9	K_2O	1.79
10	L.O.I	17.1

Table (3): Chemical Analysis of Attapulgite after grinding

NO	Chemical symbol	Content %
1	**SiO_2**	**51.8**
2	**Al_2O_3**	**8.99**
3	**Fe_2O_3**	**4.88**
4	**TiO_2**	**0.58**
5	**CaO**	**7.11**
6	**MgO**	**5.52**
7	**SO_3**	**0.68**
8	**Na_2O**	**1.01**
9	**K_2O**	**1.8**
10	**L.O.I**	**17.21**

C Preparing mixtures

Five kg. of soil mixed properly with the specified water content (measured by Pfefferkoorn method)[3], for 15 minutes to get required consistency.

D Samples Formation

Samples of bricks were made using a laboratory extruder machine (fig.1) with dimensions shown in Table 4.

Figure (1): Laboratory extruder machine

Table(4) Samples Dimensions

Dimensions	Purpose
25x38x75 mm	For Mechanical & Physical Tests
25x38x175 mm	For Modulus of Rupture Test

E **Samples Drying**

After formation, samples were weighted in a laboratory balance has accuracy of (0.1 gm.) and measuring it's dimensions by Vernier Caliper, then samples left to be dried in air for complete week. Then dried in un oven drying for 24 hours in 110 ^oC [6].

F Samples Firing

The firing temperatures of samples used are (750 , 800 , 850 , 900 , 950 , 1000) ^oC, where samples were arranged inside electrical furnace of maximum temperature of 1200 ^oC as shown in (fig.2), with firing rate of 4 ^oC per minute for soaking time one hour to give enough time to burn all organic materials, releasing second oxide calcium, after completing process of firing, samples were left inside furnace to cool gradually for 24 hrs. inside the furnace .

Fig(2) Arranging Samples inside furnace

III Testing after firing

A. Firing longitudinal shrinkage (L.S.).

Longitudinal shrinkage were calculated as follows [3]:

L.S % = (Hd – Hf) / H d x 100 were:

L.S. = Longitudinal Shrinkage

H d = sample length before firing (mm)

H f = sample length after firing (mm) .

B. Firing Bulk density

Bulk density were calculated as follows [6].

B.D = Wf / Vf were:

B. D. = Bulk density (gm. / cm3) .

W f = weight of sample after burning (g.) .

V f = volume of sample after burning (cm^3) .

C. Firing Compressive Strength

Compressive Strength were calculated as follows [7]:

Compressive Strength (N/ mm2) = P/A were:

P =Load (N).

A = Area (mm^2).

D. Efflorescence

Efflorescence were calculated according to [7].

Water absorption = (Ws– Wd) / Wd x 100 were :
Ws = weight of sample after sinking it in water (g).
Wd = weight of sample before sinking it in water (g).

F. Modulus of Rupture

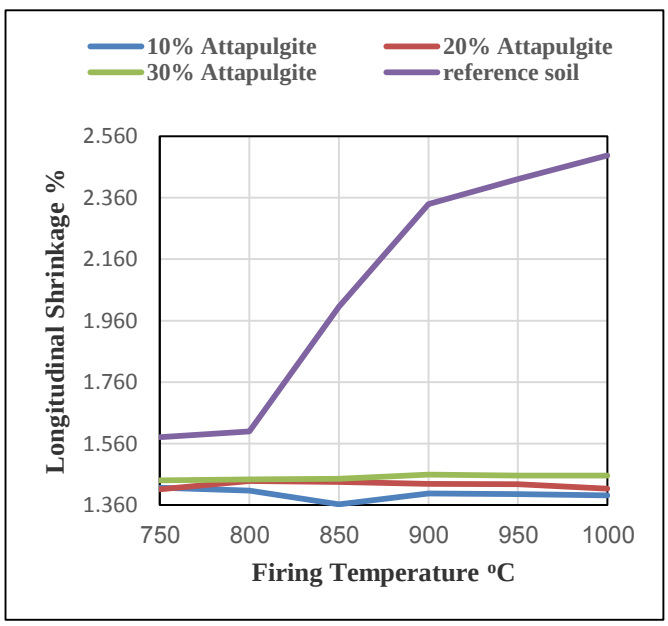
Modulus of rupture were calculated as follows [3].
Modulus of rupture (N/mm^2) = 3PL / 2bd2 were:
P = breaking load (N).
L = distance between supports (mm).
b = sample width (mm).
d = sample thickness (mm).

IV. Results & Discussion

A. Longitudinal shrinkage

Fig.3 below represents the effect of firing temperature and replacement % of Attapulgite on the longitudinal shrinkage of clay bricks, notice that a significant decrease in longitudinal shrinkage with using Attapulgite clays reached to (10.1%, 10.1%, 28.4%, 38.9%, 40.9%, and43.3%) with firing temperature of (750,800,850,900,950, and 1000) oC respectively.

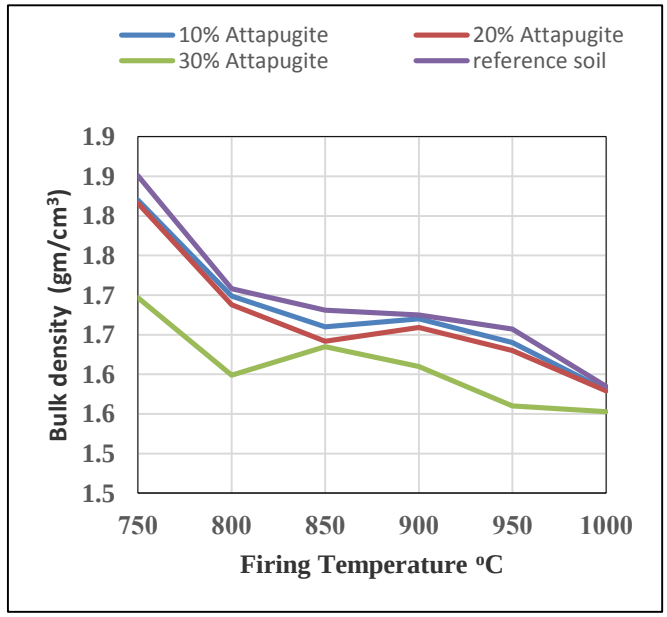
Firing shrinkage increases with higher temperatures, to obtain products of uniform size, manufacturers control factors contributing to shrinkage. Because of normal variations in raw materials and temperature variations within kilns, absolute uniformity is impossible. Consequently, specifications for brick allow size variations [8].



Figure(3) Effect of firing temperature and replacement % of Attapulgite on the longitudinal shrinkage of clay bricks

B. Bulk density

Fig. 4 below represents the effect of firing temperature and replacement % of Attapulgite on the bulk density of clay

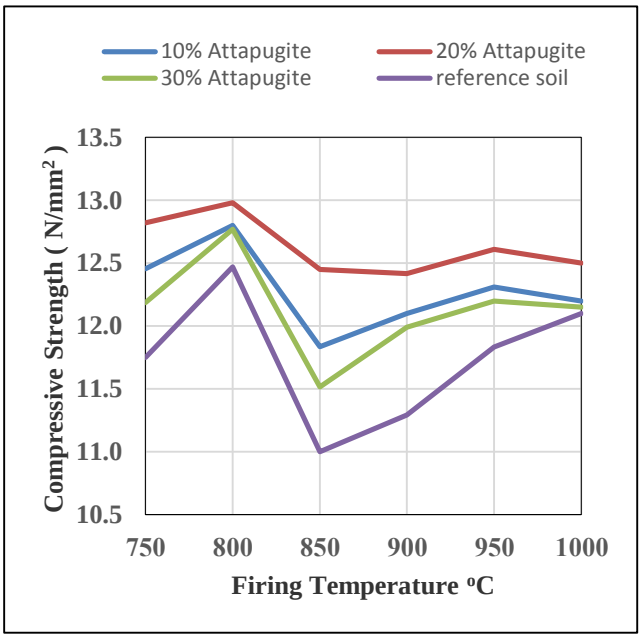


Figure(4) Effect of firing temperature and replacement % of Attapulgite on the bulk density of clay bricks

bricks, it is clearly that there is a significant decrease in bulk density with using Attapulgite clays reached to (1.9%,1.2%,2.3%,1%,1.6%,and 0.4%) with firing temperature of (750,800,850,900,950, and 1000) °C respectively. The most important factors affecting bulk density are water content, firing temperature, soaking time, components of raw materials, and the type and proportions of materials additives [9].

C. Compressive Strength

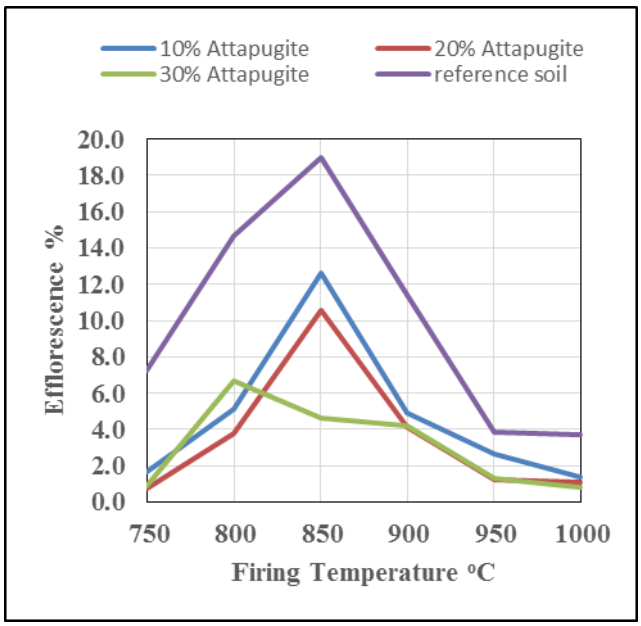
Fig. 5 below represents the effect of firing temperature and replacement % of Attapulgite on the compressive strength of clay bricks, notice that there is a significant increasing in compressive strength with using Attapulgite clays reached to (9.1%, 4%, 13.6%, 10%, 6.5%, and 3.3%) with firing temperature of (750,800,850,900,950, and 1000) oC respectively. Compressive strength is affected by several factors, some of these leads to increase the compressive strength & others leads to decrease the compressive strength , these factors including: firing temperature, shape, dimension, and components of raw materials.



Figure(5) Effect of firing temperature and replacement % of Attapulgite on the Compressive strength of clay bricks

D. Efflorescence

Fig. 6 below represents the effect of firing temperature and replacement % of Attapulgite on the efflorescence of clay bricks samples, it is clearly that there is a significant decrease in efflorescence reached to (89%,74%,44%,64%,67%, and71%) with firing temperature of (750,800,850,900,950, and 1000) oC respectively .



Figure(6) Effect of firing temperature and replacement % of Attapulgite on the Efflorescence

E. Water Absorption

Fig. 7 below represents the effect of firing temperature and replacement % of Attapulgite on the water absorption of clay bricks samples, there is a significant increasing in water absorption reached to (25.3%,18.7%,8.2%,2.5%,2.8%, and 5.6%) with firing temperature of (750,800,850,900,950, and 1000) °C respectively . Porous clay bricks as the main component of the masonry structure, their capacity of absorbing liquids and/or moisture, from both the immediate surrounding and the other building materials in association with them, can be referred as their capillarity [10].

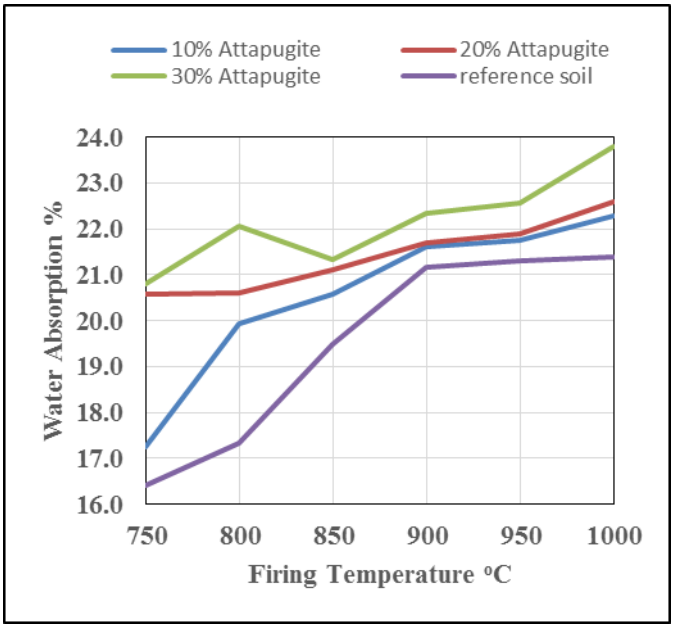


Figure (7) Effect of firing temperature and replacement % of Attapulgite on the water absorption

F. Modulus of Rupture

Fig. 8 below represents the effect of firing temperature and replacement % of Attapulgite on the modulus of rupture of clay bricks samples, it is clearly that there is a significant increasing in modulus of rupture reached to (3.4%,19.2%,136%,11.5%,41%, and 18.9%) with firing temperature of (750, 800, 850, 900, 950, and 1000)°C respectively.

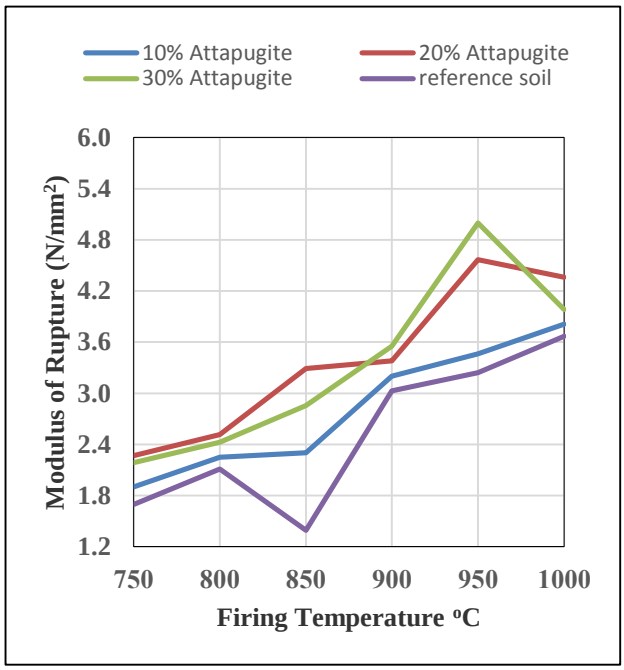


Figure (8) Effect of firing temperature and replacement % of Attapulgite on the modulus of rupture

V. Conclusions

Using Attapulgite clays as a partial substitute by weight of the clay used in the manufacturing local clay bricks leading to improve the properties of bricks, by reducing shrinkage burning and reduce density with significant improvement to compressive strength. It should be noted that there was a good improvement in modulus of rupture & efflorescence. Taking into consideration the negative effect of using the Attapulgite clays on water absorption.

IV. References

1- د.رائد متي وسوسن فؤاد ، **"صناعة الكتل الفخارية الخلوية من المواد المحلية "**، مركز بحوث البناء، مجلس البحث العلمي ، بغداد. 1984 .

2- د.سامي عبد الرسول ،**" خمسون قرنا لاستعمال الطابوق في بلاد مابين النهرين "** ، مركز بحوث البناء ، بغداد،1984.

3- BEECH, D. G., **"Testing Methods for Bricks and Tile Manufacture",** Brit. Ceram. Res. Assoc., Publication No.841, 1974.

4- C. C. Swan, **"Grain size distributions and soil particle characteristics",** University of Iowa, 53:030 Class notes, USA, p.p11.

5- Clean Building Technology for Nepal, **"Green brick making manual",** Nepal, 2008, p.p.26.

6- المواصفة القياسية العراقية (IQS) رقم 25 ، **" الطابوق الطيني"** ، الجهاز المركزي والسيطرة النوعية ،بغداد، 1988.

7- المواصفة القياسية العراقية (IQS) رقم 24 ، **"طرق اخذ النماذج وفحوص طابوق البناء"**، الجهاز المركزي والسيطرة النوعية ،بغداد ، 1988.

8- The Brick Industry Association, **"Technical Notes on Brick Construction"**, Reston, Virginia, USA, 2006. p.p 6.

9- S. Karaman, S. Ersahin, and H. Gunal, **"Firing Temperature and Firing Time Influence on Mechanical and Physical Properties of Clay Bricks",** J. of Scientific & Industrial Research Vol.65, 2006, p.p157.

10- Mariarosa R., Michele D., Davide G., Guia G., and Francesca M., **"Predicting the Initial Rate of Water Absorption in Clay Bricks",** Institute of Science and Technology for Ceramics, via Granarolo, 64, 48018 Faenza, Published on Construction and Building Materials, 23. CNR-ISTEC, (2009), Italy. p.p. 2623-2630.

Improving Work Flow on Construction Projects in GCC Countries

Mohamed Abdelaal, Hassan Emam and Peter Farrell

Abstract— **Productivity problems often lead to time and cost over-runs on construction projects. These problems manifest themselves in a lack of coordination, unidentified scope leading to poor planning, lack of monitoring and control, unrealistic schedules, poor implementation, and incorrect resource allocation. There it is a concern about how to best plan and control projects, and what is the optimised strategy to be followed depending on type and size of projects and the culture surrounding them. Large scale construction projects are particularly vulnerable to cost and time overruns, as a result of productivity problems. Therefore methodologies have been developed to reduce the risk of overruns and improve project outcomes; a number of these methods are based upon lean production. Integration of Last Planner Systems (LPSs) and Location-Based Management Systems (LBMSs) aims to achieve lean goals through a social process, by trying to make planning a collaborative effort and by improving the reliability and commitments of team members. The method for the study was a quantitative questionnaire supported by qualitative interviews with practising professionals in GCC countries. It was found that implementation of LPS within projects created predictable and reliable project plans in full detail, identified and removed constraints before they became obstacles, improved logistics at sites and supported the completion of projects within agreed durations and cost.**

Index Terms— **Construction, Time, Cost, LPS, LBMS**

I. Introduction

LARGE scale construction projects are particularly vulnerable to cost and time overruns, as a result of productivity problems. Therefore methodologies have been developed to reduce the risk of overruns and improve project outcomes; a number of these methods are based upon lean production. The application of lean construction is based upon treating the construction site as a temporary production line. The last planner System (LPS) and Location-Based

Management System (LBMS) both aim to achieve the lean goals of decreasing waste, increasing productivity and decreasing variability.

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Mohamed Abdelaal is a PhD candidate at the University of Bolton, UK(maa4mpo@bolton.ac.uk).
Hassan Emam is a PhD candidate at the University of Bolton, UK(he2mpo@bolton.ac.uk).
Peter Farrell is a Reader in Construction and Commercial Management at the University of Bolton, UK(P.Farrell@bolton.ac.uk).

II. The Use of LBSM and LPS in Construction Projects

The location of projects are hierarchical and are defined by a Location Breakdown Structure (LBS) or Location-based Management System (LBMS). Bills of Quantities for projects are spread over project locations and sub locations, and each location has defined works which must be completed before moving teams to other locations; thus data on quantities per location is the base from which to start LBMS, and the use of that is to move from master schedules of projects to more detailed and focused locations or sub-locations and work packages that combine several areas and trades of projects. To be completed, coordination is required with the various parties and subcontractors under the supervision and control of main contractors. LBMS integrates the Critical Path Method (CPM) into flow line scheduling. Logic can be automatically generated by considering tasks composed of multiple locations. The goal of location based planning is to optimise labour flow (Kenley and Seppanen[1]), so that work does not wait and workers are busy all the time. But problems with it are that it requires more data than CPM scheduling, including quantities by location, productivity rates (which vary from one project to another) and the availability of skilled labour under supervision of accountable subcontractors. Also the location breakdown structure needs to be decided before the start of the planning. All these data requirements can limit the ability of planners working with CPM scheduling. In LBMS, there are four stages of information: baseline, current status, progress and forecast. It has been noted that this kind of planning facilitates only limited coordination between contractors and subcontractors. Ideally it should include clients or client representatives, to ensure all project parties have the same level of knowledge and information, thus enhancing client orientation towards the phased hand-over of large projects, instead of having one date for substantial completion. It can be of more benefit for clients to receive hand-over location by location, thus other interfaces or final stage contracts can be occupied by location, and then their works completed much earlier. That will give clients a clear picture about the final product in early stage of project closure. Figure 1 illustrates how three tasks are programmed using the Line of Balance (LoB) planning method.

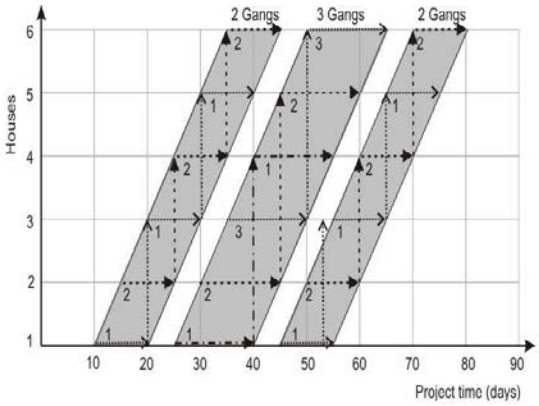

Figure 1. Balanced production of three tasks in line of balance (Kenley and Seppanen1 after NBA [2]).

The last planner system (LPSs) aim to increase productivity by only allowing the planning of assignments which are ready to enter weekly work plans. It thus concentrates on ensuring work is ready by having materials and resources available, site works accessible, all approvals attended, assignments well defined and produced by a person or a group of people called the 'last planner'. In LPSs, master schedules are limited to phased milestones, special milestones, and long lead time items. These phased schedules are planned by teams who will do the work by using pull techniques - working backward from target completion dates. Once the phase schedule is ready the look-ahead will be extracted from it, to assign the next week tasks at the end of the week. Percentage of plan complete (PPC) is calculated and any failed assignments are recovered and preventive action applied in order to avoid its re-occurrence in the future. The disadvantages of LPS are that it requires many workshops to be held with assigned work teams, and lots of reporting through the working week that might affect concentration on the task itself. Also the method assumes forecasts with same productivity rates, without considering any design changes or new instructions that might change the methodology of works that are to be completed. Figure 2 illustrates the stages in the LPS.

LBMS is mainly a technical, data-driven system. Certain conditions and requirements need to be collected as inputs to the system in order to ensure continuity of work flow; such as site conditions, weather conditions, type and size of project, procurement, culture, production rates, and approvals. All provided information aids decision-making and supports optimised schedules, resources and time and as result good cost control. While LPS is primarily a practical control system focusing on improving the work execution, it also includes a planning component: phase schedules. LPS concentrates more on the social process of continuous improvement, collaborative planning and improving the reliability of commitments

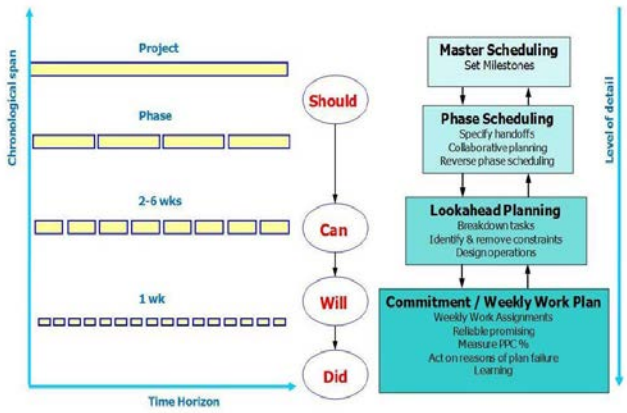

Figure 2. Planning stages/levels in the Last Planner System (adapted from Ballard, 2000 [3].)

III. Literature Review and Issue Identification

Project planning has been defined as "the process of choosing the one method and order of work to be adopted for a project from all the various ways and sequences in which it could be done" (Antill and Woodhead [4]). Planning processes are "those processes performed to establish the total scope of the effort, define and redefine the objectives, and develop the course of action required to attain those objectives" (Project Management Institute, PMI [5]). The application of lean construction is based upon treating construction sites as temporary production lines and aims to decrease waste, increase productivity and decrease variability" (Miller et al[6]).

The Last Planner System (LPS) and Location-Based Management System (LBMS) both aim to achieve the lean goals of decreasing waste, increasing productivity and decreasing variability (Seppanen et al [7]). LPS aims to achieve these goals primarily through a social process, by trying to make planning a collaborative effort and by improving the reliability of commitments of team members (Ballard [3]). LBMS is primarily a technical system, which transforms quantities in locations and productivity information to reliable durations, makes buffers explicit, and forecasts future performance based on historical trends and alarms of future production problems (Kenley and Seppanen [1]). Recent research related to reliability of location-based plans found location-based reliability metrics and PPC (Last Planner's weekly plan reliability metric) to be heavily correlated (Seppanen, 78-82[8]). Cascading delay chains were found which resulted from main contractors (MCs) lack of understanding of specialist mechanical, electrical and plumbing (MEP) subcontractor work. A lack of common understanding between the MCs and subcontractors about required resources, affected the start dates of subcontractors' work leading to location congestions and slowdowns; there is often a failure to openly discuss production problems in meetings (Seppanen, 155-1568). An enlightening feature of LBMS is that was it found to be capable of initially

forecasting 29% of the production problems (Seppanen, 89-90 [8]), and with further work able to further forecast at a level of 90%; 57% of them over two weeks before the problem appeared (Seppanen, 125-126 [8]). However, the information was not used in a systematic fashion and most of the early warnings which later resulted in production problems were not discussed (Seppanen, 154 [8]). LBMS implementation without adequate social process will lead to sub-optimal results even when the information required to successfully manage projects is available to the decision-makers (Seppanen [8]). Both LPS and LBMS research have reported case studies where durations were compressed, schedule conformance improved, and productivity increased (Kenley and Seppanen, 519-521 [1]; Ballard and Howell [9]).

IV. METHODOLOGY

This research investigates best practice possibilities to combine LPS and LBSM in construction. The research proposes a process to integrate LPS and LBMS in pre-bid master scheduling (pull) phase scheduling, look-ahead scheduling, and weekly planning, and to observe the result of implementing the process and measuring the impact on project performance and the feedback from different project parties. The main goal of the research is to provide essential information about LPS and LBSM and their use in construction projects in Gulf Cooperation Council (GCC) countries.

The data collection instruments used in the research are a questionnaire survey, followed by interviews. A questionnaire was preferred as the best effective and suitable data-collection technique for the study. The questionnaire was a self-administered tool with web-designed questions. A questionnaire in a web-survey format comparatively requires less duration and saves cost for researchers while permitting participants to respond to the questionnaire at their convenience. However, for this approach the response rate is usually lower as compared to face-to-face interviews. Data was collected for the literature review from books, journals and articles. A survey was given to employees from different professions involved with construction in GCC countries

A LPS AND LBMS QUESTIONNAIRE

Questionnaires surveys were designed to provide a feedback on the implementation process of LPS and LBMS in construction projects in GCC countries. The questionnaire was divided into four sections. The first two sections were to establish the profile of the participants and that of their organisations. Subsequently, the next section reviewed the benefits recorded in the implementation of LPS and LBMS while the last section dwelt with the critical success factors of implementation. The questions focused on the barriers, benefits and critical success factors of the implementation of LPS and LBMS. They also focused on the performance of projects in relation to current construction practices.

B SURVEY AND QUESTIONNAIRE REVISION

A face-to-face discussion was conducted with ten construction professionals. This procedure improved the validity of the survey. Questionnaires were sent by e-mail to contractors, architects, owners, project managers, and project engineers of various construction organisations. 16 replies were received (2 from client organisations, 5 from consultants and 9 from contractors). Mindful of a relatively low response rate, it was decided to supplement the data collection process with qualitative interviews with construction professionals.

C QUESTIONNAIRE DISTRIBUTION

The target groups in this study were professionals from the construction industry. A list of 547 building-construction organizations was obtained from the Construction Week online, the Gulf Cooperation Council's (GCC) magazine. The sample size can be calculated with the following equation for a 95% confidence level (AlShahri et al [10]; Moore et al [11]):

$$n = \frac{n'}{\left(1+\frac{n'}{N}\right)} \qquad (1)$$

Where, n= total number of population, N = Sample size from a finite population, n' = sample size from an infinite population= S2/V, S2 = the variance of the population elements and, V = a standard error of the sampling population. (Usually, S= 0.5, and V = 0.06).n'= S2 / V2 = (0.5)2 + (0.06)2 = 69.44, for N=547, n = 69.44 / [1 + (69.44 / 547)] = 62. To obtain 95% of confidence level, it was calculated to send the questionnaire to minimum of 62 participants.

D DATA COLLECTED FROM THE WEB SURVEY

To successfully achieve the objective of the study, one of the most important phases is collection of accurate data. Data collection is a procedure of collecting data records for a certain sample or population of observations (Bohrnstedt and Knoke [12]).

E ANALYTICAL METHOD USED

In order to facilitate the study, after the literature review and the focus interviews, a plan was formulated for collecting field information and creating an evaluation process and numerical values. It was necessary to provide straightforward communication to participants to ensure a clear understanding of all the applicable definitions, procedures, and guidelines that were used in collecting data. Because the data-collection process included individuals, two different ways were used to analyse the survey results.

F RANKING

The empirical data gathered was both descriptive qualitative and quantitative in nature and they were used to establish the link between the literature reviews. Questions were asked using questionnaires and interviews. The categorisation and ranking enables priorities to be allocated (Bryman [13]). In

carrying out this research, the Relative Importance Indices (RII) was used. The same method was adopted in this study. The five-point Likert scale ranged from 1 (very low importance) to 5 (very high importance) was adopted and transformed to RII for each factor as follows:

$RII = \Sigma W/A*N$, where W is the weighting given to each factor by the participants (ranging from 1 to 5), A is the highest weight (i.e. 5 in this case), and N is the total number of participants.

For the interpretation of the *RII* values, *RII* is ranked from the highest to the lowest.

V. Results

A Last Planner System – LPS.

TABLE 1 OVERVIEW OF RESULTS OF LPS IMPLEMENTATION IN CONSTRUCTION PROJECTS IN GCC COUNTRIES

Results	0	1	2	3	4	RII	Rank
LPS was very effective within this project	0	2	2	8	4	0.69	2
The results achieved from implementation on previous projects are satisfactory	0	2	4	10	0	0.62	3
The Weekly Work Plans and PPC were very useful	0	2	0	9	5	0.75	1
It was difficult carrying out the implementation	1	2	7	3	3	0.62	3

The findings from the survey suggest that implementation of LPS in construction projects in GCC, is useful.

TABLE 2 BARRIERS DURING LPS IMPLEMENTATION IN CONSTRUCTION PROJECTS IN GCC COUNTRIES

Barriers	0	1	2	3	4	RII	Rank
Inadequate supervision	0	2	5	8	1	0.62	5
Fluctuations and variations	0	0	8	6	2	0.66	4
Subcontractor's lack of involvement	0	2	1	8	5	0.73	1
Resistance to change	0	1	5	7	3	0.69	2
Negative cultural issues	1	2	7	3	3	0.62	5
Lengthy approval procedure by client	2	4	2	0	4	0.67	3

The findings from the survey suggest that subcontractor's lack of involvement is the major barrier being considered during the LPS implementation in construction projects in GCC.

CRITICAL SUCCESS FACTORS TO THE IMPLEMENTATION OF LPS IN CONSTRUCTION PROJECTS IN GCC.

Question 4 asked about critical success factors to the implementation of LPS in construction projects in GCC countries. The findings from the survey suggest that required project team training, management support and involvement of all stakeholders are the major success factors being considered to LPS implementation.

TABLE 3 BENEFITS OF THE IMPLEMENTATION IN CONSTRUCTION PROJECTS IN GCC

Benefits	0	1	2	3	4	Mean	RII	Rank
Identifying and addressing potential problems before they become obstacles in the project	0	0	0	12	4	3.21	0.80	6
Reducing the incidence of bad news and to get what bad news there is early	0	0	3	8	5	3.07	0.77	9
Developing supervisory skills and reducing the load on management	0	0	1	7	8	3.43	0.86	2
Creating a more predictable and reliable production programme	0	0	1	7	8	3.43	0.86	2
Delivering projects more safely, faster and at reduced cost	0	0	1	5	1	3.57	0.89	1
Improving construction logistics on projects	0	0	0	9	7	3.43	0.86	2
Improving predictions of labour required	0	0	1	10	5	3.21	0.80	6
Reduces the risk of catastrophic loss	0	0	1	10	5	3.21	0.80	6
Completes projects on schedule	0	0	1	8	7	3.36	0.84	5

The findings from the survey suggest that delivering projects more safely, faster and at reduced cost is the major benefit considered to the LPS implementation in construction projects in GCC countries.

B Location Based Management System – LBMS.

Question 1 asked about the participant's company policy for developing their construction schedule. The findings from the survey suggest that schedules are developed for all projects.

Question 2 asked about the basis for the development of construction schedules, and the participants expressed their view that schedules were developed based on the priority factors of the project requirements. The findings from the survey suggest that project duration is still the major priority factor being considered during the development of construction schedules.

Question 3 asked about the responsibility for developing a construction plan and schedule. The survey findings about organisation charts revealed that, generally, only one planner is involved in the planning and scheduling process in construction projects. Planning and scheduling can, however, be made more effective by involving experienced team members, project planners and construction managers,

considering the value and complexity of the construction projects.

Question 4 asked about major delay factors in linear construction projects. The participants were asked to rank the major problems facing companies in linear construction projects. From the analysis of the findings, it was concluded that planners need to focus more carefully on the impacts of possible change orders and the relocation of utilities, and to prepare effective resource planning.

Question 5 asked about the participant's company existing methods for developing their construction schedules. The survey results showed that the majority of companies still highlight CPM as the key tool in the development of construction plans and schedules.

Question 6 asked about the participant's company existing practices for developing their construction schedules. The survey results showed that the majority of companies still highlight previous experience as the key influencing factor in the development of construction plans and schedules; however, a few companies use intuitive methods and 'rules of thumb' for project planning and scheduling.

Question 7 asked about the participant's company currently used software for developing their construction schedule. The findings from the survey suggested that different construction companies use different types of planning and scheduling software based on available expertise and the contract requirements. The survey results revealed that Sure Track is most used software in planning and scheduling processes.

Question 8 asked about how often participant companies produce project schedule reports. The conclusion to the findings from the survey was that the majority of construction companies of category-A still use monthly construction plans and schedules.

Question 9 asked about the participant's company existing practices techniques used for planning and scheduling of infrastructure and road works projects. The findings from the survey revealed the majority of construction companies used decision on site by experience. The survey results showed that there is no existing practice in the use of software for visual simulation.

Question 10 asked about the participant's company policy for developing their construction schedule. The respondents were asked to rank the critical factors considered in infrastructure and road works planning. According to the survey results soil characteristics is the most critical factor that affects construction planning and scheduling.

Question 11 asked about anticipated application of visual tools at different stages of the project (tender, planning, construction and/or operational stage). The participants agreed that visualisation tools are applicable and beneficial at different stages of construction projects.

Question 12 asked about participants about the importance of visualisation tools in construction project operations. They were asked to rank the application areas/influencing factors such as improvement in communication of scheduling information; pre-information of activity sequences; and crew/equipment conflicts that assist in updating in a schedule production and identification of idle time. Also to determine the site application areas for the existing visualisation tools. The participants ranked from the most important to least important factors which have high impact on visualisation systems in earthwork planning. The most important factor was found to be visualisation tools.

VI. CONCLUSION AND SUMMARY

This research provides study and knowledge of implementation of LPS in construction projects in GCC countries. The study sought the views of clients, consultants, and contractors on the outcome of construction projects, especially public projects that influence national economies. Lean construction is a change initiative which seeks to improve construction project performance through the application of lean tools, especially the Last Planner System. Thus Design Science Research, a research methodology advocated by Lean Construction practitioners, is adopted to implement a solution that can bring about change to a phenomenon using an Action Research approach. Action Research is inherently a change oriented approach in which a process is studied and change is introduced and observed. The findings from the survey are that implementation of LPS revealed within projects created predictable and reliable project plans in full detail, identified and removed constraints before they became obstacles, improved logistics at sites and supported the completion of projects within agree durations and cost. In previous studies, comparing successful projects handled by multi-national firms in GCC countries, with typical LPS projects identified in literature reviews, there was a lot similarities in terms of project outcomes; although differences existed in the way projects were managed. Nevertheless, during LPS implementation, the following hurdles were confirmed as anticipated from the literature reviews: cultural issues, lengthy approvals, resistance to change, supervision and quality control, sub-contractors involvement, fluctuations and variations. Accordingly, a framework is developed to overcome the hurdles identified. This framework included the need to; identify purpose, identify stakeholder's impact, obtain sponsorship, build cross functional teams, create measurement indices, create a right working climate and provide for training on lean techniques and LPS. This framework was further validated by industry practitioners within the field of the study and positive feedback was obtained from focus group discussions. Considering the survey results above, it is concluded that visualisation tools for earthwork operations are beneficial for resource planning and aid in producing effective construction schedules.

VII. RECOMMENDATIONS

Construction projects are high risk and often lead to a disputes and claims as work progresses, which then subsequently

further affects progress. The environment within construction organisations should be suitable to successfully complete projects. In construction, it is necessary to identify potential problems in advance, in order to avoid and overcome possible impacts on cost or project time. Stemming from the research, the following recommendations are suggested in this research.

The proposed framework is not a pick and choose tool box or a rigid step-by-step framework, rather it is a guideline as to what should be in place to promote the successful and effective implementation of LPS. Implementing LPS is usually a lengthy and sometimes a cumbersome process, although it promises to improve planning, control and coordination. Hence it requires a lot of commitment and patience from practitioners seeking to implement it for the first time, knowing that planning and control are dynamic and iterative processes. Location-based schedules and visually analysing scheduling information of leaner scheduling projects, like earthworks improves its planning, control and coordination. Construction professionals agreed that location-based schedules and visual analysing is valuable in resource planning at the required locations, and when necessary, throughout earthwork operations, particularly in linear projects. Location-based schedules and visual analysing is also useful in communicating scheduling information amongst project stakeholders. Location-based schedules and visual analysing works as a logical decision-making tool in producing resource schedules for linear projects in a lean and effective manner, improves site productivity, scheduling overview, workflow establishment, and control of site progress, and reduces the production time and costs of construction projects

VIII. FUTURE RESEARCH

Further research should focus on the holistic barriers of implementing of LBSM and LPS and development of a universal implementation framework that can fit into any construction environment. In the same vein, further work should be undertaken on applying the same research in other developing countries. Similarly, additional research should be made in the adoption of other lean construction tools and techniques within GCC countries.

REFERENCES

[1] Kenley, R. and Seppänen, O. Location-based management for construction: *Planning, scheduling and control*. Spon Press, London, 2009.

[2] NBA 1968. *Programming House Building by Line of Balance*, The National Building Agency, London: 24, 1968.

[3] Ballard, G. *The Last Planner System of production control*. Ph.D. Thesis., Faculty of Engineering, University of Birmingham, U.K., 2000.

[4] Anthill, J., and Woodhead, R. *Critical Path Methods in Construction Practice*," Fourth Edition, John Wiley & Sons, New York, NY. , 1990.

[5] PMI. Project Management Institute, Communications area of knowledge, *A Guide to the Project Management Body of Knowledge* (PMBOK® Guide)". Fourth Edition, 2008.

[6] Miller, C., Packham, G. and Thomas, B. Harmonization between Main Contractors and Subcontractors: A Prerequisite for Lean Construction?' *Journal of Construction Research*, 2002, 3, No. 1, pp. 67^82.

[7] Seppanen, O., Ballard, G. and Pesonen, S. The Combination of Last Planner System and Location-Based Management System. *Lean Construction Journal*, 2010, 6, No. 1, pp. 43^54.

[8] Seppanen, 0. *Empirical Research on the Success of Production Control in Building Construction Projects*. Ph.D. Diss. Helsinki University of Technology, Finland, 187 pp. (available at http://lib.tkk.fi/Diss/), 2009.

[9] Ballard, G. and Howell, G. An Update on Last Planner. *Proceedings of the 11th Annual Conference of the International Group for Lean Construction*, July 22-24, Blacksburg, Virginia, 2003.

[10] Al-Shahri, M., Assaf S., A., Atiyah S., and AbdulAziz.A. The management of construction company overhead costs." *International Journal of Project Management*, 2001, 19, pp. 295^ 303.

[11] Moore, K.M., Mantua, N.J., Kellogg, J.P. and Newton, J.A. Local and large-scale climate forcing of Puget Sound oceanographic properties on seasonal to interdecadal timescales. *The American Society of Limnology and Oceanography*, Inc, 2008.

[12] Bohrnstedt, G, and Knoke, D. *Statistics for Social Data Analysis* (3rd Edition). F.E. Peacock Publishers, Inc., Itaska IL., 1994.

[13] Bryman, A. *Social Research Methods*. 3rd ed. Oxford: Oxford University Press, 2012.

Mohamed Abdelaal is a planning and scheduling professional with a civil engineering background, certified project management profession and currently he is a PhD candidate at the University of Bolton. He has various experience in the construction industry on a variety of projects such as residential, commercial, airports, bridges, road works, marine, treatment plants and infrastructure.

Hassan Emam is a planning and scheduling professional with a civil engineering background and Master of Business Administration in Construction and currently he is a PhD candidate at the University of Bolton. He has diversified experience in the construction industry on a variety of projects such as residential, commercial, airports, bridges, marine, and railways.

Peter Farrell is a Reader in Construction and Commercial Management at the University of Bolton. He has industry experience in planning and quantity surveying, whilst most of his post-qualification work was as a construction site manager.

Internally Cured of Steel Fiber-Reinforced Self-Consolidating Concrete

Ikbal N. Gorgis, Maan S. Hassan and Aymn H. Ali

Abstract- Application of internal curing for concrete using locally available Porcelinite as a lightweight aggregate has received increasing attentions in recent years. The main aim of this study is to investigate the combined effect of steel fiber and internal curing on the strength characteristics of Self-Consolidating Concrete (SCC). The experimental work includes two stages. First stage involved conducting several trial mixes and then choosing the one that conform to international standards in terms of fresh properties. The second stage was carried out to investigate the ability of internal curing method by replacing 15% of sand with saturated fine lightweight aggregate (LWA) as internal curing material to study the change in the fresh and hardened properties of SCC. Four concrete mixes were used with different volume fractions of hooked steel fibers were incorporated 0%, 0.5%, 1%, and 1.5%. Results showed that adding steel fibers adversely affect SCC workability and thus more dosage of SP should be added to stay within the standard limits. The presence of steel fibers provides slight increase in compressive strength while significant enhancement in tensile properties was observed. Furthermore, replacement of fine aggregate by pre-wetted fine LWA causes an increment in hydration which leads to higher compressive and tensile strengths. Results of rate of absorption (sorptivity) revealed that the implementation of steel fibers has beneficial effects, while the presence of fine LWA has adverse negative effects.

Keywords: flexural, lightweight aggregate, Self-consolidating concrete, steel fiber, toughness**.**

I. INTRODUCTION

The development of self-consolidating concrete (SCC) makes an important milestone in improving the product quality and efficiency of the building industry. SCC homogeneously spreads due to its own weight, without any additional compaction energy and does not entrap air. Self-compacting concrete (SCC) was defined by Okamura [1] as concrete that is able to flow in the interior of the form work, fling it in a natural manner and passing through the reinforcing bars and other obstacles, flowing and consolidating under the action of its own weight. One of the main labors needed for placing and finishing the concrete. All these advantages in using SCC is the minimization of skilled benefits decrease the costs and reduce the time of the building process over constructions made from traditionally vibrated concrete [2].

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Ikbal N. Gorgis, is with the UOT team (e-mail: Iqbalkorkess@yahoo.com).
Maan S. Hassan, is with UOT team (e-mail: author@lamar.colostate.edu).
Aymn H. Ali, UOT student (e-mail: author@nrim.go.jp).

Segregation resistance of SCC is enhanced by modifying the mix proportions, *e.g.* reducing the w/c ratio, increasing the fines content and/or incorporating super-plasticizer (SP) and a viscosity modifying admixture [3,4 & 5].

The term “curing” is used to describe the action taken to maintain moisture and temperature conditions in a freshly placed cementitious mixture to allow hydraulic-cement hydration and, if applicable pozzolanic reactions to occur [6]. The objectives of curing are to prevent the loss of moisture from concrete and, when needed, supply additional moisture and maintain a favorable concrete temperature for a sufficient period of time.

Many types of aggregate are used to produce concrete. They are commonly classified into three groups according to its weight, heavy weight aggregate (HWA), normal weight aggregate (NWA) and lightweight aggregate (LWA) [7]. The industrial use of natural LWA was started in 1845 in Germany by Ferdinand Nebel from Koblenz who produced masonry blocks from pumice, with burnt lime as binder [8]. The LWA have low particle density because of the cellular pore structure within the particles normally developed by heating certain raw materials to incipient fusion. Higher porosity of LWA may affect transport of water and ions in concrete. LWA contains many small microscopic pores; this enables the aggregate to absorb between 15%-25% of its weight with water. This means LWA can supply the required water/moisture required for internal curing [9].

Internal curing refers to the process by which the hydration of cement occurs because of the availability of additional internal water that is not part of the mixing water [10]. The water supplied from LWA called (Internal Curing Water) and the porous material (LWA) called (Internal Curing Material). The benefits of using internal curing in concrete are to reduce "cracking, autogenous shrinkage, porosity and permeability". Beside it will helps to complete the hydration of cement which is not hydrated by the mixing water. This will leads to increase concrete density, early and late age compressive and flexural strengths as well as enhancement in the durability.

The distribution of water reservoirs in the mixture is of primary importance in the internal curing process. The distance of the saturated LWA from the point in the cement paste, where the relative humidity (RH) drop takes place, determines the efficiency of the internal curing. If the water reservoirs are well distributed within the matrix, shorter distances have to be covered and the efficiency of the internal curing process is increased. These considerations lead to the choice of small size of LWA rather than large size of LWA for internal curing purpose [11].

The introduction of steel fibers in concrete is another issue of interest on the concrete technology. Steel fibers proved to have the potential to increase the post-cracking energy absorption capacity of cement based materials, enhancing the ductile character of concrete structures behavior, mainly of those with high redundant supports [12].

Plain concrete is a brittle material with low tensile strength and poor fracture toughness; it imposes numerous design constraints and often leads to long term durability problems. Therefore, discrete fibers with adequate mechanical properties can be added to concrete to improve toughness; increase resistance to impact; reduce spalling of the reinforcement cover; and improve abrasion resistance and flexural and shear strength [13].

The objective of this study is to evaluate the effects of hooked end steel fiber on the fresh and hardened properties of SCC with partial replacement of fine normal weight aggregates with lightweight aggregate. Trail mixes were done to study the fresh properties of SCC using locally source raw materials to conform international standards [14&15] as presented in Table (I).

Table I

Test methods for workability properties of SCC [14&15]

No.	Test	Range
1	Slump flow	650-800 mm
2	T50 cm slump flow	2-5 sec
3	J-Ring flow	580–780 mm
4	V-funnel	6- 12 sec
5	Increase in V-funnel at $T_{5\,min}$	+3 sec, max
6	L-box $(H_2/H_1)^*$	0.8- 1.0

*H_2/H_1 the height of concrete at the end of horizontal section to that remaining in the vertical section of L-box.

II. EXPERIMENTAL WORK

A. Materials and Concrete Composition

Locally source raw materials were provided to produce the SCC concrete mixes. Fine aggregate (Zone 2) and coarse aggregate, with max size of 10mm conform to Iraqi specification No.45/1984[16] were used. Limestone powder of 100 kg/m^3 with a surface area of 3150 cm^2/gm (Blain method) and silica fume of 50 kg/m^3 conform to ASTM 1240-05[17] were implemented to increase the volume of mortar and then enhance workability. Cement content was 400 kg/m^3 which satisfy the physical and chemical requirement of Iraqi specification No. 5-1984[18].

Porcelinite stones were crushed into smaller size particles washed and cleaned with water in order to remove dust resulting from crushing process and afterward dried by spread in air. The crushed particles are sieved into different size fractions, and then every size of Porcelinite aggregate is partially replaced by volume with the same size of sand with a 15% percentage to have the same grading as original aggregate which satisfies the grading requirements of the Ref. [16]. Table (II) shows the chemical and physical properties of it. Before using the Porcelinite aggregate in the mix, they were soaked in water for (24) hours to bring the aggregate particles to saturated condition.

Hooked ends steel fibers which are commercially known as Sika Fiber were also used throughout the experimental program. Table (III) indicates the properties of steel fiber used.

Table II

Oxide composition	Chemical Compositions %	Physical Properties	
SiO2	62.02	Shape	Crushed
CaO	11.55	Apparent Specific Gravity	1.35
MgO	7.2		
Al2O3	2.7		
Fe2O3	0.87	Bulk Density (Kg/m^3)	855
TiO2	0.18		
SO3	0.3	Absorption	30%
L.O.I	13.86		

Chemical and physical properties of Porcelinite stones

Table III

Properties of the used steel fibers*.

Description	Hooked end
Length	30 mm
Diameter	0.5 mm
Aspect ratio (*L/D*)	60
Relative Density	7800 kg/m^3
Ultimate Tensile	1180 MPa

*According to the manufacturer

A chemical admixture based on modified polycarboxylic ether (Glenium 51) was used as a high range water reducing agent plus Stabilizing agent. Glenium 51 is free from chlorides and complies with ASTM C 494[19] Types F High range water reducing / Super plasticizer Concrete Admixture Standards. The water used was potable water from the water-supply network system (tap water); so, it was probably free from suspended solids and organic materials.

A. Mix Design

For the production of SCC, the mix design should be performed so that the predefined properties of the fresh concrete are reached. The basic components of the mix composition of SCC are the same as those used in conventional concrete. However, to obtain the requested properties of fresh concrete in SCC, a higher proportion of fine materials and the incorporation of chemical admixtures are necessary [7]. The mix constituents shall be identified so that segregation and bleeding are prevented while workability enhanced.

A series of trial mixes were then carried out and final five concrete mixes were obtained in compliance with standard acceptable. The total powder content was 550kg/m^3, which consist of cement 400kg/m^3, silica fume 50kg/m^{3}and limestone 100kg/m^3, the W/C ratio of 0.4

(160 kg/m^3) and coarse aggregate content of 805 kg/m^3, the other materials are listed in Table (IV). Resultant compressive strengths at 28 days were above 50 MPa and workability values were above 650 mm and up to 790 mm, for all mixes.

B. Mixing of SCC

The concrete was mixed using a drum mixer of 50 L capacity. Mixing procedure follows the laboratory procedure outlined by Emborg [20], and modified by ASTM C 192M -07 [21] as follows: 1) Fine aggregate was added to the mixer with 1/3 of water and it was mixed for 1 min; 2) Following that, the powder mixture of (cement, limestone and silica fume) was added; 3) After that, the coarse aggregate wass added with the last 1/3mixing water and 1/3 of superplasticizer, and mixing for 1.5 minutes; 4) Then, the remaining 2/3 of the superplasticizer was added and mixed for 1.5 minutes, the mixture was then discharged and cast in molds. The total time of mixing was 5 minutes.

C. Casting and Curing of SCC

The steel molds (100×100×100 mm cubes) for compression tests and cylinder with (dia.=100, h=200 mm) for splitting test, and (100× 100× 400 mm prisms) for flexural test were well cleaned. The internal faces were thoroughly oiled to avoid adhesion with the concrete after hardening. SCC mixes do not require compacting, so the mixes were poured into the tight steel molds (cubes, cylinders and prisms) until these molds were fully filled without any compaction. The moulds were covered with polyethylene sheet for about 24 hours to prevent loss of moisture from the surface and to avoid plastic shrinkage cracking. Then the specimens were demoulded for curing. Specimens were kept after demoulding in plastic bags sealed and saturated until the age of test.

III. EXPERIMENTAL TESTS

The experimental test program concludes to stages the fresh and hardened test. The fresh test concludes the slump, J-Ring, L-box and V funnel. These entire tests were down according to specification [15 & 22] and the results were recorded.

The hardened tests were, (1) Compressive Strength; was determined according to B.S 1881: part 116, 1989 [23]. The average compressive strength of three cube specimens was recorded. This test was conducted at 7, 28 and 90 days of age. (2) Splitting Tensile Strength; was done according to ASTM C496 –07 [24] and the average splitting tensile strength of three specimens were recorded. This test was conducted at ages 28, and 90 days. (3) Modulus of Rupture; third point loading test according to ASTM C78-02 [25] was carried out on 100×100× 400mm simply supported prisms. The specimens were tested at 28, and 90 days, and the rate of loading was about 0.015 MPa/sec. The average strength of three specimens was recorded, and it was indicated that fracture occurs within the central third for all specimens. (4) Sorptivity test; sorptivity is the material property used to quantify the resistance to absorption. A lower value of the sorptivity means that water penetrated into the concrete more slowly, and is indicative of higher quality concrete. This test method is used to determine the rate of absorption (sorptivity) of water according to ASTM C 1585[26] for both the concrete surface and internal concrete by measuring the increase in the mass of a specimen resulting from absorption of water as a function of time. The test consists of exposing the bottom surface of a sample to water and measuring the increase in mass resulting from absorption. Before the test is conducted, samples are conditioned for 14 days. The specimens were prepared by cutting a cylinder to three discs with a length of 50 mm and diameter of 100 mm obtained from the molded cylinders by using an electrical saw. The samples were first placed in a 50 °C and 80% RH environment. After three days of conditioning, the samples were removed from the oven and placed in individually sealed containers [Seal the side and top surface of each specimen with a suitable sealing material], where the samples are retained for 15 days to allow internal moisture equilibrium before the test begins. The mass of the sealed specimen was measured and recorded as the initial mass for water. The support device was placed at the bottom of the pan and the pan is filled with tap water so that the water level is 1 to 3 mm above the top of the support device. The absorption test involves recording the incremental mass change measurements during the first six hours after the sample came in contact with water and take one measurement every day for the next eight days. The amount of absorbed water was normalized by the cross-sectional area of the specimen exposed to the fluid.

Table IV
Mix proportions for fiber reinforced SCC

No.	Description of mixes	Mixes	Sand kg/m^3	LWA Fine kg/m3	Dosage of SP L / m^3	Steel Fiber kg/m^3
1	Ref. SCC Mix with V_f = 0%	SCC 0%	805	0	15	0
2	SCC Mix with V_f = 0% and 15% LWA.	IC-SCC 0%	684	72	15	0
3	SCC Mix with V_f = 0.5% and 15% LWA	IC-SF-SCC 0.5%	684	72	16.5	42
4	SCC Mix with V_f =1.0% and 15% LWA	IC-SF-SCC 1%	684	72	17.5	84
5	SCC Mix withV_f =1.5% and 15% LWA	IC-SF-SCC 1.5%	684	72	19	126

IC: means "Internal Curing" by using fine light weight aggregate as partial replacement of fine aggregate.

IV. RESULTS AND DISCUSSIONS

A. Fresh Tests

A reduction in workability was observed associated with adding steel fibers, subsequently, more water and/or higher dosage of high range water reducing agent were added to keep the workability measurement tests (slump,

J-ring, L-box, and V-funnel test values), as much as possible within the acceptable standard limits as indicated in Table (IV). Accordingly, it was suggested that, from practical point of view, steel fibers have definite adverse effect on workability properties of fresh SCC. Consequently, higher chemical admixture dosages should be added. Table (V) shows the results of fresh test.

B. Hardness tests

In terms of hardness test, the results that listed in Table (VI) showed that all concrete specimens exhibited a continuous increase in strength with increasing curing age. This is referred to the increase of the bond strength between the concrete ingredients and the continuous hydration process which increased the dense hydrated calcium silicate in concrete with increase curing age.

Table V
Test results for all fresh mixes.

Mix Description	Slump flow		J-Ring D (mm)	L – Box H1/H2	V-Funnel (Sec)
	D (mm)	T_{500} (sec)			
Ref SCC 0%	775	3.5	755	0.96	6.2
IC-SCC 0%	768	4	737	0.92	6.7
IC-SF-SCC 0.5%	725	4.4	702	0.88	7.9
IC-SF-SCC 1%	689	4.8	669	0.85	9.8
IC-SF-SCC 1.5%	660	5	651	0.8	11.7

Table VI
Test results for all specimens of SCC mixes

Mixes Description	Compressive strength (MPa)			Splitting tensile strength (MPa)		Modulus of Rupture (MPa)	
	7 days	28 days	90 days	28 days	90 days	28 days	90 days
Ref SCC 0%	34.3	55	68.2	3.5	4.1	5.6	6.5
IC-SCC 0%	34.3	55.5	70.4	4	4.6	6.7	8
IC-SF-SCC 0.5%	35.8	58.3	74.3	5.1	5.8	9.5	10.1
IC-SF-SCC 1%	36.7	60.8	77.4	6.2	7.1	11.1	12
IC-SF-SCC 1.5%	38.2	62.1	80.2	7.3	8.2	12.7	13.6

Compressive Strength

The increase in compressive strength of the SCC containing steel fibers may be due to the increasing super-plasticizers dosage to retain and maintain their flowability and passing ability in the fresh state. In fact, the increase in compressive strength may be associated with uniform dispersion of fine fibers throughout self-consolidating concrete of very high flowability, leading to consistent internal integrity. Also this improvement in the compressive strength of the steel fiber reinforced SCC refer to the control of cracking and the mode of failure by means of post cracking ductility. The results show clearly a slight increase in compressive strength for concretes incorporated SF for both ages 28 and 90 days.

Internally cured concretes shows similar compressive strengths performance, in comparison with Ref SCC concrete (mix 1), the increases in the 28 days compressive strength of the mixes 2, 3, 4 and 5 were (0.9%, 5% , 10.5% and 12.9%) respectively and the increases after 90 days were (3.2%, 8.9 %, 13.5%,and 17.6%) respectively. It may be attributed to the continuous hydration of the mixture at later ages that promoted by the extra water stored in the LWA particles.

Also, this was due to improvement of the interfacial transition zone, enhanced hydration because of internal curing, and absence of shrinkage induced micro-cracking which can sustain hydration process and filling the pores with hydration products and increase the compressive strength of concrete.

Splitting tensile strength

The increases in the 28 days splitting tensile strength of the mixes 2, 3, 4 and 5 as a percent from reference mix 1 were (14.2%, 45.7%, 77.1% and 108.6%) respectively and the increases in the 90 days were (12.2 %, 41.5%, 73.2%, and 100%) respectively as shown in Table (VI). The improvement caused by steel fiber and internal curing is much obvious for conducted splitting tensile test than for compressive test results. The increment in strength at 90 days for instance, reached for compressive strength about (3%) for mix 2, mean while, the splitting strength for the same mix reached to (12.2%).

In comparison with mix 1 of SCC concrete, the increases in the 28 days modulus of rupture strength of the mixes 2, 3, 4 and 5 were (19.6%, 69.6%, 98.2% and 126.8%) respectively and the increases in the 90 days were (23%, 55.4 %, 84.6% and 109.2%) respectively as shown in Table (VI). The improvement caused by steel fiber is much obvious for modulus of rupture strength test than splitting tensile test and compressive test results. The mix 5 (IC-SF-SCC 1.5%) has the highest values of compressive, splitting and modulus of rupture strength value at 28 and 90 days among all mixes while the references mix (mix 1) SCC has the lowest strength value among all mixes at the same ages.

The results show clearly that the important increment all strength for concretes with SF and internal curing was at 28 and 90 days more than that of SCC with SF only, these results may be associated with the continuous hydration of the mixture at later ages promoted by the extra water stored in the LWA and could be the cause of that increase. Also, this was due to improvement of the interfacial transition zone, enhanced hydration because of internal curing, and absence of shrinkage-induced micro-cracking which can sustain hydration process and filling the pores with hydration products and increase the modulus of rupture of concrete. The improvement caused by internal curing is much obvious for conducted for modulus of rupture than splitting tensile and compressive test results.

Density

An average of three tests for each mix the densities of the mixes were determined and listed in Table (VII), the densities of the studied mixes are in the range of (2560 – 2634) kg/m^3. This range is greater than the range of the conventional concrete densities which is (2300 – 2400) kg/m^3 [27]. This increment refers to the low water/powder ratios and the employment of the superplasticizer, high powder content and the steel fibers in the mixes. The increases in the density of mixes 3, 4 and 5 as a percent of mix 2 at 28 days were (0.58%, 1.2%, and 2 %) respectively.

Table VII
Densities of the mixes

Mixes Description	Density (kg/m^3) at 28 days
Ref SCC 0%	2578
IC-SCC 0%	2560
IC-SF-SCC 0.5%	2575
IC-SF-SCC 1%	2591
IC-SF-SCC 1.5%	2611

V. FLEXURAL TOUGHNESS AND LOAD DEFLECTION CURVES

Prismatic with100*100*400mm in dimension were tested according to ASTM C1609 [28] for 28, and 90 days of curing for flexural strengths and toughness. The load deflection curves were presented in Figs. (1 & 2) and toughness values were listed in Tables (VIII). It was characterized by the post-peak portion of the area under the load-deflection curve obtained during a flexural test on 100*100*400 mm beams in a four-point loading arrangement for deflection of L/150.

Linear-elastic material behavior characterizes SCC, so the specimens fail explosively without any warning as in Fig. 1 for mixes without steel fiber in which the failure is clear to be brittle. For specimens incorporating steel fibers, the load-deflection behavior and consequently the ductility and fracture toughness can be improved. This can be traced back to the fact that, the fibers are able to transfer emerging loads by bridging the cracks. Here increasing the fiber content from 0 to 42 kg/m^3 to 84 kg/m^3 and finally 128 kg/m^3 make an impact after the appearance of cracks.

The property of flexural toughness relates to the ability of the concrete to absorb energy, after micro-crack formation, while the fibers hold the matrix together.

Flexural toughness can be defined as the area under the load-deflection curve in flexure up to deflection of (L/150) mm, which is the total energy absorbed prior to complete separation of the specimen. This was done to allow for removal of the instability part from the load-deflection curve to calculate the toughness values. The failure of SF-SCC suffers damage by gradual development of single or multiple cracks with increasing deflection, but retains some degree of structural integrity and post-crack resistance even with considerable deflection. On the other hand, the presence of steel fibers led to a continuation of the load-carrying capacity beyond the peak load implying an improved post-peak toughness as indicated in Table (VII). The increase in the toughness value for mixes with internal curing by fine LWA curing was clearly noticed after 28 days and 90 days curing. Mix 5 (IC-SF-SCC 1.5%) has the highest toughness value at 28 and 90 days among the mixes while and the reference Mix (mix 3) (SF-SCC 0.5%) has the lowest toughness value at the same age.

Table VIII
Toughness values of SCC mixes with LWA and steel fiber.

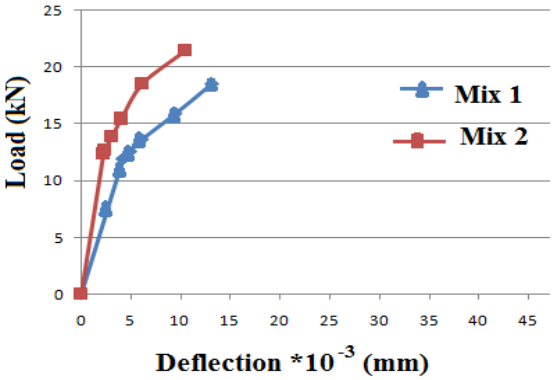
Mix No.	Mixes description	Toughness after 28 days	Toughness after 90 days
3	IC-SF-SCC 0.5%	38.9	48.4
4	IC - SF-SCC 1 %	47.7	56.7
5	IC-SF-SCC 1.5%	55.2	63.6

VI. SORPTIVITY TEST

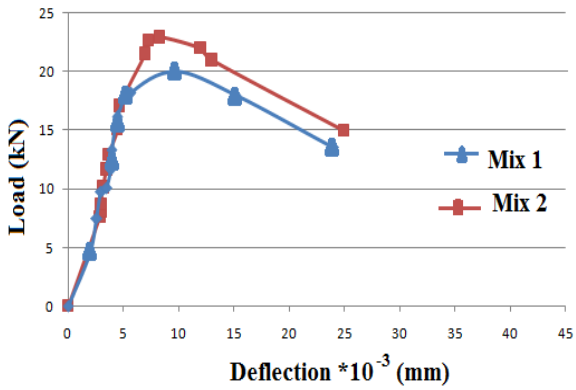
Water ingress into a non-saturated concrete structure is due to sorption, driven by the capillary forces. If the water is on top of the concrete surface, gravity will also play a role in the water penetration. To measure the sorption coefficient of concrete, a new test proposed by ASTM for standardization was used [29]. Tables (IX, X and XI) show the values of absorption for 3 SCC mixes. The results indicated that all concrete specimens exhibited a continuous reduction in absorption values with time of curing. The total absorption for all SCC specimens exposed to water was below 10% by weight. This gives an indication of good concrete (with low permeability) and it may be attributed to the continuous hydration of cement which decreases the absorption of SCC, as well as, due to the dense microstructure of SCC with low W/C ratio used and the presence of silica fume. This causes modification to the microstructure of concrete, reduces the capillary pores leading to better packing, increase the density and lead to reduction in the absorption percentage [30].

The normalized water absorption, *I*, was the change in mass divided by the product of the cross-sectional area of the test specimen by the density of water. For the purpose of this test, the temperature dependence of the density of water is neglected, and 4 decimal number as 0.0001and a unit g/mm^3 were used in the equation of sorptivity test according to ASTM C1585 [31].

The absorption test was carried on 3 mixes 1, 2 and 5 to conclude the effect of steel fibers and internal curing by fine LWA on the absorption results. Beside the decrease in the sorptivity of mix 5 may be due to the increasing in super-plasticizers' dosage to retain and maintain their flowability and passing ability of this mix in the fresh state. The increment in the super-plasticizers' dosage will lead also to more uniform microstructure and as a result of that the water sorption decreases and hence the durability performance of the mixes is improved. The increase in the sorptivity values of mix 2 may related to the presence of pre-wetted fine LWA which has more pores than ordinary fine aggregate which lead to more absorption.



a) 28 days



b) 90 days

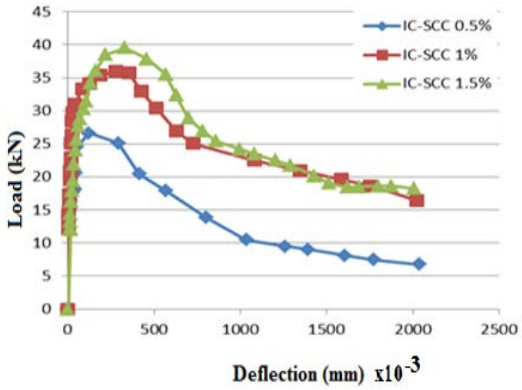
Fig. 1. Flexural load- deflection curves for mixes 1 & 2 at: a) 28 days, and b) at 90 days

Table IX
Absorption test for Ref mix 1 (SCC 0%)

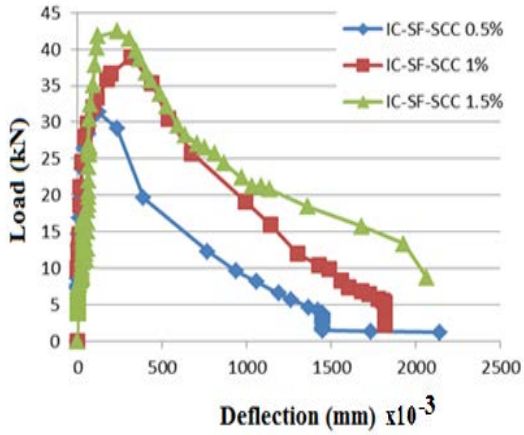
Time(sec)	$\sqrt{Time}$ ($\sqrt{sec}$)	Mass (gm)	Δ Mass (gm)	(I)(mm)
0	0	990	0	0
600	24	999	9	1.1464
3600	60	1002.4	12.4	1.5796
7200	85	1004	14	1.7834
92220	304	1009.8	19.8	2.5222
268500	518	1012.6	22.6	2.8789
432000	657	1013.7	23.7	3.0191
691200	831	1014.3	24.3	3.0955

Table X
Absorption test for Ref mix 2 (IC-SCC 0%)

Time (sec)	$\sqrt{Time}$ ($\sqrt{sec}$)	Mass (gm)	Δ Mass (gm)	(I) (mm)
0	0	852.3	0	0
600	24	858.1	5.8	0.7388
3600	60	865.7	13.4	1.707
7200	85	866.4	14.1	1.7961
86400	304	873	20.7	2.6369
259200	518	876.4	24.1	3.07
432000	657	877.5	25.2	3.2101
691200	831	878.8	26.5	3.37579



a) 28 days



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b) 90 days

Fig. 2. Flexural load- deflection curves for mixes3, 4 & 5 at: a) 28 days and b) 90 days.

Table XI
Absorption test for Mix 5 (IC-SF-SCC 1.5%)

Time (sec)	$\sqrt{Time}$($\sqrt{sec}$)	Mass (gm)	Δ Mass (gm)	(I) (mm)
0	0	942.4	0	0
600	24	947.2	4.8	0.6114
3600	60	952.7	10.3	1.3121
7200	85	953.1	10.7	1.3630
86400	304	961.3	18.9	2.4076
259200	518	963.2.	20.8	2.6496
432000	657	964.3	21.9	2.7898
691200	831	965.3	23	2.9299

VII. CONCLUSIONS

Based on the experimental work results in this investigation, the following conclusions can be drawn:

1. It has been verified that by using the slump flow, L-box, J-ring and V-funnel tests, SCC can be achieved by using locally available materials and has the ability to self-consolidating under its own weight only, without any external vibration or compaction. Adding fiber has a definite adverse effect on workability properties tests of fresh SCC. For which demand higher water or higher chemical admixture dosages to

keep the workability values within the acceptable limits. Fine LWA has a definite adverse effect on all workability properties of fresh SCC due to the higher roughness surface of fine LWA than ordinary fine aggregates.

2. All SCC mixes that incorporated hooked end steel fiber have slightly higher compressive strength at all curing ages and for both curing types (internal and normal curing) used where the maximum increase in compressive strength was12.9% at 28 days and 17.6% at 90 days.
3. The influences of steel fibers on the tensile strengths are better than their influences on the compressive strength.
4. For all mixes the results of modulus of rupture were higher than splitting testing results in a decreasing order for both curing types (internal and normal curing); and even with the presence of steel fibers.
5. The fine aggregate replacement as internal curing material caused better enhancement in mechanical properties (strengths) of SCC than coarse aggregate replacement.Meanwhile, the replacement for coarse aggregate caused decreasing in compressive strength at the same percentage.
6. The improvements in SCC strengths which caused by using fine LWA for internal curing and steel fibers were more than that caused by steel fibers only. The influences of internal curing on the flexural and tensile strength are better than their influences on the compressive strength.
7. Toughness of SCC was increased with the increase of steel fibers content and also with the application of internal curing for both 28 and 90 days ages.
8. There was an increase in the density of SF-SCC due to the presence of steel fibers, and there was a decrease in the density of internally cured SCC mixes due to the use of fine LWA for internal curing.
9. Regardless the used curing methods, the sorptivity of SCC decreases with the increase in steel fibers content for instance when the sorptivity of SCC 0% reached to 3.09mm after 8 days, the sorptivity of SF-SCC 1.5% reached to 2.77mm during the same time.
10. Application of internal curing, results in higher sorptivity values when compared with normally cured concretes for instance when the sorptivity of SCC 0% reached to 3.09 mm after 8 days and the sorptivity of IC-SCC 0% reached to 3.37mm after 8 days too.

REFERENCES

1. Okamura, H., “Self-Compacting High Performance Concrete,” ACI Concrete International, Vol. 19, No. 7, pp.50-54 July 1997.
2. Goodier CI. Development of self-compacting concrete. Proc ICE – Struct Build2003; 156(4):405–14.
3. Campion, J. M., and Jost, P., “Self-Compacting Concrete: Expanding the Possibility of Concrete Design and Placement” ACI Concrete International, Vol. 22, No. 4, pp.31-34, April 2000.
4. ACI Committee 237R-07, “Self-Consolidating Concrete”, Reported by ACI Committee 237, April 2007, Farmington Hills, MI 48331, USA.
5. EFNARC, "Specification and Guidelines for Self-Compacting Concrete", London, UK: Association House, pp. 32, February 2002.
6. American Concrete Institute Committee (308-01) "Guide to Curing Concrete", pp. 31.
7. American Concrete Institute Committee 213R-03, "Guide for Structural Lightweight Aggregate Concrete", Reported by ACI committee 213, ACI Manual of Concrete Practice, 2003.
8. Mehta, P.K., and Montero P.J.M., "Concrete: Microstructures, Properties and Materials", McGraw-Hill, United States of America, Third Edition, 2006, pp.40, 46, 49, 52, 55, 57 and 78.
9. Hoff, G.C., "The Use of Lightweight Fines for the Internal Curing of Concrete", Northeast Solite Corporation, 2002.
10. American Concrete Institute Committee (308-213) R-12,"Report on Internally Cured Concrete Using Prewetted Absorptive Lightweight Aggregate", June 2013, Farmington Hills, MI 48331, USA.
11. Bentz, D.P. and Snyder, K.A., "Protected Paste Volume in Concrete – Extension to Internal Curing Using Saturated Lightweight Fine Aggregate", Cement and Concrete Research, Vol. (29), No, (11), 1999, pp. 1863-1867.
12. Johnston, C. D.,”Fiber-reinforced cements and concretes”, Gordon and Breach Science Publishers, Amsterdam, The Netherlands, pp.(316), 2001.
13. Sujith, K. C.P, “Experimental study on steel fiber reinforced self compacted concrete with silica fume as filler material “, India Maniple University, India 2009.
14. EFNARC, “Specification and Guidelines for Self-Compacting Concrete” ISBN 0 953973344, 32 pp 2002.
15. EFNARC,"The European Guidelines for Self-Compacting Concrete Specification, Production and Use", 2005.
16. المواصفة القياسية العراقية رقم (45) لسنة1984”*ركام المصادر الطبيعية المستعمل فيالخرسانة والبناء*“, *الجهاز المركزي للتقييس والسيطرة النوعية*.
17. ASTM C1240 – 05” *Standard Specification for Silica Fume Used in Cementitious Mixtures*”, Annual Book of ASTM standard, Vol.04.02, 2005.
18. المواصفة القياسية العراقية رقم (5) لسنة 1984”الاسمنتالبورتلاندي", الجهازالمركزي للتقييس والسيطرة النوعية.
19. MBT Degussa Construction Chemicals, "*Glenium 51*", 06/97 MBT-ME Revised 06/2002, www.mbt-middle-east.com
20. Emborg, M., *"Mixing and Transport"*, Final report of task 8.1, Betongindustri AB, BriteEuRam, Sweden, 2000, 65 pp.
21. ASTM C 192/C 192M -07, *“Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory"*,ASTM International.
22. Teknik, D. B. *“Measurement of Properties of Fresh Self- Compacting Concrete”*,. Final Report, September, PP.15, 55, 57, 2005.

23. B.S 1881: Part 116, *‘’Method for Determination of Compressive Strength of Concrete Cubes’’*. , British Standards Institution. , 1989.
24. ASTM C496. (2004), *‘’Standard Test Method for Splitting Tensile Strength for Cylindrical Concrete Specimens’’*.Annual Book of ASTM standard, Vol.04.02, 2004.
25. ASTM C78-02.*’’Standard Test Method for Flexural Strength of Concrete"*, Annual Book of ASTM standard, Vol.04.02, 2002.
26. ASTM C 1585-13,*"Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic- Cement Concretes"* ,American Society for Testing and Material International, 2013.
27. الدليل الاسترشادي المرجعي رقم274 "فحوص الخرسانة وطرق تعيين كثافة الخرسانة المتصلبة", الجهاز المركزي للتقييس والسيطرة النوعية, 1992
28. ASTM C1609 M-10, *“Standard Test Method for Flexural Performance of Fiber Reinforced Concrete (Using Beams with Third-Point Loading)”*, Annual Book of ASTM Standard, Vol.04.02, 2010.
29. Dale P. Bentz and Mark A. Ehlen, "Sorpitivity-based service life predictions for concrete pavements"Reprinted from 7th International Conference on Concrete Pavements. Proceedings, Vol1, International Society for Concrete Pavements. , Orlando, Florida ,pp.181-193, 2001.
30. Jayeshkumar P.and Dr Umrigar F. S., " Evaluation of Sorptivity and Water Absorption of Concrete with Partial Replacement of Cement by Thermal Industry Waste (Fly Ash)",Vol 2, Issue 7, pp. 248, January 2013.
31. ASTM C 1585-13,*"Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic- Cement Concretes"* ,American Society for Testing and Material International, 2013.

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الجزء 2: محور هندسة مواد البناء وإدارة المشاريع