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The effect of Persian Gulf tidal zone exposure on durability of mixes containing silica fume and blast furnace slag

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Abstract

In this research the performance of cement paste and concrete mixes incorporating 7 and 10 percent of silica fume (S.F.) as a cement replacement was investigated in three exposure conditions. The results showed that plain type II portland cement performed better than blended S.F. cement under cyclic wetting and drying conditions. Silica fume specimens under cyclic wetting and drying conditions in simulated seawater exhibited higher strength loss compared to plain type II portland cement where cured under potable water. In addition, the greater the silica fume amount used in the mixes, the more the capillary water absorption under tidal zone exposure or/and under wetting and drying simulation. Further, the ternary blended GGBS (Ground Granulated Blast furnace Slag) mix was the worst performing mix in all exposure conditions.

Keywords: concrete durability; concrete deterioration; cement replacement materials; blast-furnace-slag; marine conditions.

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1. Introduction

In recent decades the use of silica fume (S.F.) as a supplementary cementitious material has increased substantially in concretes used for marine structures in the Persian Gulf. The reason for use of silica fume in these structures is claimed to be an increase in the durability of coastal structures (including those within the tidal zone).

The performance of concrete in sea water has been extensively studied [1–9]. The findings of these studies are at the same time inconclusive and sometimes controversial [10 and 11]. This reflects the complex nature of the durability problem which is attributed to one or more of the following factors [12].

- Characteristics of the ingredient materials and mix design characteristics such as water-cement ratio, cement to aggregate and fine to coarse aggregate ratios, degree of compaction, curing type and age, etc.
- Test methods adopted to assess the concretes.
- Duration and severity of the exposure period.
- Variability of the seawater concentration and the variation in its temperature.
- Specimen type (paste, mortar, concrete), shape and size.
- Use of small specimens to predict the behaviour of structural concrete.
- The dominant deterioration mechanism or mechanisms that can operate simultaneously.

One of the lessons learned from the case histories of concrete deterioration in seawater is that the individual processes of deterioration tend to limit themselves to different parts of a concrete structure, depending on the tidal lines [4]. Thus, a structure can be divided into three zones: the atmospheric zone (above the high-tide level), the tidal zone (between the tide lines), and the submerged zone (below the low tide level). The most

severe deterioration usually occurs in the tidal zone, which is the focus of this investigation.

The most aggressive chemicals that affect the durability of concrete structures in seawater are chlorides and sulfates with their associated cations. The chloride dissolved in water increases the rate of leaching of portlandite and thus increases the porosity and capillary absorption of concrete, and leads to loss of stiffness and strength. Calcium, sodium, magnesium, and ammonium sulfates are (in increasing order) harmful to concrete as they react with hydrated cement paste leading to expansion, cracking, spallings and loss of strength [13].

It is noteworthy that the hydrated cement phase that binds chloride ions is tricalcium aluminate (C_3A) forming an insoluble compound, Friedel's salt (calcium chloroaluminate hydrate), thereby removing a portion of chlorides from corrosive attack. This is the same phase that, in excess of a certain critical level, reacts with sulphate ions in the hardened state to form ettringite and gypsum, thereby leading to degradation of concrete into a non-cohesive granular mass and to disruptive expansion and cracking [14]. Although type V cement can increase the chemical resistance to sulphate attack, it also tends to reduce the corrosion resistance of reinforcement in presence of chloride ions. This is due to the lower content of C_3A in type V cement compared to type I and II that reduces the chloride binding capacity of cement and therefore promotes steel corrosion [15-17] in concrete. Type I cement with higher C_3A content is however more vulnerable to sulphate attack and performs worse than the other types of cement [18]. Other studies [11 and 19] show that type II cement performs better than types I and V cements in marine environments; for this reason type II cement was chosen for this investigation.

Moreover, use of supplementary cementitious materials such as silica fume and blast furnace slag with portland cement has increased significantly in the Persian Gulf region.

The rate at which the hardened cement paste is deteriorated due to exposure to harmful chemicals mainly depends on the concentration of the chemicals in water, the time of exposure, and the chemical resistance of the concrete. Extensive investigations have been carried out on the use of silica fume and blast furnace slag in concrete during the past two decades and have led to their widespread application in the construction industry [20-24]. Many national standards also determine the degree of attack, and this is primarily on the basis of the concentration of the aggressive substances. However, the chemical resistance of high-performance concrete using silica fume or combined silica fume and blast furnace slag is an issue that has not yet received sufficient attention from the research community [24]. Hence, this investigation was undertaken to study the effect of silica fume and GGBS blended with type II cement on progressive deterioration of pastes and concretes in the marine tidal zone of the Persian Gulf, and in laboratory tanks simulating severe conditions.

2. Experimental program

An experimental program was designed to produce a high-performance paste and concrete for the best protection in marine tidal zone by optimising silica fume [25] and GGBS replacement for the type II cement. The materials used and the experimental procedures are described in the following sections.

2.1. Materials

Locally available Type II portland cement (ASTM C 150 Type II) complying with Iranian specification 389 was used. The chemical composition, compound composition (calculated complying Bogue), specific gravity and the specific surface of the cement used are shown in Table 1.

Locally available silica fume and ground granulated blast furnace slag complying with ASTM C 1240 and ASTM C 989-93 (except the lower fineness of GGBS) were used with naphthalene sulphonate based high-range water reducing admixture (HRWRA) to achieve constant workability in all paste and concrete mixes. Table 1 details the chemical composition and the nitrogen BET fineness of the SF and the Blaine fineness of the GGBS used in this investigation. The slag activity index of the GGBS used was calculated as 67 percent based on ASTM C989 definitions.

Commercially available crushed siliceous aggregates complying with ASTM C33-78 were employed for both coarse and fine fractions to make concrete specimens. The relative density, absorption and fineness modulus of the fine aggregate were 2.65, 4.2% and 3.72, respectively. The specific gravity and absorption for coarse aggregate were 2.64 and 1.8%, respectively.

Reagent grade MgSO_4 , Na_2SO_4 , NaCl and MgCl_2 were used for sulphate and chloride exposure in wetting and drying simulation ponds. Potable water was used throughout for mixing, initial 7 days curing of all specimens, and curing in the control laboratory water tank. The chemical analysis of the water used is shown in Table 2.

2.2. Mix proportions

There were four basic mixes in which silica fume and GGBS were incorporated to make paste and concrete mixes. The first mix was the control Type II cement mix. The second mix was the optimum amount of silica fume replacement, i.e. 10 percent, which was adopted from the preliminary studies on physical and mechanical properties reported by Ganjian et al. [25]. The experiments [25] indicated that 10% replacement by silica fume achieved the highest compressive strength at the age of 28 days. In the third mix, 7 percent silica fume replacement was used in order to evaluate the effect of the amount

of silica fume on the strength loss. The fourth mix was based on the optimum replacement amounts of blended silica fume and GGBS with type II cement, which were 10 and 50 percent by weight of cement respectively as obtained by several trial mixes. The amount of silica fume in such trial mixes was kept constant at 10% of total mass of cementitious materials, and the amount of GGBS was varied to reach the highest relative compressive strength. The mix designs and characteristics of all pastes and concretes are given in Tables 3 and 4, respectively.

The amount of water in mixes was chosen based upon the total content of cementitious materials used in each mix. The water-cementitious materials ratio and per cent flow were kept constant for all mixtures and HRWRA was added to give constant workability. The required HRWRA per cent by mass of cementitious materials was obtained by trial and error from preliminary mixes.

2.3. Mixing and casting

There was particular interest in selecting a mixing procedure that would minimize dusting, especially during the initial stage of mixing and subsequent clustering of the SF particles after contact with water. For this reason the dry and wet methods of mixing SF and cement were examined. The dry mixing procedure of the portland cement, silica fume and GGBS proved to be the most effective in the dispersion of the SF particles and the most reliable method to obtain a homogenous mixture [25].

The following mixing procedure was used for paste mixes:

- 1- Dry mix the portland cement and SF (and GGBS if required) in a Hobart ASTM mixer at low speed for 60 seconds. It is imperative to use a plastic cover to prevent escape of fine particles.
- 2- Mix the required water with superplasticizer in a measuring vessel.

3- Without stopping the mixer, gradually add all of the solution (water and HRWRA) in next 60 seconds.

4- Stop mixer and change to medium speed. Resume mixing for further 60 seconds.

5- Stop mixer. Hand mix with a spatula by scraping the sides and dispersing the large visible clusters for 60 seconds.

6- Leave the mixture to stand for 30 seconds.

7- Resume mixing at high speed for 2 minutes.

Total time required is therefore six and a half minutes. The following mixing procedure was used for concrete mixes using a tilting drum mixer of 0.15 cubic meter capacity.

The interior of the drum was initially wetted with water to prevent absorption. The coarse aggregate fractions were mixed first, followed by the cement, part of the required amount of sand, and the water containing half of the required amount of superplasticizer. The final mixing stage involved the addition of the SF (and GGBS if required), and the remaining sand. One-half of the super-plasticiser was always retained with one litre of water to be added during the last three minutes of the mixing period.

50 mm and 100 mm cube moulds were used for paste and concrete specimens respectively, and a vibrating table was used to achieve full compaction. After casting, all paste and concrete specimens were covered with wet burlap in the laboratory at 20 ± 1 °C and 65 % relative humidity for 24 hours.

2.4. Initial curing

After de-moulding, all specimens were initially cured in a potable water-curing tank at laboratory temperature for 7 days and then transferred to the three specific exposure conditions. Chemical analysis of potable water used for making mixes and used for initial curing is given in Table 2.

2.5. Long term simulation ponds and tidal zone site exposures

Specimens were kept in three different conditions for longer ages as follows:

A) In fresh potable water curing tank, control curing regime (for both paste and concrete specimens) at 20 ± 1 °C.

B) In a simulation pond under cyclic wetting and drying in synthetic seawater solution (for both paste and concrete specimens). The concentration of major ions in the solution used for the simulation pond is given in Table 2. Simulation pond temperature was kept at about 37 to 42 °C according to dominant temperature and time intervals of wetting and drying cycles within tidal zone of south coast of Iran by using electric heater rods for drying and wetting period respectively. An ultraviolet lamp was mounted on the top of each pond to bring sunlight effect to the specimens. Owing to the fact that wetting and drying cycles were set to occur at 6 hourly intervals and temperature was kept higher than real site environment, the simulation conditions were harsher and more severe than site exposure tidal zone conditions. The solution in the simulation ponds was refreshed every two weeks.

C) In a coastal environment on the north coast of the Persian Gulf, in the south of Iran. The paste specimens with the same characteristics of those cured in water tank (condition 'A') and in the simulation ponds (condition 'B'), were placed in the seashore of Kish Island in the south of Iran. The concentration of major ions in the Kish Island's seawater is given in Table 2. In this exposure site the specimens were kept in tidal zone for 6 months, protected from waves by surrounding rock barriers. The specimens were on average subjected to 18 hours of wetting and 6 hours of drying per 24 hours. The average seawater temperature was approximately 33 °C.

2.6. Testing program

The various paste and concrete mixes were tested for compressive strength development, at ages of 3, 7, 28, 90, and 180 days. These tests were performed in accordance with BS 1881 (Part 116 1983) [26].

The water absorption test was carried out on pastes and concrete cubes in accordance with the method specified by RILEM-CPC-11.2 (RILEM 1994) [27] at 180 days of age. The Linear correlation between initial water absorption and square root of time was obtained. The tangent of the line i.e. the absorption factor was calculated to compare the effect of the admixtures on the pore structures of concrete and paste samples.

3. Experimental results and discussion

The results of the compressive strength tests for paste mixes are shown in Figs. 1 to 3 and for concrete mixes in Figs. 4 and 5. Reduction in compressive strength of paste and concrete specimens exposed to laboratory simulation ponds and site tidal zone compared with corresponding control strengths are plotted in Figs. 6 to 8. Water absorption measurements were taken at 3, 6, 24 and 72 hours and the absorption equations for each mix are given in Tables 5 and 6 for pastes and concretes respectively. The calculated absorption factors are shown in Figs. 9 and 10 for paste and concrete mixes respectively.

All of the paste and concrete mixes cured in fresh potable water (control curing) show an increase in compressive strength with age as expected (Figs. 1 and 4). The strength increases sharply up to 28 days and rate of increase slows at later ages. In both paste and concrete mixes, the optimum S.F. mix exhibited the highest strength development as expected due to the superior pozzolanic activity and pore refinement process of S.F. The blended GGBS/SF mix gave the lowest strength development. This was due to the lower portland cement contents in these mixes and the slower rate of reaction of GGBS.

In general, as expected, the S.F mixes show higher strength development than type II cement in control curing condition. This is in agreement with the absorption test results in Figs. 9 and 10, which show lower absorption factors for the control curing specimens. However, this strength development trend changes in other exposure conditions for paste and concrete. In simulation pond exposure, portland cement type II exhibits greater compressive strength development and less loss (Figs. 2 and 5). In general after initial 7 days curing the strength gain continues up to 28 days for all paste and concrete mixes in both exposure conditions but the harmful effect of environmental condition and seawater chemicals causes strength loss in pastes (Fig. 2) and slower rates of strength gain in concretes (Fig. 5). It can be postulated that the reason for further strength gain in concrete specimens (compare to loss of strength of paste specimens) is attributed to the bigger size of concrete specimens as the volume-related ion-intrusion/attack to the specimens would be less than for smaller paste specimens that are more sensitive to ion attack and induced cracks. In other words, the intrusion of ions throughout the bigger concrete specimens would not be deep enough to have a major influence on the compressive strength of samples when compared to paste specimens. Thus, further strength loss should be expected at later ages for concrete specimens in these exposure conditions.

Portland cement type II mixes exposed to laboratory simulation ponds or site exposure tidal zone exhibit lower percentage strength loss (see Figs. 6 to 8). Figs. 2, 5 and Figs. 6 to 8 show that S.F. paste and concrete mixes have greater strength loss compared to type II portland cement mixes in tidal zone and simulation pond exposure conditions, in contrast to their control curing condition in which S.F. mixes exhibit highest strength development. The greater intensity of the attack on portland cement plus silica fume specimens as compared to the portland cement specimens may be due to the absence of

magnesium hydroxide (Brucite) in the portland cement plus silica fume mixes. Magnesium hydroxide, formed as a result of calcium hydroxide reacting with magnesium sulphate, is relatively insoluble in water and therefore blocks the pores and protects the C-S-H gel from further attack. Its absence in portland cement plus silica fume specimens, therefore, makes the C-S-H gel more prone to magnesium sulphate attack. In addition, cyclic wetting and drying exposure will affect the crystallization of salts and results in degradation of the protective layer, facilitating the ingress of ions. The greater intensity of the attack may also be the result of the formation of an additional amount of C-S-H gel due to the pozzolanic reaction ('pozzolanic C-S-H gel'), which is different in composition from the C-S-H gel produced by the hydration of alite and belite of 'portland cement C-S-H gel' with respect to lower density and fewer hydrated molecules of water as explained by Cohen et al. [28] and Kjellsen et al. [29].

As shown in Table 5, replacement of cement with silica fume in pastes exposed to simulation ponds and site tidal zone has made the gradient coefficient of capillary equation to be increased by square root of time elapsed. This increase depends on the amount of silica fume replaced in the mix. The higher the amount of silica fume, the higher the gradient coefficient of absorption equations (see Figs. 9 and 10). This finding is in accordance with the research results and mechanism developed by Kjellsen [29]. It should be mentioned that the effect of amount of silica fume on absorption factor of concrete samples was not as distinctive as of paste mixes. This may be due to the size effect and less cement matrix in concrete specimens. Moreover, longer exposure of these specimens will lead to greater absorption factor in these specimens. In addition, the small accessible pore volume of these concrete specimens due to salt crystallization can be another explanation to the above observation. These figures also show that the absorption factors of specimens in simulation ponds are lower than corresponding

specimens in potable water. This is due to partial blockage of pores as a result of the salt crystallization in the pores network. Furthermore, using GGBS in the mix causes the absorption factor to be profoundly increased in simulation ponds and tidal zone exposure conditions. Therefore it can be concluded that the pore structures of GGBS plus SF mixes under wetting and drying condition in deleterious solutions are more porous than others. GGBS is considered to be effective in refining of the pore structure and improving the durability of concrete against sulphate attack; however, the higher percentage of GGBS, which is a slow-reacting cementitious material (especially the GGBS used in this study with lower fineness value than ASTM specification), together with silica fume replacement and insufficient initial curing could make GGBS mixes more vulnerable to sulphate attack. It is considered that the hydration product at 7 days when placed in site tidal zone or simulation pond exposure is not dense enough to prevent the ingress of ions [30]. Therefore, it is imperative to cure the concrete for an extended period of time when using supplementary cementitious material in wetting and drying exposures. Moreover, the deterioration of binder matrix due to magnesium sulphate and decomposition of C-S-H gel to M-S-H, which is non-cementitious, can be another probable reason for this phenomenon.

These findings are in agreement with the results of other investigators [28–33]. Cohen and Bentur [28] showed that adding 15% silica fume to cement paste (type I and V) exposed to 5% sodium sulphate solution resulted in improving its durability, but when exposed to magnesium sulphate solution a major deterioration resulted due to softening of C-S-H and converting it to hydrated magnesium silicate with loss of strength and mass. With the presence of silica fume, the decrease in strength observed was 5 to 10 times larger than in pastes without this pozzolan [28]. Likewise, Hekal et al. [31] reported that partial replacement of portland cement with 10 to 15% of silica fume did

not show a significant improvement in sulphate resistance of hardened cement pastes. The decreased amount of magnesium hydroxide, due to its reaction with hydrated silicate, and also the formation of $M_4SH_{8.5}$ which is non-hydraulic, increase the exposure of cementitious C-S-H to aggressive components. In the same way, the durability study on mortar specimens carried out by Moon et al. [32] and Lee et al. [33] confirmed that the conversion of secondary C-S-H gel into M-S-H (or M-C-S-H) gel, which results in the softening of cement matrix, eventually led to the pronounced deterioration of silica fume blended mixes exposed to magnesium sulphate environment. Magnesium silicate formation in concrete exposed to seawater was reported by Cole [34] and by Roy et al. [35] who illustrated that the formation of Mg-rich phases is normally associated with the near-surface regions of concrete exposed to Mg-containing solutions such as seawater. Furthermore, investigation carried out by Nehdi et al. [36] indicated that decalcification of CSH was significant in presence of magnesium sulphate and silica fume did not show any additional benefits in resisting surface deterioration and even led to worse performance than OPC specimens. Also Brown and Doerr [37] established the mechanism by which Mg is transported throughout the pore structures of site concretes even though magnesium hydroxide is known to be highly insoluble in pore solution of concrete. Further studies are needed to determine the behaviour and mechanism of deterioration of ternary blended mixes.

In summary, the results discussed in this section clearly demonstrate the complex effect of silica fume on the durability of cement mixes in seawater and under wetting and drying conditions. The presence of S.F. is beneficial in normal water, or other solutions where the deterioration mechanism is the result of expansion and believed to be associated with ettringite formation. In seawater solutions with high concentration of

magnesium sulphate, a major cause for attack is decomposition of C-S-H; in this case a silica fume addition (and GGBS) becomes detrimental.

4. Conclusions

The results of an investigation into potential mixes for use in tidal zone marine structures in Persian Gulf waters revealed the detrimental effect of silica fume. The specific conclusions that can be drawn from this study are as follows:

1. In potable water, the S.F. mix exhibits the highest strength development, while the blended ternary GGBS - SF mix exhibits the lowest strength development.
2. In laboratory simulation ponds and site tidal zone exposures, type II portland cement exhibits better compressive strength development. Type II portland cement mixes exhibit lower percentage of strength loss than S.F. mixes or blended ternary S.F. - GGBS mixes.
3. The strength loss and the capillary absorption factors show that in general the higher the amount of silica fume, the higher the strength loss and the higher the absorption factor in simulation and tidal zone exposure conditions.

Furthermore, using GGBS in the mix causes the absorption factor to be profoundly increased in simulation ponds and tidal zone exposure conditions. However, for concrete samples stored in simulation pond, SF and GGBS had less influence in the absorption factor than the paste mixes.

4. This study indicates that the effect of silica fume in various seawater environments is not clear-cut. There is a need for a thorough understanding of its influence on the different deterioration mechanisms to predict its potential for improvement in durability. For a long-term durable performance of concrete in marine and offshore tidal zone structures, S.F. or GGBS cement can be used in low magnesium sulphate

concentration waters. To further enhance the resistance against deterioration by sulphate ions and / or salt crystallisation, additional protective measures, such as the application of a water-resistant epoxy based coating, may be needed.

5. It is imperative to cure the concrete for an extended period of time when using supplementary cementitious material in wetting and drying exposures.

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Figure Captions:

Fig. 1- Compressive strength development with age for paste mixes cured in fresh potable water (control curing).

Fig. 2 - Compressive strength development with age for paste mixes exposed to simulation ponds.

Fig. 3 - Compressive strength development with age for paste mixes exposed to site tidal zone.

Fig. 4 - Compressive strength development with age for concrete specimens cured in fresh potable water (control curing).

Fig. 5 - Compressive strength development with age for concrete specimens exposed to simulation ponds.

Fig. 6 - Reduction in compressive strength of pastes exposed to simulation ponds.

Fig. 7- Reduction in compressive strength of concretes exposed to simulation ponds.

Fig. 8 - Reduction in compressive strength of pastes exposed to site tidal zone.

Fig. 9 - Absorption factors for pastes exposed to different conditions.

Fig. 10 - Absorption factors for concretes exposed to different conditions.

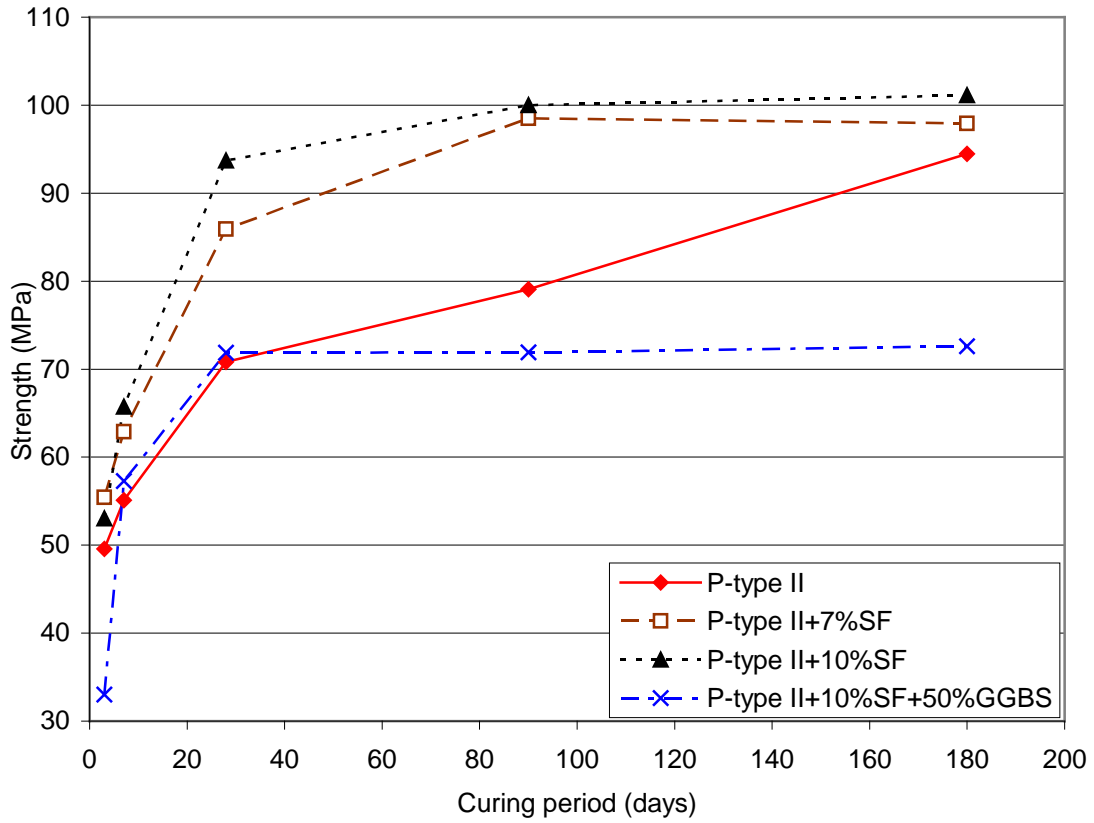


Fig. 1- Compressive strength development with age for paste mixes cured in fresh potable water (control curing).

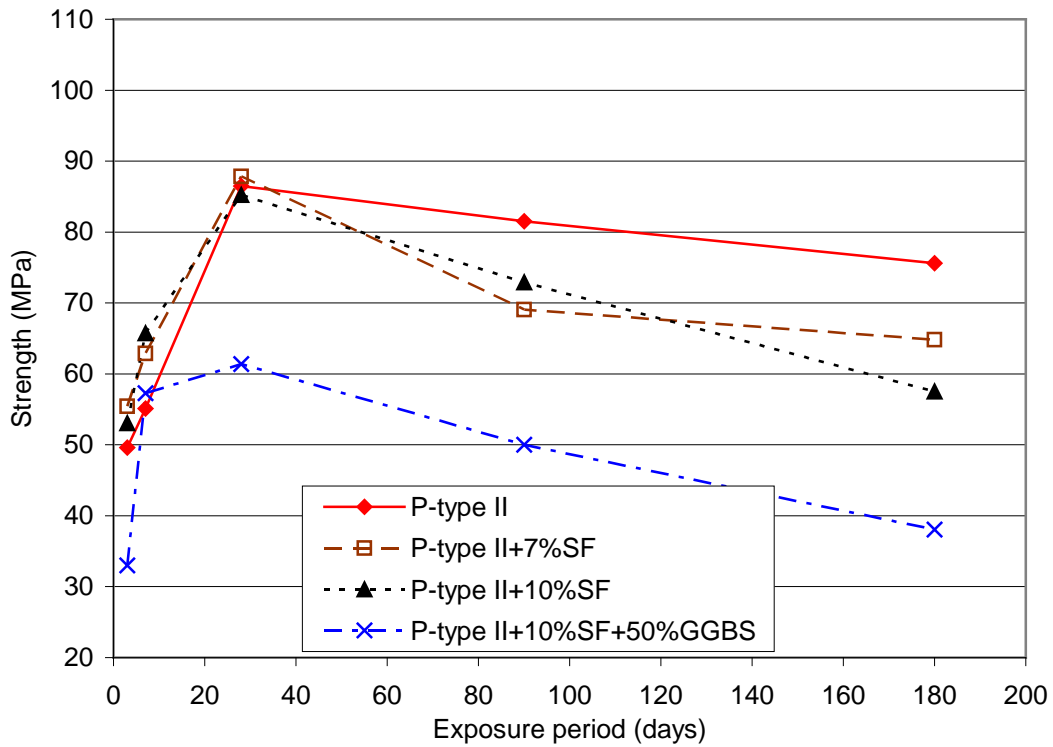


Fig. 2 - Compressive strength development with age for paste mixes exposed to simulation ponds.

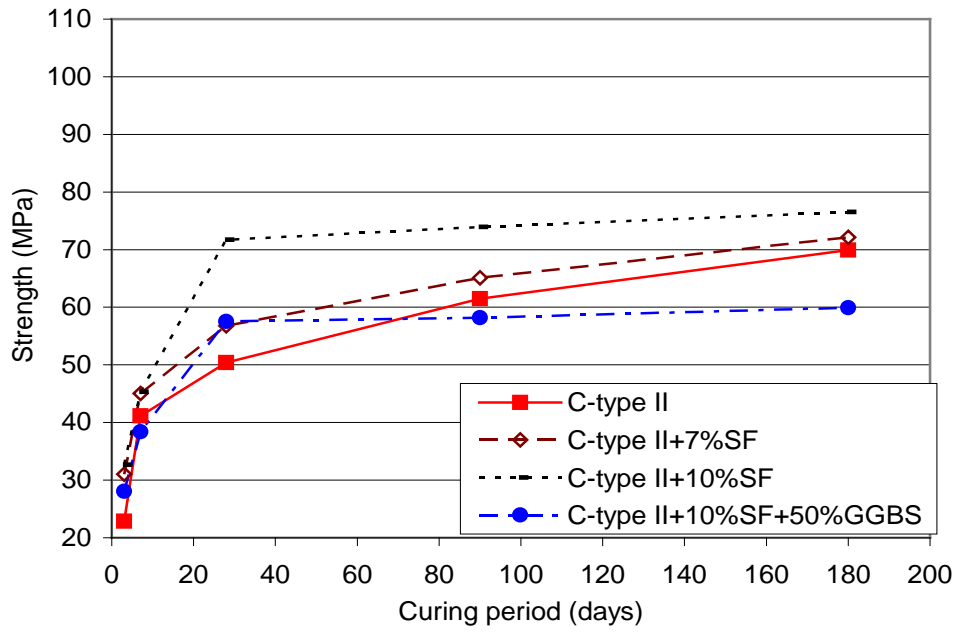


Fig. 3 -Compressive strength development with age for paste mixes exposed to site tidal zone.

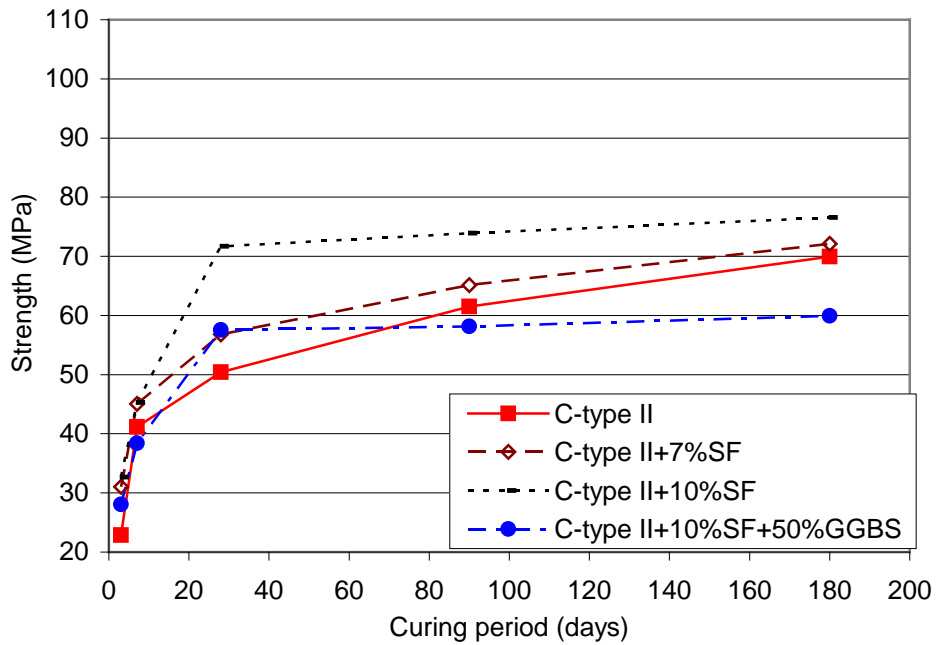


Fig. 4- Compressive strength development with age for concrete specimens cured in fresh potable water (control curing).

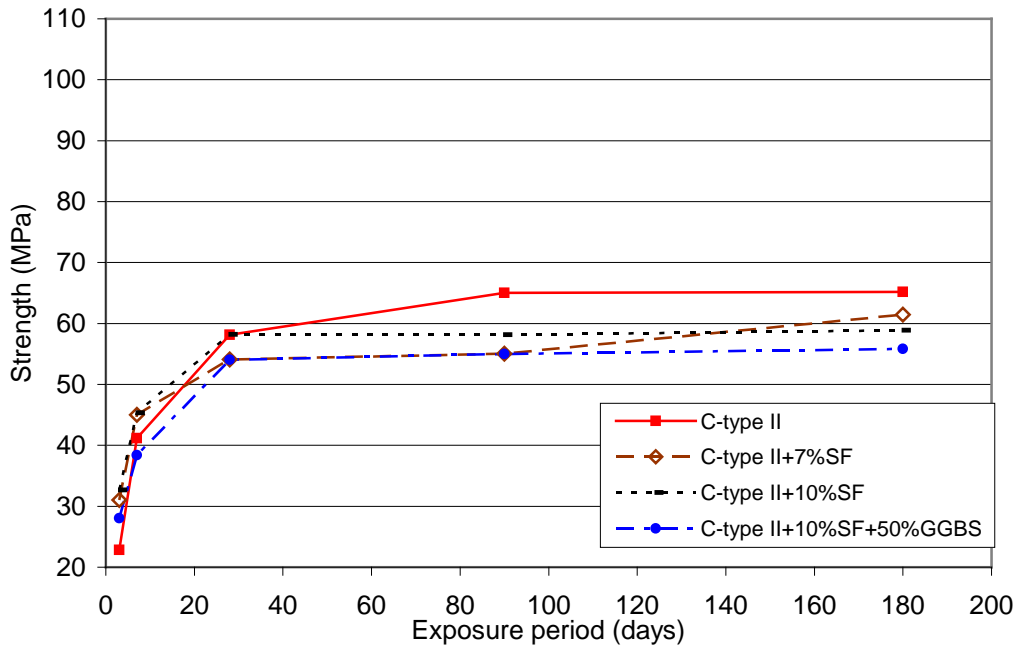


Fig. 5– Compressive strength development with age for concrete specimens exposed to simulation ponds.

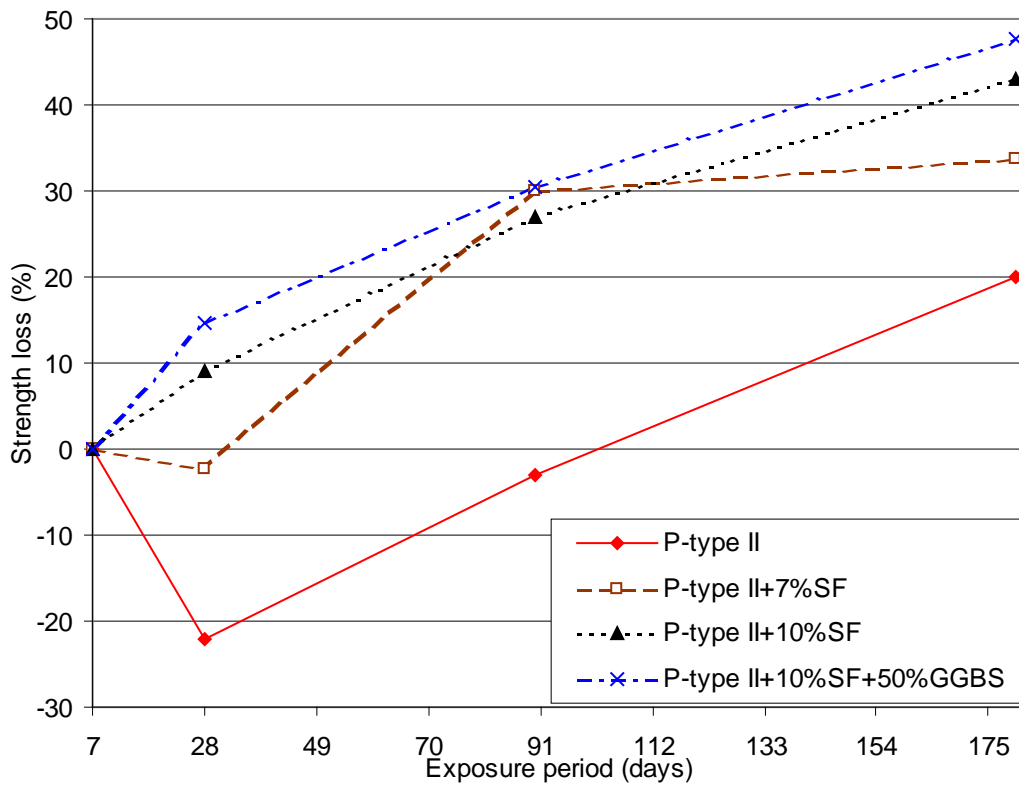


Fig. 6- Reduction in compressive strength of pastes exposed to simulation ponds.

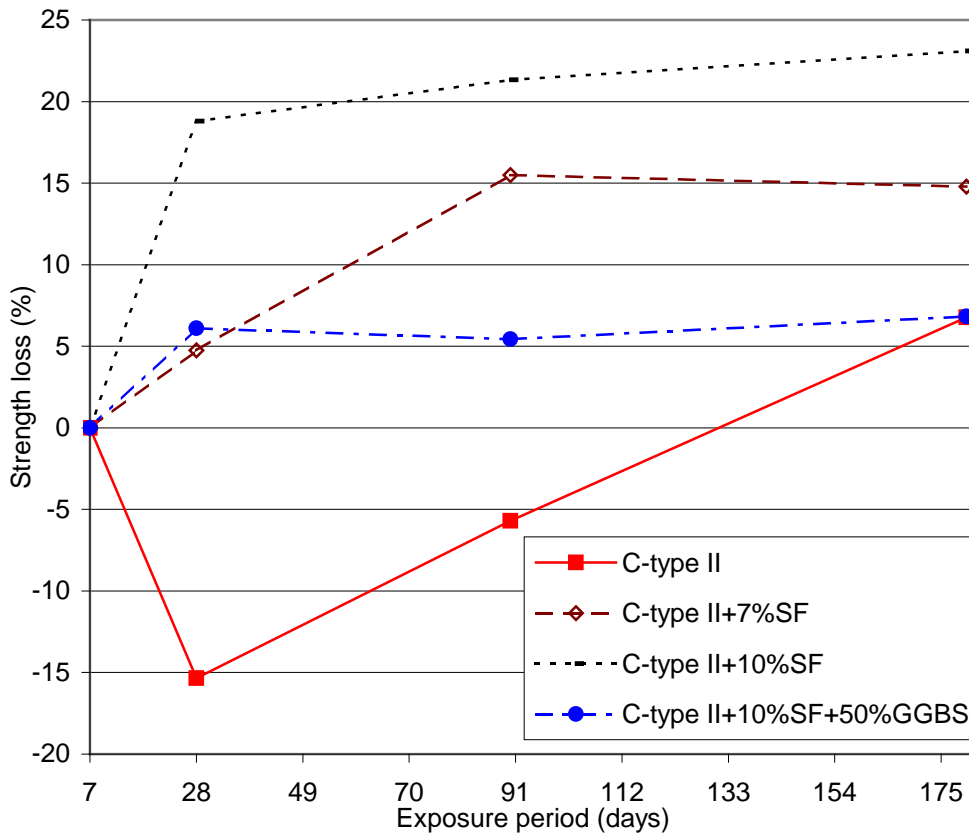


Fig. 7- Reduction in compressive strength of concretes exposed to simulation ponds.

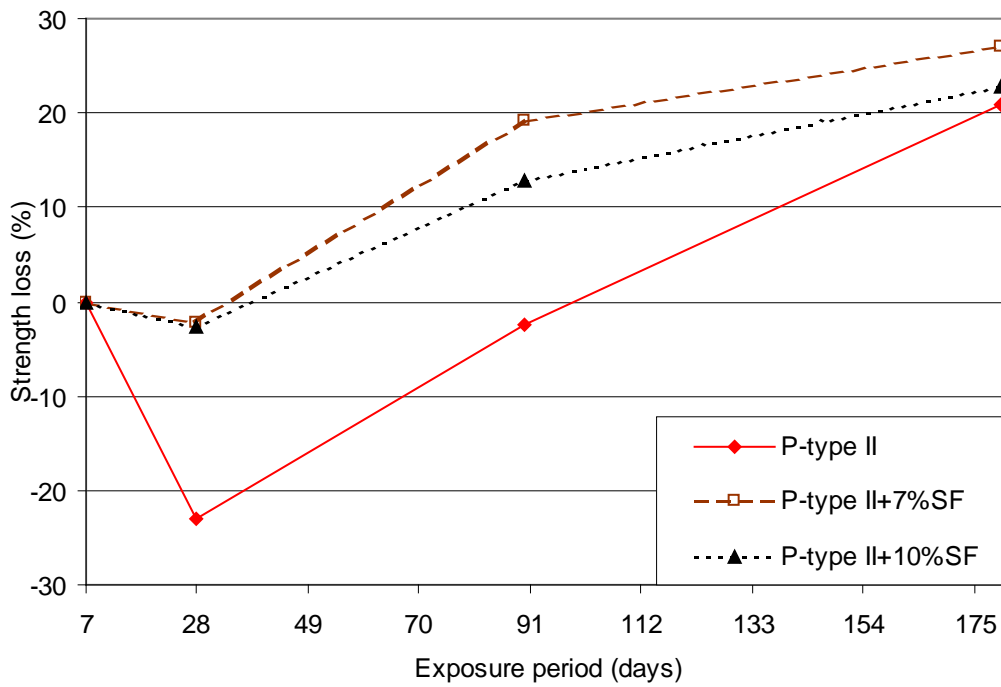


Fig. 8- Reduction in compressive strength of pastes exposed to site tidal zone.

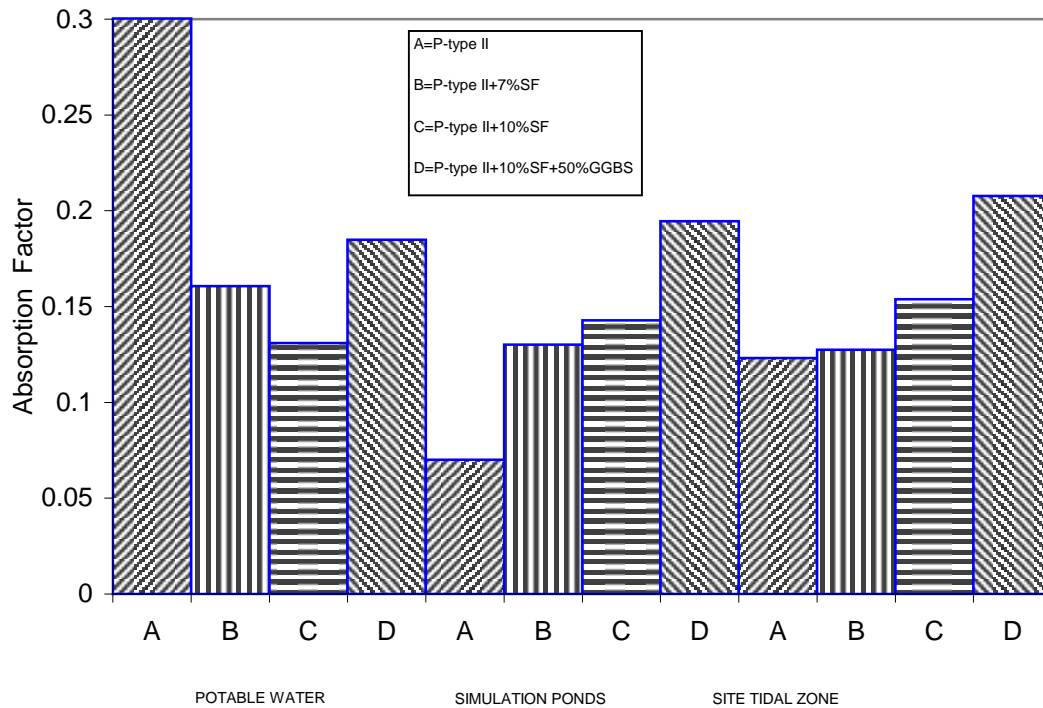


Fig. 9- Absorption factors for pastes exposed to different conditions.

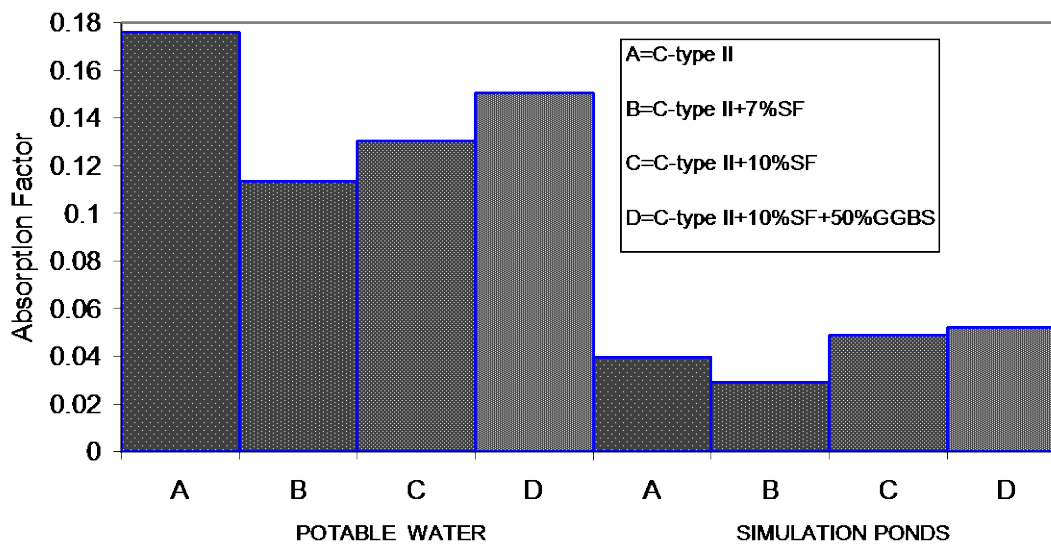


Fig. 10- Absorption factors for concretes exposed to different conditions.

Table 1 - Chemical analysis and physical properties of cement, silica fume (SF) and ground granulated blast furnace slag (GGBS) used.

Chemical composition (%)	Cement	SF	GGBS
SiO ₂	21.7	91.7	33.03
Al ₂ O ₃	4.9	1.27	10.87
Fe ₂ O ₃	3.2	1.45	0.25
CaO	62.4	1.69	38.45
MgO	3.58	0.6	10.26
SO ₃	1.84	0.45	<0.1
Na ₂ O	0.35	---	0.65
K ₂ O	0.95	---	0.78
Cl	0.017	0.08	0.007
Loss On Ignition	1	2	0.25
Physical properties			
Relative Density	3.15	2.2	2.5
Specific surface (m ² /kg)	290.8	14 310 ³	200
Compound composition (%)			
C ₃ S	46.3	---	---
C ₂ S	27.1	---	---
C ₃ A	7.6	---	---
C ₄ AF	9.7	---	---

Table 2 - Chemical analysis of the water used for mixing, initial curing, control curing, Kish island site exposure seawater and simulation ponds exposure

Constituent	Tap water	Kish island seawater
	ppm	ppm
Na	173	11400
K	6	397
Ca	24	450
Mg	16	1600
Cl	372	22330
SO ₄	23	3070

Table 3 - Characteristics of the pastes made.

<i>Paste mix</i>	<i>W/B</i>	<i>Superplasticizer (%)</i>	Flow (%)
P-type II	0.2	1.5	63 ± 5
P-type II+7%SF	0.2	1.8	64 ± 5
P-type II+10%SF	0.2	2	65 ± 5
P-type II+10%SF+50%GGBS	0.2	2	65 ± 5

Table 4 - Concrete mix proportions made (1 m³)

Concrete mix	Cement	SF	GGBS	Sand	Gravel	Water	W/B	Super-plasticizer	Approximate Slump
	(kg)	(kg)	(kg)	(kg)	(kg)	(kg)	(ratio)	(kg)	(mm)
C-type II	400	-	-	736	1104	160	0.4	4.8	20
C-type II+7%SF	372	28	-	728	1104	160	0.4	5.2	20
C-type II+10%SF	360	40	-	723	1104	160	0.4	6	20
C-type II+10%SF+50%GGBS	160	40	200	680	1104	160	0.4	6	20

Table 5 - Results of absorption by capillary test for pastes**(I=absorption percent, $x=\sqrt{t}$).**

Paste mix	Capillary absorption equation	Correlation coefficient	Exposure condition
P-type II	$I=0.3003x+0.881$	0.92	Potable water
P-type II+7%SF	$I=0.1607x+0.492$	0.90	Potable water
P-type II+10%SF	$I=0.131x+0.484$	0.93	Potable water
P-type II+10%SF+50%GGBS	$I=0.1849x+0.0581$	0.98	Potable water
P-type II	$I=0.0701x+0.0824$	0.99	Simulation ponds
P-type II+7%SF	$I=0.1301x+0.1261$	0.99	Simulation ponds
P-type II+10%SF	$I=0.1428x+0.228$	0.99	Simulation ponds
P-type II+10%SF+50%GGBS	$I=0.1945x+0.2133$	0.98	Simulation ponds
P-type II	$I=0.1231x+0.4368$	0.94	Site tidal zone
P-type II+7%SF	$I=0.1275x+0.4791$	0.94	Site tidal zone
P-type II+10%SF	$I=0.1539x+0.4792$	0.96	Site tidal zone
P-type II+10%SF+50%GGBS	$I=0.2077x+0.4982$	0.96	Site tidal zone

Table 6 - Results of absorption by capillary test for concretes

(I=absorption percent, $x=\sqrt{t}$).

Concrete mix	Capillary absorption equation	Correlation coefficient	Exposure condition
C-type II	$I=0.1762x+0.908$	0.97	Potable water
C-type II+7%SF	$I=0.1136x+1.1037$	0.96	Potable water
C-type II+10%SF	$I=0.1303x+0.9177$	0.96	Potable water
C-type II+10%SF+50%GGBS	$I=0.1507+0.5141$	0.97	Potable water
C-type II	$I=0.0399x+0.2396$	0.91	Simulation ponds
C-type II+7%SF	$I=0.0292x+0.197$	0.97	Simulation ponds
C-type II+10%SF	$I=0.0489x+0.2802$	0.99	Simulation ponds
C-type II+10%SF+50%GGBS	$I=0.0521x+0.2942$	0.98	Simulation ponds