ERMCO

EUROPEAN READY MIXED CONCRETE ORGANIZATION

XVII. ERMCO CONGRESS

MILITARY MUSEUM • ISTANBUL • TURKEY

4 - 5 June 2015

PROCEEDINGS

Organized by

Turkish Ready Mixed Concrete Association

XVII. ERMCO CONGRESS PROCEEDINGS

Published by Turkish Ready Mixed Concrete Association.

Türkiye Hazır Beton Birliği

Selvi Çıkmazı No: 2 Plaza K Kat:3

34805 İstanbul, TURKEY

Phone: +90 (216) 322 96 70

Fax: +90 (216) 413 61 80

info@thbb.org - www.thbb.org

www.ermco2015.com

First Publication: June 2015

ISBN: 978-605-84253-0-9

PREFACE

Founded in 1988 and located in Istanbul, Turkish Ready Mixed Concrete Association (TRMCA) is an NGO (non-governmental organization) which promotes standardized ready mixed concrete production and use in Turkey. TRMCA is also representative of the interests of ready mixed concrete producers in the country. TRMCA is a leading authority and resource for development and control of concrete quality in Turkey. TRMCA also takes part in the development and distribution of national and international standards. Our organization provides necessary tests, technical resources, and educational programs for concrete producers and other parties who work in the fields related to concrete.

TRMCA is made up of three parts: at the heart of TRMCA there is Quality Assurance System (KGS), and also there are Construction Materials Laboratory and Education and Training Unit.

Main goal of our organization is to promote and widen the use of controlled and standardized concrete. For this purpose we have founded Quality Assurance System in 1995 and started a certification program. This program aims to increase the number of registered concrete producers among the country. Necessary tests for the certification are conducted in our laboratory by highly qualified technical staff. Construction Materials Laboratory is located in the campus of Yıldız Technical University and employs a chemical engineer (PhD), a civil engineer (MSc), a physicist engineer (MSc) and six technicians.

TRMCA works in collaboration with international organizations. TRMCA is a proud member of ERMCO since 1991 and we have been an active participant of the activities of ERMCO ever since.

We are glad to host ERMCO congress for the second time in Istanbul on 4-5 June. After 20 years, we are happy to reunite the ERMCO family here, in Istanbul. We also have the honor of hosting Dr. Surendra P. Shah as the keynote speaker in the congress.

We hope and believe that XVII. ERMCO Congress will be remembered as a worthwhile and productive congress for all participants.

As TRMCA, we thank to all participants coming from all over the world, our scientific committee and sponsors.

Yavuz Işık

President of TRMCA

PREFACE

Friends, colleagues and participants in the ERMCO Congress,

A few words of introduction to the Proceedings of the Congress. Times remain difficult for the ready-mixed concrete industry in most of Europe. The shining exception to this picture is here in Turkey, which is among the leaders in cement and concrete production, not only in our region, but also globally. In Turkey, significant advances are being made in developing housing, infrastructure and services. In this development, of course, full use is being made of the world's favourite construction material, ready-mixed concrete, which remains one of the engines of progress; and of course the Turkish ready mixed Association, THBB, is leading progress made in the country.

After 20 years, our three-yearly Congress has come round again to Istanbul and in view of the current position of our industry, and in view of this stunning city venue, we are delighted that THBB has hosted the meeting. I feel that 2015 presents an opportunity to take stock of where we are, and more importantly, where we are going. What is new? What does the future hold for the readymix industry? What does readymix have to offer in the 21st century? I am therefore delighted that THBB has been able to put together such a varied and interesting programme which addresses some of these issues. Papers presented in key sessions of the Congress, such as the Sustainability of Concrete Solutions, and Contributions of Concrete to our Society, help us see more clearly what the answers might be to some of these questions, to asses our material continuous brilliant future.

Stein Tosterud

ERMCO President

SCIENTIFIC COMMITTEE
(in alphabetic order)

ERMCO

Francesco Biasioli

John Gibbs

Tom Harrison

TRMCA

Yılmaz Akkaya	Abdurrahman Güner	Canan Taşdemir
Süheyl Akman	Mustafa Karagüler	Mehmet Ali Taşdemir
Fevziye Aköz	Fahriye Kılınçkale	Mustafa Tokyay
Saim Akyüz	M. Hulusi Özkul	Fikret Türker
Bülent Baradan	Erbil Öztekin	İsmail Özgür Yaman
Turhan Y. Erdoğan	Turan Özturan	Nabi Yüzer
Şakir Erdoğdu	Kambiz Ramyar	Asım Yeğinobalı

TABLE OF CONTENTS

SESSION ONE

Sustainability of concrete solutions

RECYCLING RINSING WATER OF TRUCK MIXERS IN READY-MIXED CONCRETE PRODUCTION

M. Hulusi Ozkul and U. Anil Dogan

Faculty of Civil Engineering, Istanbul Technical University, Istanbul, Turkey

Abstract

The purpose of this paper is to analyze the possibility of reuse of waste wash water obtained from a ready-mixed concrete plant as mixing water in concrete production. The wash water contains suspended particles, for this reason, its density is higher than that of tap water. In the concrete production, several densities for mixing water, such as 1.00, 1.02, 1.04, 1.05 1.10, 1.15 and 1.20 kg/ℓ were used. Since wash water contains solid particles, a reduction is made in concrete mixture either from the cement content (those named as cement-reduced mixtures) or from the sand content (sand-reduced mixtures) by equal volume. Compressive strength tests were performed at the ages of 3, 7 and 28 days. Permeability properties were obtained by using capillary water absorption at 28 days of age. Test results showed that concretes prepared with wash water were able to meet the performance criteria in terms of compressive strength, workability and sorptivity when the density of the mixing water was chosen as less than 1.05 kg/ℓ.

1. INTRODUCTION

The disposal of excess concrete and rinsing water from ready-mix concrete truck mixers are becoming an increasingly greater environmental concern. The rinsing water of truck mixers at a ready-mixed concrete plant is called wash water. High demand for ready-mixed concrete, increased the amount of concrete to be disposed which returned for any reason. Considerably high amount of water is consumed to wash out the remaining concrete in the truck mixers, which is classified as hazardous waste due to its high alkalinity. Since the quantities of fresh water required to neutralize 1 ℓ of waste water having pH values of 10, 11 and 12, are 10,000, 100,000 and 1,000,000 ℓ, respectively, recycling of wash water is increasingly becoming a more important issue. Hence, waste fresh concrete is discharged in settling basins both for economic and environmental reasons.

An average of 500 litres of water is used to clean a truck mixer after each loading [1] and about 1000 litres is added as mixing water in each truck. In other words, around 50 to 100 litres of wash water is discharged per cubic meter of ready-mixed concrete [2, 3]. Recycling of wash-out water as mixing water in concrete, instead of disposing, is both an environmental friendly and economical solution. In order to achieve sustainable development, we have to extend utilization of wastes without diminishing quality or varying properties. For this reason we need to investigate performance of various combinations of ingredients of recycled wash water and understand the interactions between them.

There is relatively less number of reference in the literature regarding the reuse of rinsing water in ready-mixed concrete technology. A group of researchers reported shorter setting times and lower workability for the concretes prepared with wash water [4]. In another study increasing solid content suspended in mixing water accelerated the initial setting time of concrete, due to the increasing amount of hydrated cement and calcium hydroxide within fresh concrete [5]. On the other hand, Sandrolini and Franzoni [6] found reduction of capillary water absorption and porosity due to existing of fine particles in sludge water. Similarly, Borger et al. [7] showed that ages up to 8 hours wash water can be successfully used in producing fresh concrete with the resulting concrete strength equal or higher than that of concrete made with tap water. Some researchers added retarders into the wash water to prolong the hydration reaction of cement particles suspended in water [2]. Although fresh properties were affected, no impact on mechanical properties, rapid chloride permeability and time to corrosion onset, were observed. A more recent study demonstrated significant difference between the 28-day compressive strength of concrete prepared with wash water and that of concrete made using tap water for two different mixture proportions [8]. Across all the test groups, concretes made with wash water had higher compressive strength than concrete made with tap water. On the other hand, for the plain concretes having similar workability, slight reduction in compressive strength [9] and increase in chloride penetration [10] were reported. The reactive solid particles suspended in wash water can be substituted with a part of cement or sand [11] and different effect of each case were reported.

The purpose of this study is to investigate the recycling of the significant amount of waste wash water that is released from a ready-mixed concrete plant. The fine particles exist in wash water were used as either cement or sand substitution in the concretes prepared with wash water. The effect of wash water on the compressive and sorptivity properties of concretes were investigated.

2. EXPERIMENTAL

2.1 Materials

An ordinary Portland cement, CEM I PC 42.5 (in accordance with EN 197-1) was used. Natural and crushed stone sand were utilized as fine aggregates with specific gravities of 2.65 and 2.72, respectively. Crushed limestone with a maximum size of 22 mm and a specific gravity of 2.74 was used as coarse aggregate. A high-range water-reducing (HRWR) admixture (based on polycarboxylate) was employed with a constant dosage of 1.3% (as weight percentage of cement) in all concrete mixtures except those which have equal workability with the control mix by increasing admixture content. The wash water was obtained from the basin of a ready-mixed concrete plant after agitating the slurry mixture and the specific gravity of solid particles was measured as 2.12 g/cm^3, after drying the wash water samples in an oven. The chemical composition and physical properties of wash water are given in Table 1 together with the limits (EPA and EN standards).

2.2 Methodology

Mainly two series of concretes were prepared in this study, such as cement-reduced and sand-reduced. Solid particles exist in the wash water were used as a partial replacement of either cement or sand for each series, respectively. In other words, cement content of the mixture was reduced for considering the solid particles as cementitious material, and in the second case natural sand content was reduced in considering these particles as filler material. Apart from the control mix, prepared with tap water, wash water in five different specific gravities (1.02, 1.04, 1.10, 1.15 and 1.20 kg/ℓ) were used. Since the amount of solid particles can be calculated by the specific gravity of the water, it is replaced with cement in the 4 of 8 batches and with natural sand in the remaining 4 batches, by volume. An equal volume of water to solid particles was added in order to keep constant amount of water in each mixture. The first six mixes were prepared with HRWR ratio of 1.3% and in the last two mixes (1.15MC and 1.15MS), HRWR dosage was increased to obtain workability similar to the control mix.

Table 1: Results of wash water analysis

Component	**Result**	**Limit Value**	**Related standard**
SO_4^{2-}	31 ppm	2000 ppm	EPA 9038
Cl^-	29 ppm	Prestressed concrete 500 ppm	EN 196-2
Na_2O	90 ppm	-	EN 196-2
K_2O	107 ppm	-	EN 196-2
Total alkali	160 ppm	1500 ppm	EN 196-2
pH	12.6	≥ 4	EN 1008
Odour	no	No smell, except the odour allowed for potable water	EN 1008
Colour	transparent	Pale yellow or paler	EN 1008
Suspended matter	0.115 kg/ ℓ d=1.056	Maximum 4 ml. sediment	EN 1008
Organic matter	Light yellow	Yellowish brown or paler in NaOH solution	EN 1008
Pb^{2+}	< 10 ppm	100 ppm	EN 1008
P_2O_5	< 10 ppm	100 ppm	EN 1008
Zn^{2+}	< 10 ppm	100 ppm	EN 1008
Nitrat (NO^{3-})	< 10 ppm	500 ppm	EPA 9038

In the coding of the mixtures, the starting numbers of the designations indicate the specific gravity of wash water and the last letter denotes the replaced component of the concrete (C: cement, S: natural sand) while M designates the modification of workability by increasing HRWR content.

Particularly for cement reduced concretes water to cement ratios were slightly increased because of the difference between the densities of solid particles in wash water and cement. Since the replacement was made by volume but the W/C ratio was by mass, the mixture proportions were changed to a degree.

All the concrete mixtures were prepared in a pan mixer with 0.04 m^3 volume and 150 x 150 x 150 mm cubes were used to test compressive strength on the 3rd, 7th and 28th days. For the sorptivity measurements, 70 x 70 x 280 mm prisms were tested on the 28-day of age.

Table 2: Mixture proportions of the concretes prepared in the first step

Concrete designations	**Materials (kg/m^3)**				**Slump (cm)**
	Cement	**Water**	**Solid content of mix water**	**Natural sand**	
Control	330	190	-	274	16
1.02C	320		7.58	274	13
1.02S	330		7.58	265	15
1.04C	310		13.6	274	15
1.04S	330		13.6	257	16
1.10C	276		33.9	274	15
1.10S	330		33.9	231	16
1.15C	249		51.7	274	6
1.15S	330		51.7	208	5
1.20C	223		69.4	274	4
1.20S	330		69.4	185	3
1.15MC	249		51.7	274	15
1.15MS	330		51.7	208	15

3. TEST RESULTS AND DISCUSSION

3.1 Compressive strength

The normalized results of compressive strength tests conducted on the concretes are also illustrated in Figures 1, 2 and 3 for 3, 7 and 28 days of age, respectively. The filled points and the solid line represent the sand reduced mixtures while the unfilled points and dotted line stand for cement reduced mixtures.

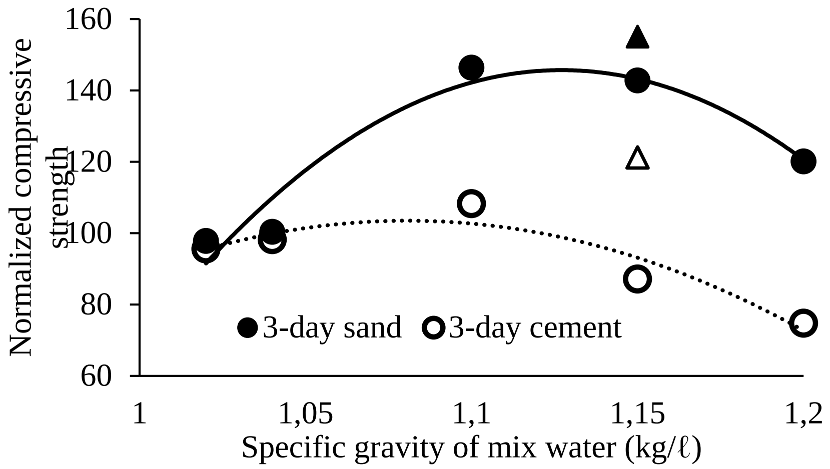


Figure 1: Normalized compressive strength at the age of 3 days (Triangle points correspond to concretes with modified workability)

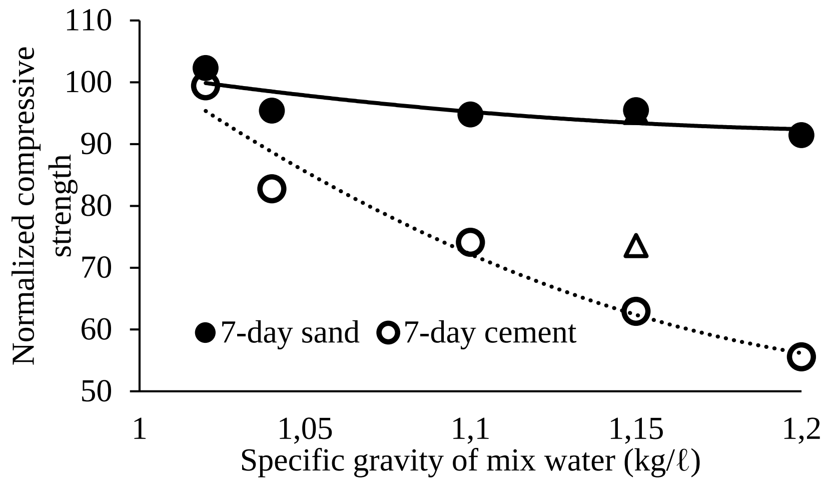


Figure 2: Normalized compressive strength at the age of 7 days

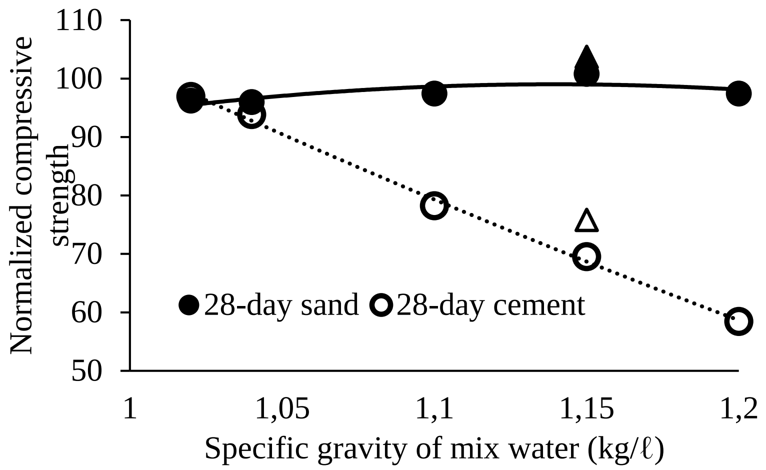


Figure 3: Normalized compressive strength at the age of 28 days

Figure 1 shows that in case of sand reduction from the mix, increasing specific gravity of the mixing water increased the early age (3-day) strength at all replacement levels since the solid particles in wash water were partially hydrated cementitious materials, for this reason, accelerated the hydration reaction. The relative reduction of the 3-day compressive strength after a certain level of specific gravity of mixing water can be attributed to the insufficient compaction of concrete due to loss of workability. The triangle points in Figures 1-3, indicating the compressive strength of workability modified concretes (1.15MC and 1.15MS), show that sufficient compaction reduced the negative effect of wash water density on the compressive strength of concrete.

As defined above, replacing the solid particles in wash water with cement by volume increased the W/C. Nevertheless, particularly 3-day compressive strengths were not influenced by the raise of W/C because of acceleration of hydration if the compaction is sufficient. The unfilled triangle in Figure 1 displays the contribution of increasing amount of solid particles in mixing water even though W/C increases.

The effect of W/C elevation on the compressive strength was noticeable for 7 and 28-day age strengths. The significant continuous decline of compressive strength with the increasing specific gravity of mixing water for cement reduced mixtures can be seen in Figures 2 and 3. Modification of the workability with additional HRWR admixture did not change this trend. Compressive strength of the sand reduced mixtures were not affected remarkably and remained almost stable.

3.2 Sorptivity

Figure 4 shows the sorptivities of all the concretes prepared in this study. The solid and dotted curves represent the sorptivity of cement- and natural sand-reduced concretes, respectively. Sorptivities of cement-reduced concrete mixtures prepared with recycled washout water were higher than that of control mixture at all density levels. This can also be ascribed to the increasing W/C because of replacing solid particles in wash water for cement by volume.

Sand reduced concretes prepared with diluted wash water having specific gravities of 1.02 and 1.04 kg/L exhibited dramatic fall of sorptivity values. This may be due to the particle packing of fines. Increasing specific gravity of mixing water dissipated this tendency and sorptivities of concretes prepared with wash water having specific gravity more than 1.10 kg/ℓ showed equal or higher sorptivities.

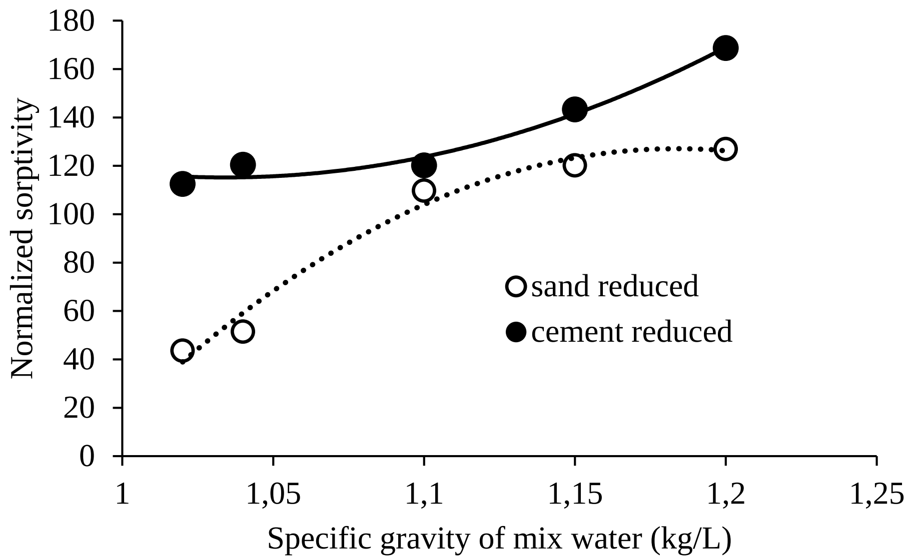


Figure 4: Normalized sorptivity of concretes at the age of 28 days

4. CONCLUSIONS

The following conclusions can be drawn from the test results:

- Employment of solid particles in the wash water in the place of a part of cement caused reduction of compressive strength at all ages due to the increase in the water/cement ratio.
- Usage of solid particles in wash water in the place of a part of natural sand increased the early strength (3-day) and did not decrease the compressive strength significantly at 7 and 28-day age. Besides recycling waste wash water, rinsing water of truck mixers can be utilized for high early strength required conditions.
- Sand-reduced mixtures showed lower sorptivities than the control mix, up to 1.10 kg/ℓ specific gravity, indicating the positive effect of fine material in the mixture.
- Partially hydrated cement particles exist in the wash water cannot take the place of binder but can supersede natural sand, with respect to compressive strength and sorptivity.
- Owing to the decreasing stock of natural sand for the demand of concrete industry, recycling washout water will ensure sustainable development in construction technology.

REFERENCES

[1] Ekolu S.O. and Dawneerangen, A., ‘Evaluation of recycled water recovered from a ready-mix concrete plant for reuse in concrete’, Journal of The South African Institution of Civil Engineering. 52, (2010) 77-82.

[2] Muszynski, L. and Chini, A., ‘Re-using wash water in ready-mixed concrete operations’, Concrete. 36, (2002), 16-18.

[3] Guidance on Concrete Wash Water in the European Ready Mixed Concrete Industry, European Ready Mixed Concrete Organization ERMCO, October 2006.

[4] Su, N., Miao, B. and Liu, F. ‘Effect of wash water and underground water on properties of concrete’, Cement and Concrete Research. 32 (2002), 777-782.

[5] Neuwald, A. D. 'Treating and Recycling Concrete Process Water', National Precast Concrete Association Archive - 2004-2008, (2010).
[6] Sandrolini, F. and Franzoni, E. 'Waste wash water recycling in ready-mixed concrete plants', Cement and Concrete Research, 31, (2001), 485-489.
[7] Borger, J., Carrasquillo, R. L. and Fowler D. W., 'Use of Recycled Wash Water and Returned Plastic Concrete in the Production of Fresh Concrete', Advn. Cem. Bas. Mat., 1, (1994) 267-274.
[8] Wasserman, B., 'Wash Water in the Mix: Effects on the Compressive Strength of Concrete', International Journal of Construction Education and Research, 8 (4) (2012) 301-316.
[9] Chatveera, B. and Lertwattanaruk, P. 'Use of ready-mixed concrete plant sludge water in concrete containing an additive or admixture', Journal of Environmental Management. 90, (2009) 1901-1908.
[10] Lobo, C., and Mullings, G.M., 'Recycled Water in Ready Mixed Concrete Operations', Concrete InFocus, National Ready Mixed Concrete Association, Spring (2003) 17-26, http://www.nrmca.org/research/33%20CIF%2003-1%20wash%20water.pdf
[11] Ferriz-Papi, J. A., 'Recycling of fresh concrete exceeding and wash water in concrete mixing plants', Materiales de Construccion, 64 (313) (2014)

EFFECTS OF RECYCLED AGGREGATE RATIO ON PROPERTIES OF FRESH CONCRETE

Yeşim Tosun (1) and Remzi Şahin (1)

(1) Atatürk University, Eng. Fac., Dept. of Civil Eng., Erzurum, Turkey

Abstract

Building demolition wastes arise greatly in due to urban renewal projects launched in various locations in Turkey. In this study, 0%, 15%, 30%, 45% and 60% of the normal (natural) coarse aggregate was replaced by the recycled aggregate. Maximum aggregate particle sizes were selected as 16 mm, 22,4 mm and 31,5 mm; and 0,06%, 0,13% and 0,20% of air-entraining admixture (AEA) were entrained into mixtures. Fly ash and superplasticizer were used as a mineral and chemical admixture, respectively. The same type (CEM I 42.5) and constant dosage (350 kg/m^3) of cement were used in the study. Water/cement ratio was kept constant 0.53 for all mixtures. Fresh unit weight, slump and air content tests were carried out in the experiments as fresh concrete properties. The analysis of the test results leads to the conclusion that increase in maximum particle size and a RCA ratio of 60% caused a decrease in slump value while the increase in the ratio of air-entraining admixture contributed to a rise in slump. On the other hand, air content with quite high values has been obtained due to use of RCA and air entraining admixtures together.

Keywords: Recycled aggregate, slump, fresh unit weight

1. INTRODUCTION

Construction materials are increasingly judged by their ecological characteristics. Concrete recycling is important for sustainable development since it is one of the best alternatives to save materials in respect of protection of natural resources. After removal of contaminants through selective demolition, screening and size reduction in a crusher to aggregate sizes, crushed concrete can be used as recycled aggregates for concrete industry.

Every year, 8-12 billion tons of natural aggregate, the most important raw materials for construction sector, are consumed in the world [1]. Concrete wastes constitute about 90% of building wastes. Heavy matrix of concrete makes it an ideal recycled material which may be used as it is, or with little strength or performance loss. However, since a sufficient awareness was not created regarding this issue yet, Recycled Concrete Aggregate (RCA) is of quite limited use and it is typically used in areas which do not retain aggregates [2]. In the United States, each year 200 million tons of concrete pavement is recycled and recycled concrete aggregate is used by law enforcement in 44 states, this is not applicable in Turkey though [3].Each year 125 million tons of excavation waste is disposed in Turkey. However, this amount is expected to grow substantially together with the introduction of urban renewal works. With the introduction of Law numbered 6306, it was stated that each year 10 million tons of construction waste would be produced in total during the first 3 years and the total amount of material to be recycled would be 6 million tons on an annual basis [4]. This study reviews the effect of the ratio of RCA replacing the normal (natural) coarse aggregate, maximum aggregate particle size and the amount of entrained air on the properties of fresh concrete.

2. MATERIAL AND METHOD

Normal Portland Cement (CEM I 42,5R), calcareous crushed stone as normal (natural) coarse aggregate, river sand as thin aggregate, limestone dust with a particle size of 0-0.125 mm as filler material have been used in tests. Grade F fly ash as mineral admixture, modified synthetic carboxylate polymer based next generation superplasticizer concrete admixture (specific gravity 1,085 g/cm^3) and air entraining concrete admixture (specific gravity 1,010 g/cm^3) as chemical admixture have been used. Aggregates with a particle size of 4-31.5 mm obtained from the recycling plant of Istanbul Metropolitan Municipality Environmental Protection and Waste Material Valuation Industry and Trade Co. (İSTAÇ) have been used as RCA. Table 1 shows the physical and mechanical characteristics of cement and fly ash used in the study and Table 2 shows the physical characteristics of aggregates. Freeze-thaw tests of the aggregates were done according to TS EN 1367-1 [5].

Water/cement ratio of 0,53, binding dosage of 350 kg/m^3, fly ash ratio of 15% (substituted by the cement) superplasticizer ratio of 1,5% were kept constant in the compositions. However, RCA ratio has been reviewed at five (0%, 15%, 30%, 45% and 60%), maximum aggregate particle size at three (16mm, 22.4 mm and 31.5 mm) and amount of entrained air again at three (0.06%, 0.13% and 0.20%) different levels. Mix design which takes these parameters and levels into consideration is given in Ref. [6].

Unit weight, slump and air content test among the fresh concrete tests the operating principles of which were defined in related Turkish Standards have been conducted.

Table 1: The chemical composition and physical characteristics of the cement and fly ash

	Cement (CEM I 42,5R)				Fly ash			
Chemical composition (%)	SiO_2	18,72	SO_3	2,98	SiO_2	53,69	SO_3	0,99
	Al_2O_3	4,54	Na_2O	0,19	Al_2O_3	20,29	Na_2O	-
	Fe_2O_3	3,43	K_2O	0,68	Fe_2O_3	11,83	K_2O	2,53
	CaO	62,25	Cl	0,0098	CaO	3,40	Cl	-
	MgO	3,34	Insol. residue	0,95	MgO	4,09	Loss of ign.	2,01
Physical characteristics	Blaine (cm^2 /gr)		3812		4020			
	Specific Gravity		3,13		1,98			
	Comp. Strength (MPa)		57,9		-			

Table 2: Physical characteristics of aggregates

	Particle density(g/cm^3)	Water Abs. (%)	Abrasion Rate (%)	Freeze-Thaw Mass Loss (%)
Crushed Stone	2.68	0.96	15.4	0.95
RCA	2.37	6.94	38.04	13.75
Fine Aggregate	2.44	4.99	-	-

3. FINDINGS AND EVALUATION

3.1 Evaluation of slump test results

Slump values for all series of cement produced in the study have been found to be grade S4 and S5 (16-21 cm and ≥22 cm) in accordance with TS EN 206-1[7]. The graphic showing the variation of average slump values by the parameters is shown in Figure 1.

It is seen from the Figure 1 that slump value decreases with the increase in maximum aggregate particle size within the concrete. RCA addition to the concrete mixes has increased the slump up to 30%, caused it to be fixed between 30%-45% and caused the slump to decrease above this ratio (60%). The increase in the ratio of air entrainer in the mixes have contributed to an increase in the concrete slump. These findings are compliant with the literature given in Refs.[8-13]

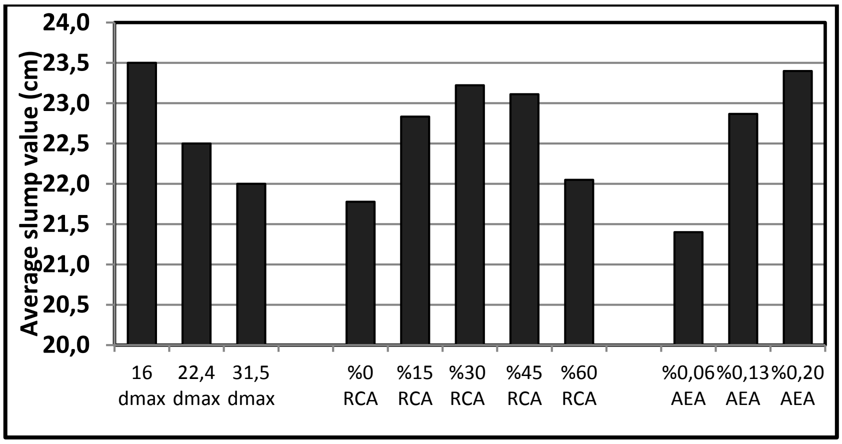


Figure 1. Average slump values by parameters

3.2 Evaluation of unit weight test results

The average unit weights of the fresh concrete are shown in Figure 2. As it is shown in Fig. 2, the increases in the maximum particle sizes of the mixes have contributed to a decrease in unit weight. In the study, the highest unit weight values (average 2,13 gr/cm^3) have been obtained with the mixes without RCA. Unit weights of concrete have been found to decrease significantly after the addition of RCA. But, while unit weights continued to decrease in parallel to the increase in RCA ratio, it was found interesting that unit weights of mixes containing RCA have increased slightly. Air entraining admixture had also the same tendency as RCA, but the density of fresh concrete decreased in proportion to the increase in the amount of this admixture. However, this decrease has not been observed with the mixes containing AEA with a ratio from 0.13% to 0.20%.

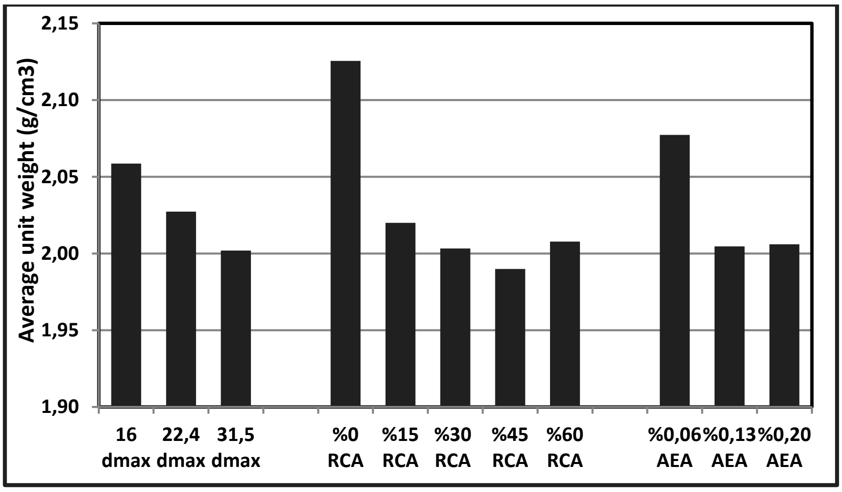


Figure 2: Average unit weight values by parameters

3.3 Evaluation of air content test results

Results relating to the average air contents of the mixes are given i Figure 3. As it is shown in the graphic in Figure 3, the air content of fresh concrete have increased in parallel to the increase in maximum aggregate particle size. Relatively less air content value in mixes without

RCA has increased significantly with the introduction of RCA with ratios of 15%, 30% and 45% in mixes, however any difference has not been observed among these three RCA ratios. It was found interesting that air content of the mixes containing a RCA of 60% have decreased slightly. As it was expected, the increase in the ratio of air entraining admixture have contributed to a rise in air content of concrete. However, it is another case found interesting in this study that concrete with an addition of air entraining admixture by 0.20% had almost the same air content with the mixes containing an admixture of 0.13%. By nature, air entraining admixture added to the concrete contributes to an increase in the air content of the concrete, but RCA contributing to an increase in the air content of concrete can be said to be caused by their high porosity.

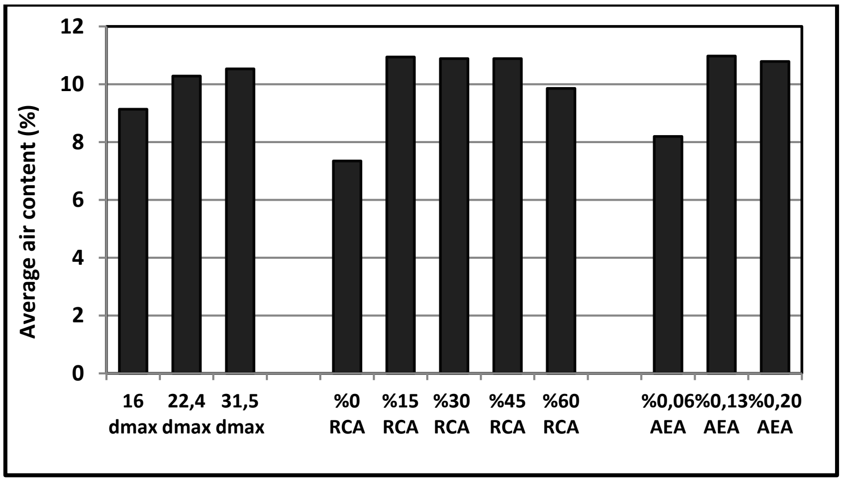


Figure 3: Average air content values by parameters

4. CONCLUSIONS

1. The increase in maximum particle size and a RCA ratio of 60% caused a decrease in slump value while the increase in the ratio of air entraining admixture contributed to a rise in slump.
2. The densities of the sample concrete produced have been found to be less than those of normal concrete due to the less density of RCAs and the use of air entraining admixture. The increase in maximum particle size, RCA ratio and air entraining admixture have contributed to a decrease in concrete density.
3. Air content with quite high values has been obtained due to use of RCA and air entraining admixtures together. The increase in maximum particle size, RCA ratio and the amount of air entraining admixture have contributed to a rise in air content.

REFERENCES

[1] Köken A., Köroğlu, M., A., Yonar., F., Atık Betonların Beton Agregası Olarak Kullanılabilirliği, *Teknik-Online Dergi*, 7 (2008).

[2] Zhao, J., L., Yu, T., Teng, J., G., Stress-Strain Behavior of FRP-Confined Recycled Aggregate, *J. of Com. for Cons.*, ASCE, (2014)

[3] Gürer, C., Akbulut, H., Kürklü, G., 2004. İnşaat Endüstrisinde Geri Dönüşüm, 5 Endüstriyel Hammaddeler Sempozyumu, İzmir.
[4] Kılıç, N., Kentsel Dönüşümde Geri Dönüşüm Atağı. İzmir Ticaret Odası, *Ar-Ge Bülten,* (2012).
[5] TS EN 1367-1, 2009, Türk Standartları Enstitüsü, Ankara.
[6] Tosun, Y., 2014. Geri Kazanılmış Agregalı Betonların CDF Yöntemi İle Donma-Çözülme (Yüzeysel Kabuk Atma) Dirençlerinin Belirlenmesi, YL Tezi, Atatürk Üniversitesi, Erzurum, TR
[7] TS EN 206 (2014), Türk Standartları Enstitüsü, Ankara.
[8] Evangelista, L., de Brito, J., 2006. Mec. behaviour of concrete made with fine recycled concrete aggregates. C&C Com. 29, 397–401
[9] Kou, S., 2006, Reusıng Recycled Aggregates In Structural Concrete, Ph.D. Thesis, The Hong Kong Polytechnıc Unıv.
[10] [Movassaghi, R., 2006, Durability of Reinforced Concrete Incorporating Recycled Concrete as Aggregate, Ph.D, U. of Waterloo.
[11] Abbas, A., A., 2007. Durability of Green Concrete as a Structural Material, University of Ottawa, Canada
[12] Durmuş, G., Şimşek, O., Dayı, M., 2009. Geri Dönüşümlü İri Agregaların Beton Özelliklerine Etkisi. Gazi Ü. Der. 24(1), 183-189.
[13] Behera, M., Bhattacharyya, S.K., Minocha, A.K., Deoliya, R., Maiti, S., 2014, Recycled aggregate from C&D waste & its use in concrete, Cons. and Build. Mat. 68, 501–516.

USE OF RECOVERED AGGREGATES AND RECYCLED MATERIALS IN JIS A 5308:2014-READY-MIXED CONCRETE

Yukikazu Tsuji (1) and Yasuji Itou (2)

(1) Ex-President, Maebashi Institute of Technology, Professor Emeritus Gunma Univ., Japan

(2) Technical Director, Japanese National Federation of Ready-Mixed Concrete, Japan

Abstract

This paper describes a framework that allows producers to use recovered aggregate collected from the newly defined “returned concrete” without consulting purchasers, and it adds ground granulated blast-furnace slag and silica fume to the environmental label of recycled materials newly established in the 2011 Amendment (revision) including fly ash, slag aggregates, recycled aggregate type-H, eco-cement and recycled water, to promote the reduction of environmental loads. Furthermore, this paper mentions also matters that were issues during deliberations, namely advanced use of sludge water with solids content greater than 3%, traceability and international harmonization with the ISO standards.

Keywords: Recovered aggregate, returned concrete, recycled materials, environmental label, Japanese Industrial Standards (JIS) A 5308 “Ready-Mixed Concrete” 2014

1. INTRODUCTION

Following the 12th revision of Japanese Industrial Standards (JIS) A 5308 “Ready-Mixed Concrete”2011 Amendment on 20 December 2011, the 13th revision was notified on 20 March 2014. The major amendments of the last revision include the introduction of the Environmental Label with Supplements and the standardization of recovered aggregate, for which this paper will give explanation with regard to the 2011 Amendment (revision).

2. THE ENVIRONMENTAL LABEL

Towards the establishment of recycling society, the JIS A 5308 has been updated by standardizing the applicability of the Eco-cement, various slag aggregates, and other recycled materials such as recycled water. On the other hand, some customers are not positive in using ready-mixed concretes with recycled materials. Hence it is necessary to appeal, not only to customers but also to the public, the significance of the use of recycled materials in ready-mixed concrete for the reduction of the environmental burden. This has led to the introduction of the Environmental Labels, which was standardized as JIS Q 14021, “Environmental labels and declaration – Self-declared environmental claims (Type II environmental labeling)”.

In the same way as precast concrete products, Type II labels are to be used to self-declare the items related to products, services and compatibility to the environmental considerations, and can be attached to the proportioning plan of ready-mixed concrete. Also, as a note of the clause 12.2 of delivery ticket for ready-mixed concrete, the following statements are to be added: “When manufactures use recycled materials as shown in Table -10A (Table 1 of this paper) the manufacturers can display a list of the materials notation and their contents just below the JIS Q 14021 Mobius Loop (Figure 1 of this paper) on the delivery ticket for the ready-mixed concrete. When applied, the manufacturers should supply the control data or test data that can assure the content of the Environmental Label upon customers’ requests. ”

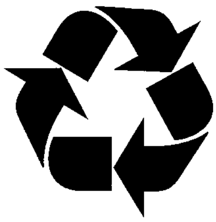
Use the Environmental Labels is at manufacturers’ discretion even if the recycled materials as shown in Table 1 are used.

The recycled materials applicable to the Environmental Label were newly prescribed in the Amendment (revision) of JIS A 5308 on December 2011, where fly ash conforming to JIS A 6201 “Fly ash for use in concrete” was only mentioned among other additions. In the subsequent update of JIS A 5308 on March 2014, ground granulated blast-furnace slag (JIS A 6206 “Ground granulated blast-furnace slag for concrete”) and silica fume (JIS A 6207 “Silica fume for concrete”) were added to the list of the recycled materials as shown in Table 1 because these materials have been specified in JIS and their qualities are assured. The notations are FA I or FA II for fly ash, BF for ground granulated blast-furnace slag and SF for silica fume.

Table 1: Recycled materials

Materials	Notation*	Materials applicable to the Environmental Labels
Eco-cement	E（or EC）	**JIS R 5214**
Blast furnace slag aggregate	BFG or BFS	**JIS A 5011-1**
Ferronickel slag aggregate	FNS	**JIS A 5011-2**
Copper slag aggregate	CUS	**JIS A 5011-3**
Electric arc furnace oxidizing slag aggregate	EFG or EFS	**JIS A 5011-4**
Recycled aggregate -class H	RHG or RHS	**JIS A 5021**
Fly ash	FA I or FA II	**JIS A 6201** type I or II conforming products
Ground granulated blast-furnace slag	BF	**JIS A 6206**
Silica fume	SF	**JIS A 6207**
Supernatant water	RW1	**JIS A 5308 Annex C** conforming recycled water without sludge
Sludge water	RW2	**JIS A 5308 Annex C** conforming recycled water with sludge

*Aggregate with G at the end of notation refers to coarse aggregate and S to fine aggregate.



RHG 30% a)/ RW2 (2.5%) b)/ FAII 10% c)

Figure 1: Notification example of product name and contents

a) Shows that this product contains recycled aggregate H for 30 percent by mass of aggregate.
b) Recycled water was specified in the Annex C as supernatant water RW1 and sludge water RW2. As shown in this notation example, sludge water with a targeted solid content of 2.5% shall be noted in parenthesis, while in case of supernatant water with an usage of 100% shall be noted also in parenthesis.
c) Usage of addition shall be noted in percent by mass of binder.

Table 2: Control methods for recovered aggregate

Method	**Storage**	**Targeted replacement rate**	**Control duration**	**Control items**	**Items to be noted in proportioning plan**	**Items to be noted in delivery ticket**
A	Storage after mixing with primary aggregate	Less than 5%	One day or number of days when shipment reaches $100m^3$	Amount of powder content is less than that of primary aggregate	Fill as A	Fill as less than 5%
B	Dedicated storage	Less than 20%	Each one batch		Fill as B	Fill replacement rate obtained from aggregate unit content by mix

3. STANDARDIZATION OF RECOVERED AGGREGATE IN JIS A 5308

For a further reduction of environmental burden, efficient use of recovered aggregate is important. Coarse and fine aggregates recovered from returned ready-mixed concrete have been defined as “recovered coarse aggregate” and “recovered fine aggregate” and specified in JIS A 5308 allowing manufacturer for using recovered aggregates without permission of the customers. Examples of recovered coarse and fine aggregates are shown in Figure 2.

a) Recovered coarse aggregate

b) Recovered fine aggregate

Figure 2: Examples of recovered aggregate

Recovered aggregate is obtained from ready-mixed concrete plants and returned concrete, where the remnants of fresh concrete in agitators, mixers and hoppers is washed with tap water or recycled water and classified into coarse and fine aggregates. Returned concrete refers to a fresh concrete returned to the plant for several reasons, including changes in customers’ conditions, inconformity to customers’ quality requirements, residues after discharge and adherent to agitator drums. Use of recovered aggregate with a replacement rate less than 5% of primary aggregate for both coarse and fine aggregates is called the Method A, while at ready-mixed concrete plants, where dedicated storage bins, carriers and weighing instruments are available, amount of recovered aggregate can be controlled per batch hence the upper limit of replacement rate can be allowed up to 20% in the Method B.

3.1 Control system of recovered aggregate

Control methods for recovered aggregate are shown in Table 2. Recovered aggregates are obtained from ordinary concrete, high-strength concrete or road concrete, and can be used when its powder content passing 75 μm sieve according to JIS A 1103 is not more than that of the primary aggregate. This is because fine particles smaller than 75μm sieve could result in an increase in water demand and a decrease in durability of hardened concrete. Also, recovered aggregate from normal aggregate with significantly different grain size, lightweight and heavyweight aggregate with significantly different densities and those from fresh concrete containing recycled aggregates cannot be used.

Recovered aggregate can be used for normal concrete and road concrete but not for high-strength concrete and lightweight concrete.

When the Method A or B is applied, it is required to control the amount of recovered aggregate to keep the replacement rate less than 5% or 20% of the primary aggregate respectively. Recovered aggregate can be added during transportation of primary aggregate with conveyer belt (Conveyer belt supply) or with a shovel at hopper immediately before aggregate is introduced to conveyer belt (Hopper supply).

(1) Conveyer belt supply

During transportation of primary aggregate with a conveyer belt to the head bin, recovered aggregate is supplied at a constant rate from the dedicated supplementary hopper to the top of primary aggregate at a replacement rate of 5%. The rate of recovered aggregate supply using dedicated supplementary hopper can be made by volume with gate opening control of the hopper or with speed control of the conveyer as shown in Figure 3. A continuous weighing instrument (belt-scale) can also be used.

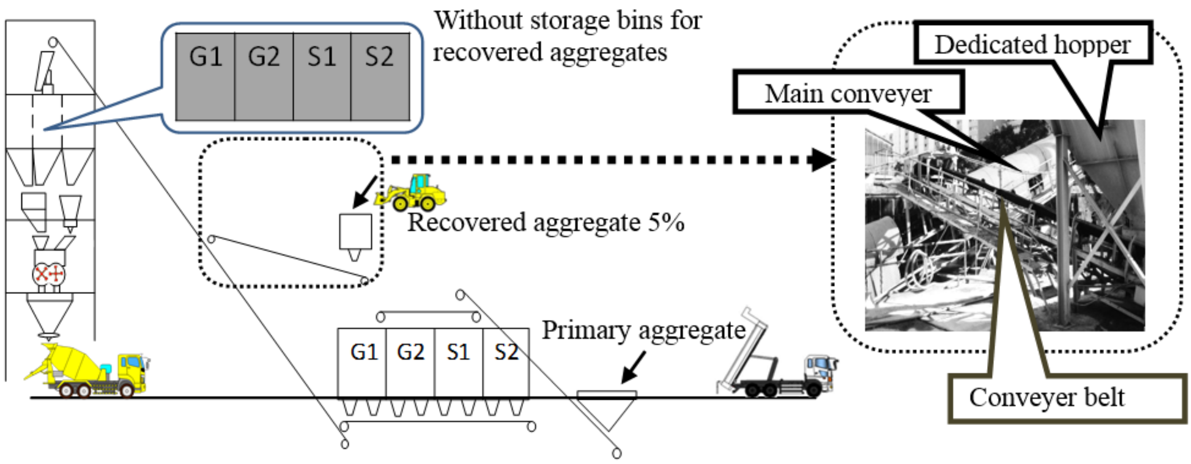


Figure 3: Conveyer belt supply procedure (Method A)

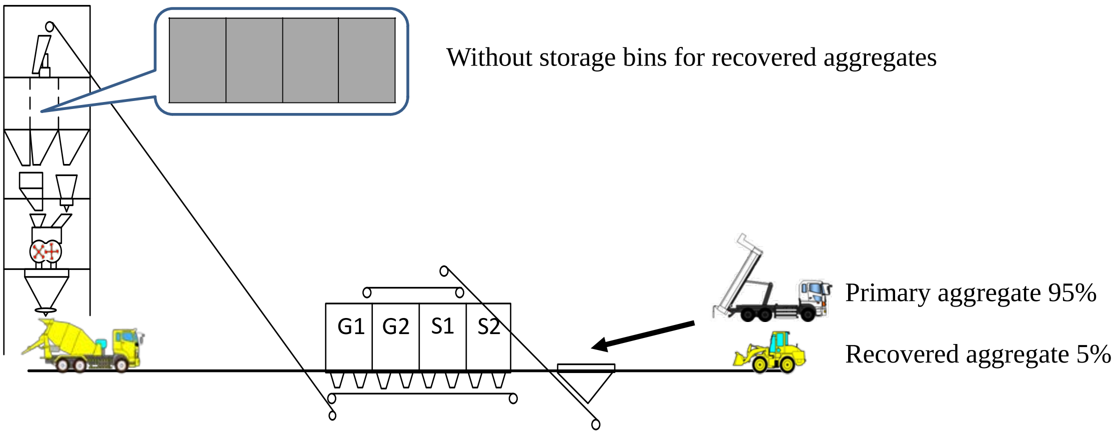


Figure 4: Hopper supply procedure (Method A)

(2) Hopper supply

During transportation of primary aggregate with a conveyer belt via hopper to the storage bin, recovered aggregate is supplied, with a shovel for example, to the hopper at each introduction of primary aggregate to have a replacement rate of 5% as shown in Figure 4.

Amount of recovered aggregate necessary for an amount of one dump truck of primary aggregate may be previously determined with a truck scale and a shovel up amount of a power shovel.

There was a fear whether the above two methods could mix primary and recovered aggregates uniformly at a replacement rate of less than 5%. However, it was confirmed through laboratory tests (Figure 5) that the primary and recovered aggregates could be mixed owing to an overturning effect developed as aggregates pass the hopper as shown in Figure 6. It is clearly shown that mixing of aggregate was insufficient at the first passage while mixing became more uniform as the number of passages or overturns increases.

For ready-mixed concrete plants with dedicated storage bin, carriers and weighing instruments, the upper limit of replacement rate can be allowed up to 20% as the Method B (Figure 7). In such a case, weighing of recovered aggregate can be made cumulatively with that of primary aggregate.

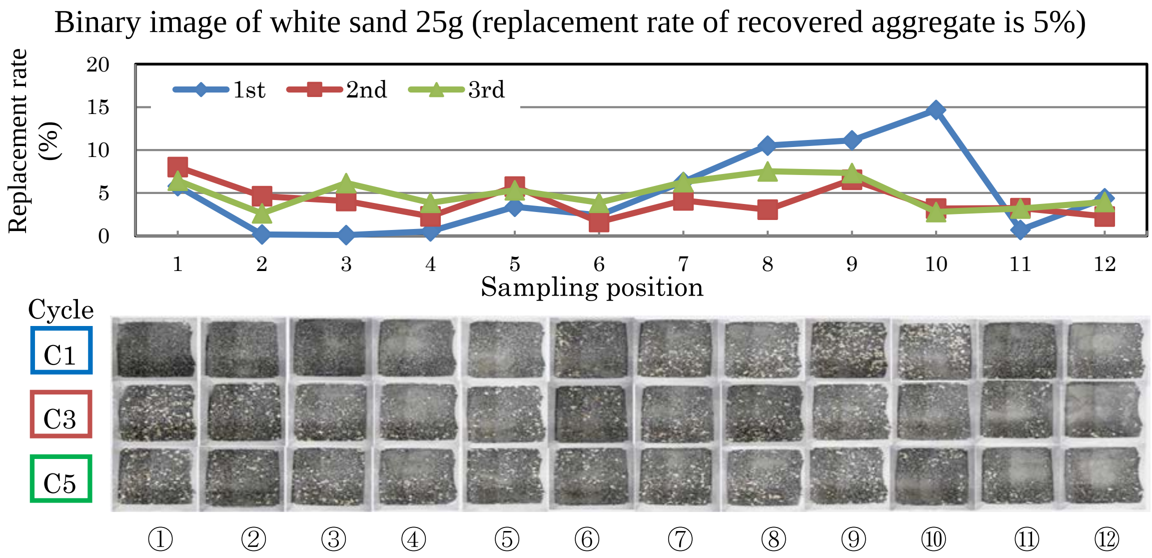


Figure 5: Binary image of recovered aggregate in white at a replacement rate of 5%

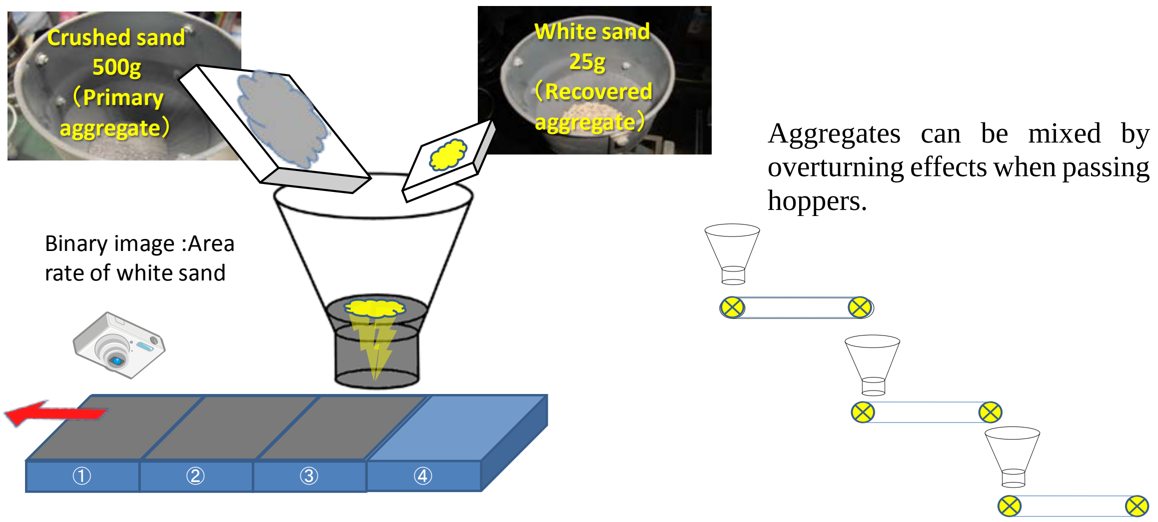


Figure 6: Mixing of aggregate by overturning effects

3.2 Duration and record of recovered aggregate control

Use of recovered aggregate in the Method A requires control of the replacement rate and its record in a control duration of one day. When the amount of shipping of ready-mixed concrete is less than 100m^3, the control duration may be extended until the amount of shipping reaches approx. 100m^3, which has been estimated as the average amount of shipping per plant referring to the annual amount of shipping and the average work day of ready mixed concrete plants.

The amount of recovered aggregate processed with dedicated storage, transportation and weighing instruments shall be controlled by batch and recorded.

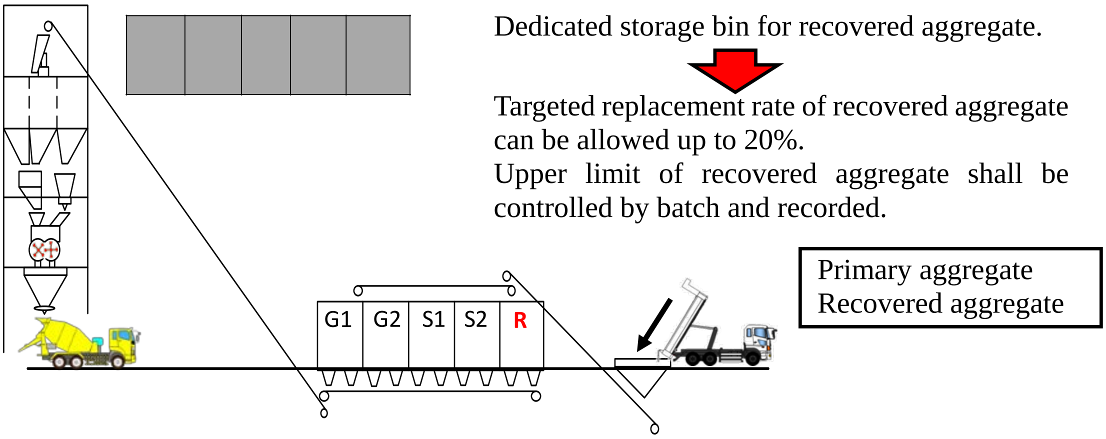


Figure 7: Case of using dedicated storage equipment (Method B)

3.3 Indication of the usage of recovered aggregate on the delivery ticket

Indication of the use of recovered aggregate with a replacement rate less than 5% shall be made by filling the relevant space in Table 10 for recovered aggregate usage of the proportioning plan specified in the clause 12.1 by "Method A", and in the case of ready-mixed concrete plants with dedicated storage bin, carriers and weighing instruments, it shall be indicated by filling the relevant space of Table 10 for recovered aggregate usage by "Method B".

In Table 11 specified in the clause 12.2 of the regulation concerning the delivery ticket of ready-mixed concrete, the space for replacement rate of "less than 5%" recovered aggregate shall be filled for Method A, and for Method B, the replacement rate obtained for unit amount of aggregate for each batch shall be recorded.

Above standardization into JIS A 5308 became possible because the properties of concretes with recovered and normal (primary) aggregates were found to be equivalent as shown in Figure 8. Examples of comparative test results are shown using aggregates sampled from three ready-mixed concrete plants. Recovered coarse and fine aggregate contents of the aggregates were varied and the properties were compared with those of the control concrete [1,2].

It was also confirmed that use of 100% recovered aggregate both coarse and fine aggregate fractions did not take effect any decrease in the properties. However, the concrete with recovered coarse and fine aggregate replacement rate of 20% showed better workability than that with 100% replacement. It was, therefore, confirmed that the limiting replacement rate for the use of recovered aggregate could be as high as 20% if both the workability and strength are considered.

4. CONCLUDING REMARKS

A development and survey project for the JIS standardization of environmental considerations and traceability upgrade of ready-mixed concrete had been in progress for three years since 2010 by the technical committee established in the Japanese National Federation of Ready-Mixed Concrete responsible for the research and drafting of the revision of the JIS A 5308. The draft of the supplements (revision) for the JIS A 5308 is an outcome of the above committee work in 2010.

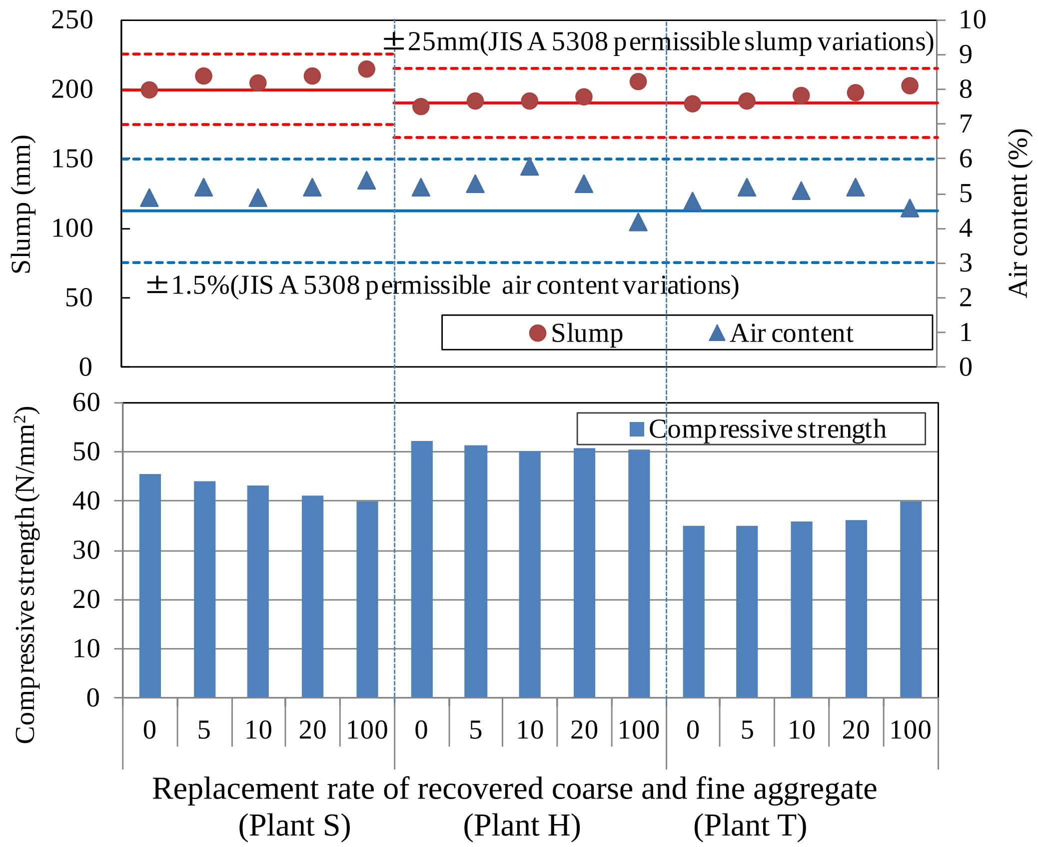


Figure 8: Comparison of concretes properties between recovered aggregate concrete and control concrete

As for the second year, activities of 2011 included international harmonization with the ISO standards, environmental issue (recovered aggregate and effective use of sludge water) and traceability upgrade and the associated survey and confirmation experiments were carried out. In the third year, the committee established a new sub-committee for drafting the JIS revision, and, executing the successive experiments and surveys, the committee completed the draft manuscript.

Among other items of the revised JIS A 5308 on March 2014, mainly, introduction of the Environmental Labels for the use of recycled materials and standardization of recovered aggregate are explained in this paper with reference to the previous Amendment (revision) on December 2011.

REFERENCES

[1] K., Yamanouchi, K. and Ito, Y. ”Recycling of wastes from ready-mixed concrete plants”, Preprints of the JSCE Annual Meeting, Section 5, 67 (2012) 1091-1092. (in Japanese)

[2] Tsujimoto, K. and Ito, Y. “Towards the effective use of recovered aggregate: Survey of concrete quality with recovered aggregates”, Cement and Concrete, 804 (2014) 29-36. (in Japanese)

A NEW SYSTEM ENABLING COMPLETE USE OF RECYCLED WATER IN BATCHING PLANTS

Mustafa Şen (1), M. Çağrı Şen (1), Ayla Şalap (1), Ender Derin (2) and Çoskun Sarıkayaoğlu (3)

(1) Özbekoğlu İth. İhr. İnş. Taah. ve Müh. Ltd. Şti., Turkey

(2) Modern Beton San. Ve Tic. A.Ş., Turkey

(3) Vur-Mak Vuruşkan Mak. San. Tic. A.Ş, Turkey

Abstract

The use of recycled water in concrete batching plants is one of the key factors for sustainable concrete production[1].

Current methods for determining the density of grey water are neither precise nor real-time, therefore the use of recycled water is not at desired level.

The new system presented in this paper is able to scientifically measure the density of grey water, prepare a mixture of grey water and fresh water at desired density and send this mixture to the batching plant to be used as batch water.

Thereby 100% usage of recycled water is made possible.

Keywords: Concrete, batching plant, recycled water, sustainable concrete, water mixture, EN - 1008, environmentally friendly, quality of concrete, density measurement

1. INTRODUCTION

The re-use of waste water (grey water or process water) accumulated during concrete production is not at a sufficient level[3].

Appendix-A of EN-1008 contains information on ratios and conditions about the usage of grey water in concrete production[5]. However, known applications are not able to measure the density in real-time and cannot utilize this value via software, so problems are experienced in sustaining concrete quality due to variations on density values[4].

If the density of waste water could be scientifically measured in real-time, mixed homogeneously with fresh water to bring the mixture to the desired density level and used in concrete production; the waste water in concrete batching plants will be 100% recycled and a huge contribution to sustainable concrete production will be achieved.

2. OBJECT

The object is to introduce a new system for using recycled water as batch water in concrete production.

The system prepares and supplies mixed water at required density on real-time measurement basis.

3. PRESENT METHODS

Presently the recycled water is diluted by fresh water and this diluted mixture is used as batch water.

Conventional method is to use this mixture at a ratio from %10 to %40 of the water requirement for the batch[2].

The ratio of use is based on the density of the mixture and producers experiences.

4. DENSITY MEASUREMENT

The most popular methods to measure density are

- Weighing a defined volume of the mixture. (See Figure 1)
- Using a density-meter. (See Figure 2)

However, both methods have week points. The weighing method is limited to the precision of user.

Figure 1: Density measurement by weighing

The density meter measures the density of the clean section of the sample because the grains settle at the bottom of the cup in less than a second.

Figure 2: Density measurement by density meter

Most important point is that both methods are not real-time and a good sampling is very difficult to achieve. It needs time, therefore cannot be exactly the same as what is used in production at that very moment.

The ideal method of measurement should be capable of measuring density with homogenized distribution of particles in water. The sample should be an exact representation of the mixture.

5. CONSEQUENCES OF PRESENT METHODS

EN-1008 instructs to measure density as frequently as possible. In practice, density measurements are performed 3 to 10 times during a shift and especially when the highest density values are reached.

Since the results are not read in real-time and do not represent the mixture as it is;

- The measured data needs to be manipulated by correction factors.
- The worst case scenario is taken into consideration and calculations.
- A lot of safety factors are inserted which increase the cost.

All above mentioned aspects result with increased cost of production but still do not solve the problem, as the uncertainty of sampling and measurement still exist. [2]

As a consequence of above; concrete production is continued with:

- Uncontrolled and non-stabilized quality of water.
- Sometimes high costs are incurred to omit the risk of low quality production.
- Sludge elimination costs are added to operation costs.

6. SUMMARY

- The recycled water must be used in batch water for sustainable concrete production.
- To reach sustainable concrete production in regards of recycled water is a non-achieved target.
- To use the recycled water in an industrial way, the density needs to be defined.

- For density measurement; a proper sampling is practically almost impossible.
- The density measurement with present methods cannot be performed on real-time basis.

7. THE NEW SYSTEM

The density of the recycled water is measured by scientific methods and electronic means. (See Figure 3). This measurement permits the system to operate under below modes;

i. Prepare a mixture at set density and deliver this mixture to batch and related data to PLC.

or

ii. Prepare a mixture and send the density information to the plant PLC during delivery of the mixture to the plant. In which case, the plant PLC makes necessary calculations for the recipe of the batch.

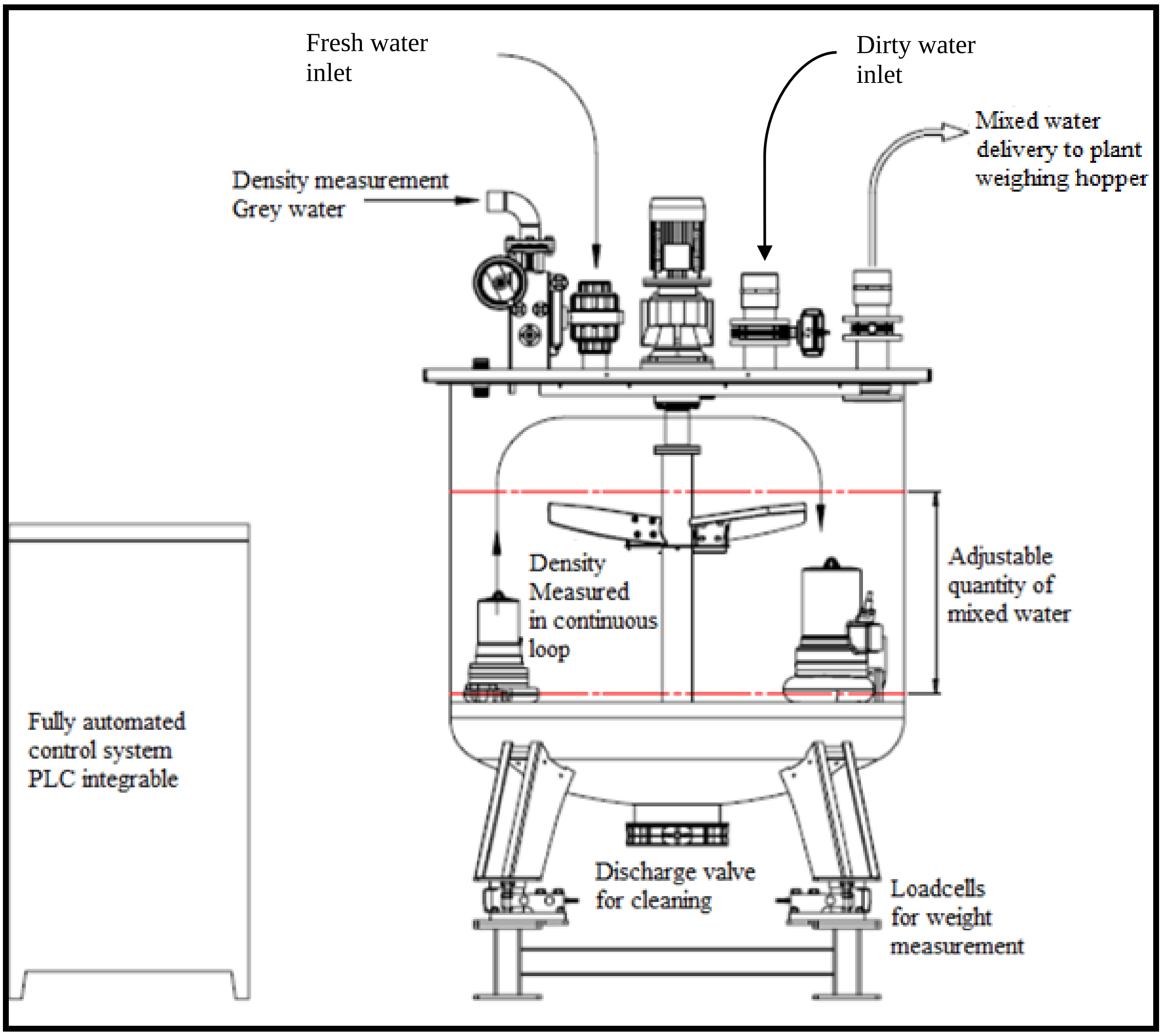


Figure 3: The new system

8. SYSTEM COMPONENTS

- Liquid Contamination Analyser (LCA)
- Electronic processor.
- Tank with agitator, pumps and discharge valve.
- Electronic weighing system with loadcells.
- Recycled water inlet.
- Fresh water inlet.
- Mixed water outlet.
- Conveying pump.
- PLC and control board.
- Cleaning system for LCA probes.

Controls are made by weight analysis under normal operating conditions. With proper calibration, the system measurements of density are precise.

9. THE PERFORMANCE GRAPHS

The accuracy and the measurements achieved from the system are shown in Figure 4 and Figure 5.

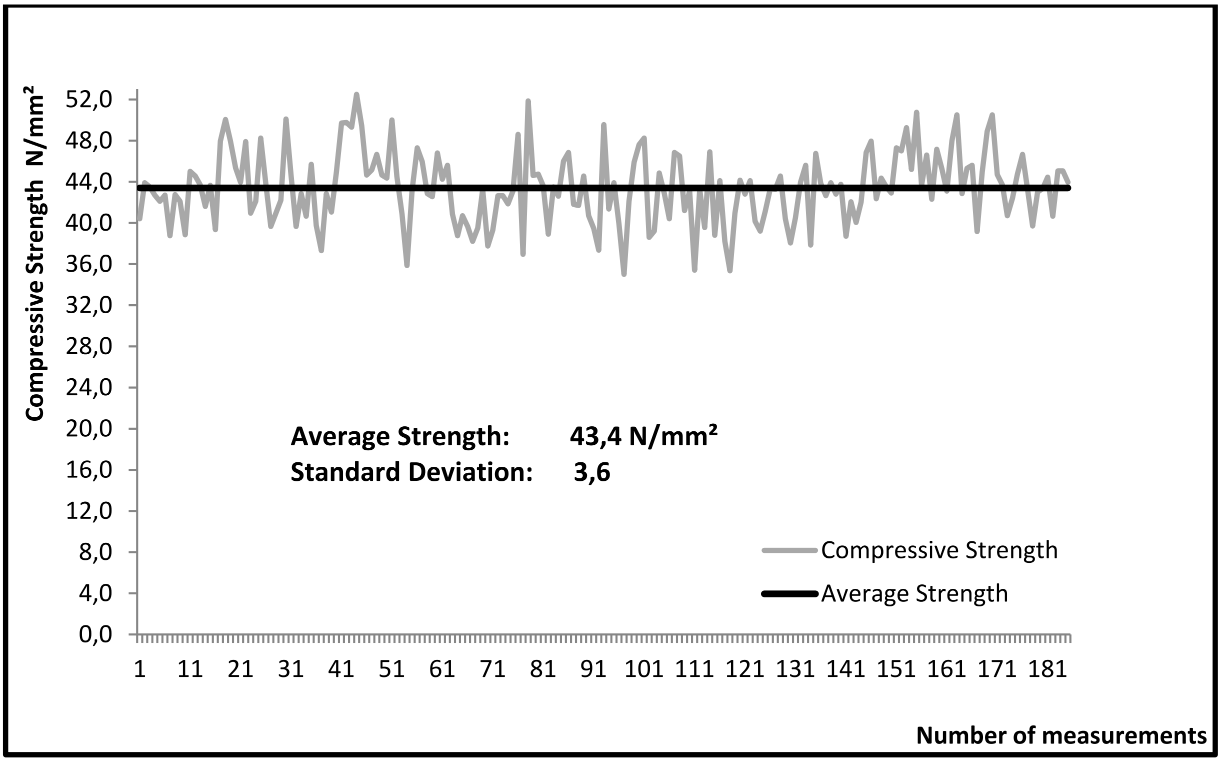


Figure 4: Compressive strength analysis of C30-C37 Concrete with presently used methods)

As seen on Figure 5, the system prepares the mixture at the set value and no significant deviation from the required target values.

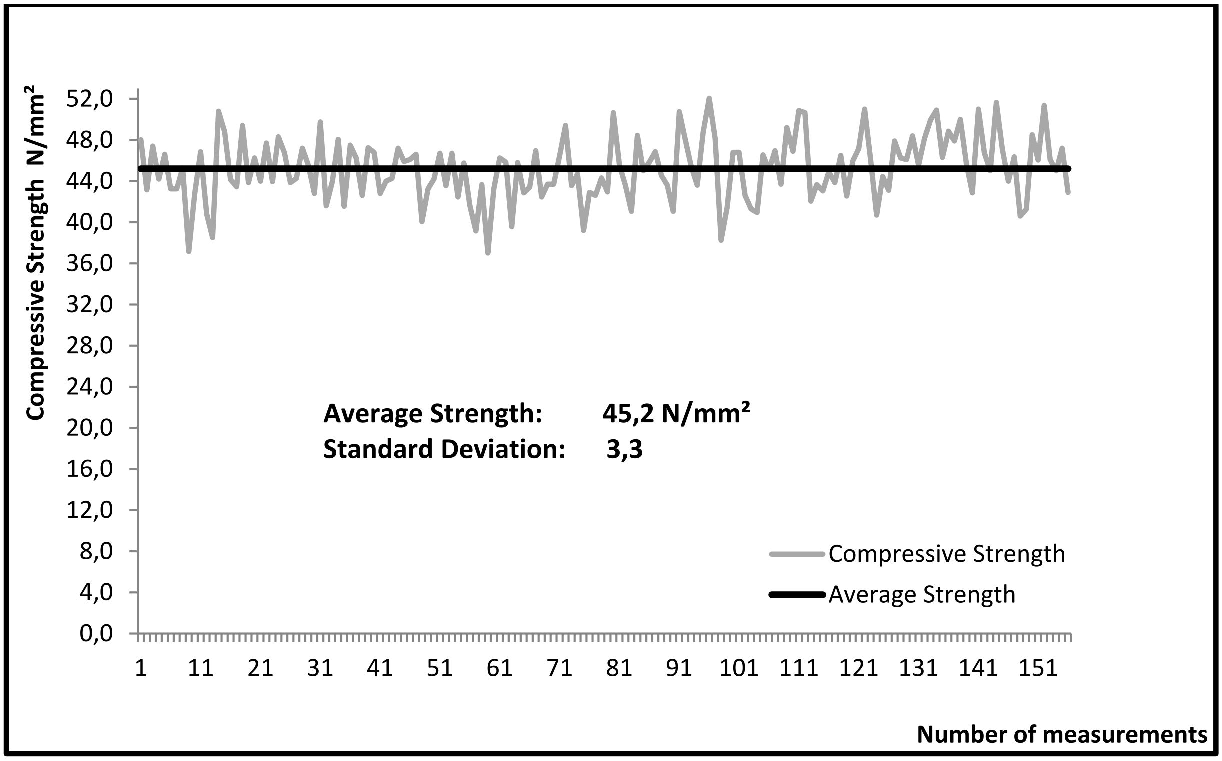


Figure 5: Compressive strength analysis of C30-C37 Concrete with CLR-S

The stability of slump and strength of concrete depends on various factors, such as; cement- water ratio, aggregate specifications and weather conditions, stability of raw materials quality.

Therefore, defining and setting the grey water density alone cannot stabilize slump and strength.

Data collected from production are given in the Figure 4-5 and the slight improvement on slump and strength shall be detected.

10. CALIBRATION

The system is controlled and calibrated by a special sample cup with defined volume.

- The sample is taken from the measurement loop with the sample cup.
- This sample is weighed on a scale.
- The net mass of sample is divided by the volume of the sample cup to find the density.
- As the density is defined, the user just calibrates the distance between LCA probes until the measured density is read on the computer display.

The system is now calibrated and ready to use. All regular controls are performed in the same way.

11. OUTPUTS

- The density value required by the plant PLC at any time.
- The net fresh water quantity supplied in the mixture.
- The solid particle quantity present in the mixture.

12. BENEFITS

- Real time measurement and verification of density for each batch continuously.
- Possibility of integration to plant PLC or to the user network.
- Almost 100% usage of recycled water.
- Sustainable concrete production.

13. PLANT PHOTOS FOR COMPARISON

Before **After**

REFERENCES

[1] Guidance on concrete wash water in the European ready mixed concrete industry, October 2006, by ERMCO.

[2] Recycled water in ready mixed concrete operations, 2003, by Colin Lobo and Gary M. Mullings.

[3] Use of recycled water in concrete production, August 2007, by Cement Concrete & Aggregates Australia.

[4] Sustainable use of resources – Recycling of sewage treatment plant water in concrete, June 2010, by Marcia Silva and Tarun R. Naik

[5] EN 1008:2002, Mixing water for concrete – Specification for sampling, testing and assessing the suitability of water, including water recovered from processed in the concrete industry, as mixing water for concrete.

MIXING METHOD TO MITIGATE THE NEGATIVE EFFECTS OF RECYCLED AGGREGATE WATER ABSORPTION IN CONCRETE TECHNOLOGY

M. Eckert (1) and M. Oliveira (2)

(1) University of Algarve, Higher Institute of Engineering, Civil Engineering Department, Faro, Portugal

(2) University of Algarve, Higher Institute of Engineering, Civil Engineering Department, Faro, Portugal

Abstract

The widespread use of natural aggregates in construction activities, together with the global population increase, gave rise to a depletion of this natural resource and to a progressive increase of its transport distances. On the other hand, the construction and demolition wastes (C&DW) are deposited in landfills and city outskirts, causing environmental and social problems. The reuse of C&DW in concrete preparation would be a good solution for both problems. Recycled aggregates show however high water absorption. At saturation, water flows from the inside of aggregates to the engaging cement paste matrix and at dryness the opposite process occurs. This water flow breaks down the aggregate-cement paste bonds and increases the W/C ratio in the interfacial transition zone, degrading this way the concrete properties.

In this work a staged mixing procedure was optimized to regulate the water flow, reducing its negative effects on concrete technology. The optimization was based on the aggregate water absorption over time and workability test over time. The physical and geometrical characteristics of the aggregates were related to the properties of concrete in its fresh state. Three types of commercial recycled aggregates were evaluated. Two types of natural aggregates were also studied for comparison purposes.

Keywords: Recycled aggregate, concrete, interfacial transition zone, water absorption, workability

1. INTRODUCTION

Construction activity is a high consuming sector, accounting for the depletion of more than 40% of the energy and more than 50% of natural resources [1]. The current consumption of natural aggregates per year and per person is 3-5 tons [2]. The global use of natural aggregate will reach 26-44 Giga Tons per year in 2030 [2]. Construction is also responsible for the production of 50% of the global waste [1]. Recycling of C&DW has been pointed by several governments as a solution to face these problems. The recycling of C&DW is however a complex challenge due to the high heterogeneity associated to these materials. C&DW and recycled aggregates (RA) prepared thereof generally show lower quality properties than natural aggregates (NA). Studies to define the conditions for their advantageous incorporation in concrete are therefore required [3]. Several works reported that the major problem of RA is their high water absorption due to the high porosity of these materials [4, 5, 6, 7]. If the origin of aggregates is crushed concrete, the amount of water absorption depends on the porosity of the mortar attached to the natural stone [4]. The water absorption capacity ranges from 5 to 15%, depending if the old mortar comes from low or high strength concrete, respectively [8]. RA obtained from ceramic CD&W can absorb more than 30% of water [9].

A major challenge of the concrete technology is the preparation of concrete, showing high performance in both, fresh and hardened states. It is well known that improving the workability reduces the properties of hardened concrete and vice-versa. The high water absorption of the RA makes this optimization even more difficult. Water addition to the mixes results in a higher W/C ratio, increasing the average distance between the binder particles, leading to high microstructural porosity of the concrete. Moreover, when the aggregates are pre-saturated with water, a water flow takes place from its inside to the involving cement paste matrix. This flow breaks the bonds [4], and leads to a higher W/C ratio on the interfacial transition zone (ITZ), which weakens the strength. It has been reported that the optimum water pre-saturation of RA should be about 80% [10].

This work reports an evaluation of the performance of different concrete mixtures prepared using distinct RA. All aggregates, including the control NA, were pre-saturated with water to its optimum moisture state, before cement addition. Tests of water absorption over time allowed the calculation of the extra water and absorption time required to reach the optimum moisture state. Based on the obtained results, a staged mixing approach [4] was followed and optimized to obtain the moisture state of the aggregates before addition of the binder to the mixture. After mixing, a slump test over time proved that there was no significant ITZ water flow and consequently no negative effects on the concrete microstructure. Several other aggregate properties were tested and related to the concrete performance in its fresh state.

2. CHARACTERIZATION OF AGGREGATES

In this work 5 types of coarse aggregates, labeled RA1, RA2, RA3, NA1 and NA2 and two types of fine aggregates, labeled RS and CS, were analyzed. The NA's and CS are of crushed limestone and RS is a fine limestone river sand.

2.1 Constituents of recycled aggregate

Table 1: Constituents of recycled aggregates (NP EN 933-11 [11])

Aggregate/Constituent	Rc (%)	Ru (%)	Rb (%)	Ra (%)	Rg (%)	X (%)	FL (cm³/kg)
RA1	74.50	21.30	3.57	0.14	0.10	0.40	0.55
RA2	37.26	26.20	34.06	0.08	0.03	2.38	0.62
RA3	52.40	10.20	37.20	0.07	0.05	0.09	7.13

Rc - Concrete, concrete products, masonry concrete blocks;
Ru - Unbounded aggregates, natural stone, treated aggregates with hydraulic binder;
Rb - Ceramic elements (e.g. bricks, roof tiles, etc.), non-floating cellular concrete and masonry blocks;
Ra- Bituminous materials;
Rg - Glass;
X - Cohesive materials (soil, clay, etc.), metals, wood, plastic, rubber, stucco;
FL - Floating materials.

The major constituent of RA is old concrete or mortar, followed by ceramic materials and natural stone. This is in agreement with literature [12, 13], where the natural stones are included in the concrete category. The crushing process might lead to a separation of natural stones from mortar. The fraction of other constituents is very low.

2.2 Sieve analysis

RA3 is coarser than RA1 and RA2 despite the same crushing process was followed (Figure 1). RA2 has significantly more fines. These differences may be attributed to the nature of constituents, but all RA can give rise to a compact size distribution near to reference grades. RA shows generally higher content in fines than NA.

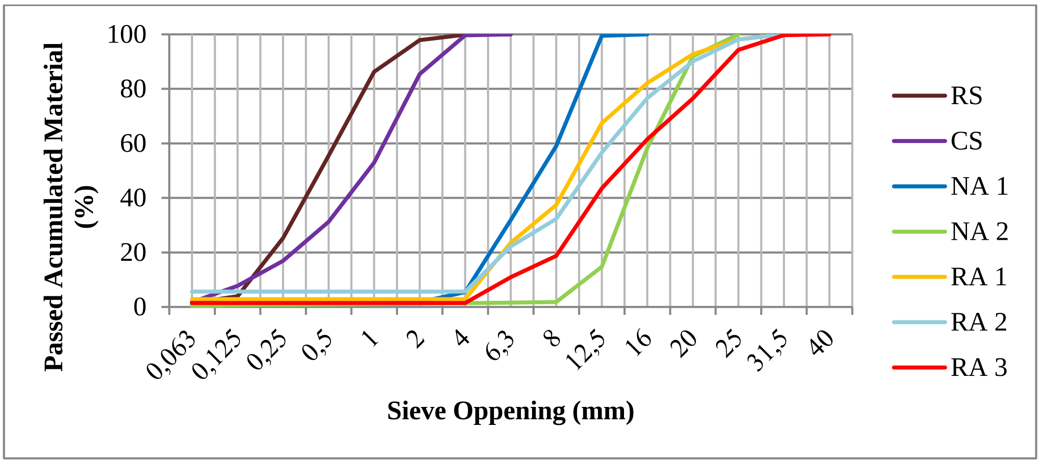


Figure 1: Aggregate sieve analysis

2.3 Physical properties of aggregates

Table 2: Physical properties of aggregates

Property/ Aggregate	NP EN	RS	CS	NA1	NA2	RA1	RA2	RA3
Size class	12620+A1 [14]	0/2	0/4	4/12.5	12.5/20	4/20	4/20	6.3/25
Fines (%)	933-1 [15]	1.5	1.8	0.8	0.3	2.9	5.6	1.4
Shape index (%)	933-4 [16]	------		15.0	10.0	9.7	24.2	29.2
Dry specific density	1097-6 [17]	2.626	2.662	2.648	2.283	2.286	2.162	2.096
Saturated specific density	1097-6 [17]	2.631	2.691	2.676	2.696	2.420	2.331	2.272
Water absorption (%)	1097-6 [17]	0.2	1.1	1.1	0.5	5.9	7.8	8.4

2.4 Specific density and water absorption

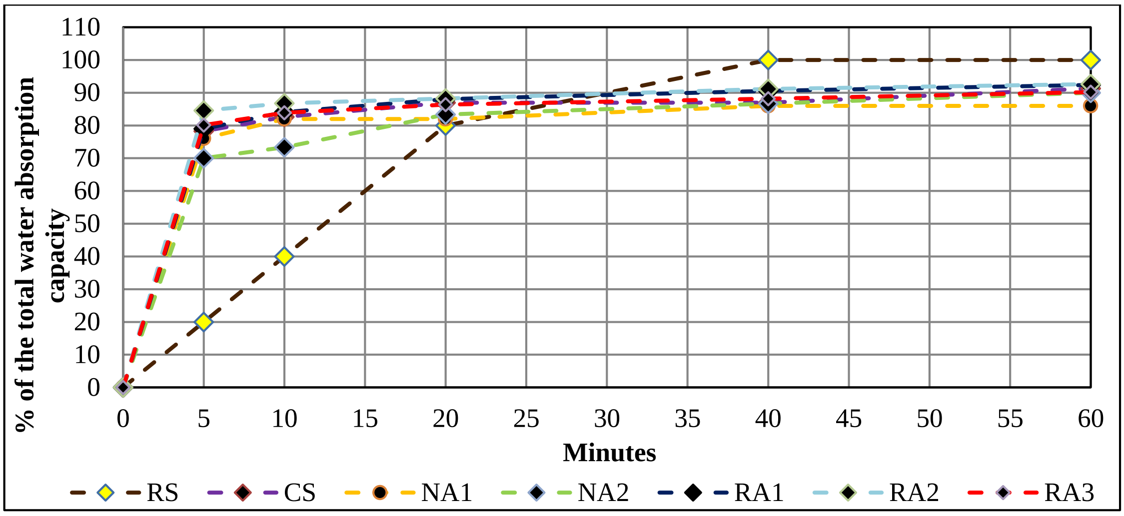


Figure 2: Aggregate water absorption over time, in percentage of the total absorption capacity

With the exception of RS, water absorption behaviour is similar for all aggregates (Figure 2). This behaviour can be separated in 2 phases. In the 1st phase, a quick rate absorption occurs involving the superficial larger pores. In the 2nd, the absorption involves the deeper and smaller pores and takes place at a slower rate. For all aggregates the transition from the 1st to the 2nd phase occurs after 5 minutes, which matches a water absorption of about 80% of its maximum. The optimum amount of pre-saturation water and pre-saturation time should take place at the end of the 1st phase, i.e. after 5 minutes. RS only absorbs 0.2% water meaning that its behaviour is negligible.

The absorption rate of the RA is somewhat faster than that of NA but the differences are negligible after 5 minutes. As expected, RA shows a higher water absorption capacity than the NA, and this increases with the content of ceramic material that can be matched to its densities.

The density of the 4 NA is in the expected range for limestone [18] and that of RA is much lower, decreasing with the content of ceramic particles. The values are also in the range obtained by others [9, 19].

2.5 Shape index

The shape index of old concrete particles is lower than that of NA but it increases dramatically with the content of ceramic materials. Ceramic elements like bricks, roof, floor or wall tiles show a dimension much smaller than the other 2 dimensions, which causes a high shape indexes of ceramic RA. The results are in agreement with the reported by others [20, 18].

3. CONCRETE TEST PROGRAM AND MIXING METHOD

3.1 Mix compositions

The compositions were calculated by following the Faury´s method. The cement and water contents were further adjusted experimentally to reach a concrete class of C30/37 [21] and an initial slump of 130±20 mm, corresponding to the workability class S3 [21]. Five compositions were prepared, a reference mix with NA (NAC) only, and 4 mixes with different types and percentages of RA (RA2C50%; RA1C100%; RA2C100% and RA3C100%). The effective W/C ratio is 0.6 for all mixes and the binder is a CEM I 42.5. Table 3 presents the mix compositions.

Table 3: Mix compositions

Materials/Mixes	NAC	RA1C 50%	RA1C 100%	RA2C 100%	RA3C 100%
RS (kg/m³)	289	274	261	250	246
CS (kg/m³)	343	325	310	297	292
NA1(kg/m³)	523	248	0		
NA2 (kg/m³)	649	308			
RA1-2-3 (kg/m³)	0	556	1059	1018	998
CEM I 42,5 R (kg/m³)	350				
Water (l/m³)	210				
Extra Water (l/m³)	9.52	31.8	51.98	69.83	69.91

3.2 Mixing method

A two-staged mixing approach (Figure 3) was designed to overcome the high water absorption of RA and mitigate any negative effects of the concrete properties, on its fresh and hardened states. After the first mixing stage, all extra water should be absorbed and the cement can be introduced without influencing the effective W/C ratio and the ITZ water flow. The second mixing stage was 2 minutes long, the time required to obtain a homogeneous consistency. A 5 minutes time was taken for the first mixing stage (Figure 2). The percentage of extra water to compensate the aggregate water absorption was also deduced from Figure 2. After mixing, a slump test was prepared during 3 hours to verify if the chosen pre-saturating time and amount of extra water led to the optimum humidity state of the aggregates.

Figure 3: Followed two-staged mixing method

3.3 Test of the workability over time

A long term workability test was performed to evaluate the water flow in the ITZ and the concrete's performance. Workability was evaluated by the slump test according to NP EN 12350-2 [22]. The first reading was performed after finishing the mix and the following readings were taken every 30 minutes for 3 hours. The materials were kept at 20.5 ± 2 ºC and the mixing room was maintained at 23 ±1 ºC. The relative humidity was not controlled. Between measurements, the fresh concrete stayed in the stopped mixer, which was covered, but the moisture exchange with the surroundings was not completely avoided. Before each test the mixer was turned on for about 20 seconds to break the concrete bonds.

4. RESULTS AND DISCUSSION

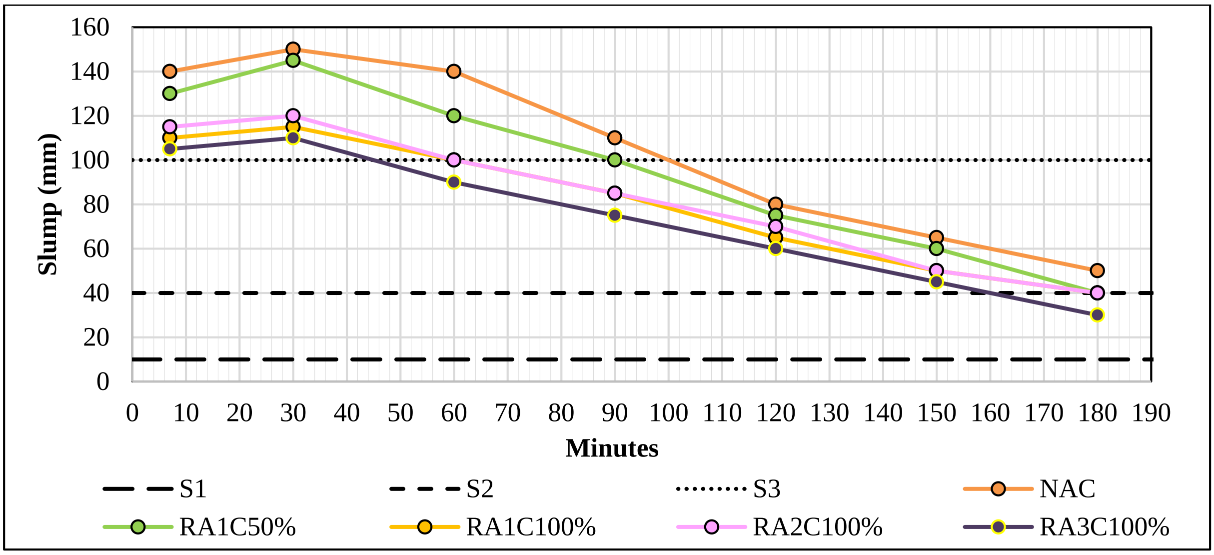


Figure 4: Slump test over time

Figure 4 clearly shows that the ITZ water flow from the RA is nearly suppressed and that the swelling after 30 minutes is almost negligible. The observed small swelling is probably due to the superficial absorption by the natural sand. The slump decrease is nearly linear and the workability changes on time follow the same pattern, indicating that there is no water absorption by the aggregates. NA mixes had significant swellings in the first 90 minutes. This result was unexpected, as their absorption rates were found to be lower than those of RA and at the stage of 5 minutes they reached only 70% of their absorption capacity. The test results shows that the staged-mixing approach, with the chosen parameters, can successfully eliminate the negative effects of recycled aggregate water absorption. The results indicate that this approach works for RA, but should not be used for OC.

Figure 4 also shows that the concrete containing NA has a somewhat higher slump peak and a larger "S3" period. The 3 mixes with 100% RA have a lower slump peak and "S3" period, but after 120 minutes the slump of all mixes became very similar. As the initial mixing water is the same for all compositions, the somewhat lower slump of the 100% RAC was assigned to the undesirable water flow explained previously to a higher surface roughness and shape indexes. After 120 min, the effect of the NA water loss relieves and the new mortar gets attached to the aggregates, lowering the initial slump differences between the mixes. On one hand, the workability of RA1 concrete is reduced due to a greater friction between the rough particles. On the other RA2C100% and RA3100% had shape indexes more than 2 times higher when compared to other mixes, affecting the rheology of the fresh concrete.

Pre-saturation of the RA reduces the fast slump decrease and the high amount of mixing water, needed to compensate this decrease, as observed by Poon et al [6].

5. CONCLUSIONS

It was demonstrated that the staged-mixing approach can be followed to reduce the ITZ water flow of RAC to negligible amounts, in its fresh state. The extra water does not affect the effective W/C and a compact ITZ structure can be develop [4]. A pre-saturation time of 5 minutes and the corresponding extra water incorporated in a two-staged mixing approach, mitigates the negative effects of recycled aggregate water absorption, in concrete technology. However, the procedures should not be followed for natural aggregates.

RA water absorption behavior follows the same pattern for all evaluated aggregates, and for 9 other not reported in this paper. Therefore, the obtained "absorption-time" graphics can be used for concrete mix design and optimization of mixing times.

It was proven that the already suggested 80% pre-saturation [10] is near the optimum because it avoids water exchange in the ITZ.

The maximum slump and "S3" period of the RAC is somewhat lower than that of NAC due to the aggregate surface roughness and higher shape indexes. The staged-mixing approach reduces the fast slump decrease of RA concrete.

It was confirmed that RA has lower properties than NA. The ceramic content increases the shape indexes, water absorption and densities of RA.

Fine RA should be tested to see if the ITZ water flow can be completely mitigated.

ACKNOWLEDGEMENTS

Financial support from the European Regional Development Fun, via Algarve Operational Program, grant QREN 30307 Multivalor, is gratefully acknowledged.

REFERENCES

[1] M. Botelho, Resíduos de Construção e Demolição, Verlag Dashoefer, 2010.

[2] B. Damineli , . F. Kemeid, . P. Aguiar e V. John, Measuring the eco-efficiency of cement use. Cem. Concr. Compos. 32, 2010, p. 555–562.

[3] E 471-2009, Guia para a utilização de agregados reciclados grossos em betões de ligante hidráuliocos, LNEC, 2009.

[4] V. W. Tam, . X. Gao e C. Tam, Microstructural analysis of recycled aggregate concrete produced from two-stage mixing approach, Cement and Concrete Research 35, 2005, p. 1195–1203.

[5] K. Crentsil Sagoe , T. Brown e . A. Taylor, Performance of concrete made with commercially produced coarse recycled concrete aggregate. Cem Concr Res, 2001, p. 31:707–12.

[6] C. Poon , . Z. ShuiL, L. Lam, H. Fok e S. Kou, Influence of moisture states of natural and recycled aggregates on the slump and compressive strength of concrete, Cement and Concrete Research 34, 2004, p. 31–36.

[7] L. Evangelista e J. de Brito, Mechanical behaviour of concrete made with fine recycled concrete aggregates, Cement & Concrete Composites 29, 2007, p. 397–401.

[8] C. Poon, . Z. Shui e . L. Lam, "Effect of microstructure of ITZ on compressive strength of concrete prepared with recycled aggregates," Construction and Building Materials, vol. 18, p. 461–468, 2004.

[9] C. S. Poon e . D. Chan, Feasible use of recycled concrete aggregates and crushed clay brick as unbound road sub-base Construction and Building Materials 20, 2006, p. 578–585.

[10] M. Exteberria, . E. Va´squez e A. R. Marı´, Influence of amount of recycled coarse aggregates and production process on properties of recycled aggregate concrete. Cement and Concrete Research, 37, 2007, p. 735–742.

[11] N. E. 9.-1. 2011, Ensaios das propriedades geométricas dos agregados. Parte 11: Ensaio para classificação dos constituintes de agregados grossos reciclados, 20011.

[12] J. Yang, Q. Du e Y. Bao, "Concrete with recycled concrete aggregate and crushed clay bricks," Construction and Building Materials , vol. 25, pp. 1935-1945, 2011.

[13] B. Mas, A. Cladera, T. del Olmo e F. Pitarch, Influence of the amount of mixed recycled aggregates on the properties of concrete for non-structural use, Construction and Building Materials 27, 2012, pp. 612-622.

[14] NP EN 12620:2002+A1 2010, "Agregados para betão," 2010.

[15] NP EN 933-1, Ensaios das propriedades geométricas dos agregados. Parte 1: Análise granulométrica, Método de peneiração, IPQ, 2000.

[16] NP EN 933-4, Ensaios das propriedades geométricas dos agregados, Parte 4: Determinação da forma das partículas-Índice de Forma.

[17] NP EN 1097-6, Ensaios das propriedades mecânicas e físicas dos agregados. Parte 6: Determinação da massa volúmica e da absorção de água.

[18] P. B. Cachim, Mechanical properties of brick aggregate concrete,Construction and Building Materials 23, 2009, p. 1292–1297.

[19] J. Yang, Q. Du e Y. Bao, Concrete with recycled concrete aggregate and crushed clay bricks,Construction and Building Materials 25, 2011, pp. 1935-1945.

[20] D. Matias, . J. de Brito, A. Rosa e . D. Pedro, Mechanical properties of concrete produced with recycled coarse aggregates – Influence of the use of superplasticizers, Construction and Building Materials 44, 2013, pp. 101-109.

[21] NP EN 206-1, Betão. Parte 1: Especificação, produção e conformidade.

[22] NP EN 12350-2, Ensaios do betão fresco, Parte 2: Ensaios de abaixamento, Portugal: IPQ, 2002.

CARBONATION RESISTANCE: ACCELERATED DETERMINATION AND CONFORMITY ASSESSMENT BASED ON SWISS STANDARD METHOD

F. Moro (1) and S. Cuchet (2)

(1) Holcim Technology, Holderbank, Switzerland

(2) Holcim CH, Eclépens, Switzerland

Abstract

In July 2013 a new Swiss standard SIA 262/1 prescribing performance tests for concrete (permeability, sorptivity, migration, freeze-thaw resistance etc.), including the evaluation of the carbonation resistance of concrete using an accelerated test came into effect. Simultaneously, new performance based specifications for concrete for the exposure classes XC3, XC4 and corresponding upper limits for the carbonation coefficient K_N, used as the measure for the carbonation-induced corrosion resistance, have been defined in the Swiss application document SN EN 206-1. The new test method is based on prCEN/TS 12390-12: Testing hardened concrete – Part 12: Determination of the potential carbonation resistance of concrete: Accelerated carbonation method.

An important quality of such a concrete test is its reliability to rate the performance of different concretes. For that, repeatability and reproducibility have been estimated in two interlaboratory studies. The results show that repeatability, reproducibility and sensitivity of the accelerated carbonation resistance method is well suited for the differentiation of different concretes.

As expected, the introduction of this new performance based specification had some influence on the composition of the XC3 and XC4 concretes and mix proportions had to be adjusted accordingly.

Keywords: Carbonation resistance, conformity, accelerated test method, SIA 262/1

1. INTRODUCTION

Driven by the necessity to lower the CO_2-footprint of concrete, the cement market is changing very rapidly. Especially with the increased use of supplementary cementitious materials (SCMs) it became apparent that the prescriptive approach used in today's concrete standards to specify durability performance is insufficient, even improper. Therefore, there is a global trend to replace these prescriptive standards with performance-based specifications (PBS) in concrete.

Based on 'prCEN/TS 12390-12: Testing hardened concrete – Part 12: Determination of the potential carbonation resistance of concrete: Accelerated carbonation method' a new test method [1] has been developed in Switzerland within the framework of a research project [2,3]. This procedure (SIA 262-1/Appendix I) is in use in Switzerland since August 2013 to determine the carbonation resistance and as a mean of the production control of concrete [4].

The object of this paper is to provide information on the repeatability and reproducibility of the test method, based on two round robin tests, as well as to present the results of production control of concretes for XC3 and XC4 exposure classes produced in Holcim plants in Switzerland.

2. EXPERIMENTAL

2.1 Materials

Within the context of the regular quality control of 21 ready-mix concrete plants of Holcim in Switzerland, the carbonation resistance of concretes for exposure classes XC3 (moderate humidity) and XC4 (cyclic wet and dry) had to be measured. The concretes mixes were produced with commercial Holcim cements and without a separate addition of SCMs at the ready-mix plant, mainly with a CEM II/B-M (T-LL) 42,5 N (Optimo 4) but some also with CEM II/B-M (V-LL) 32,5 R (Bisolvo 3R), CEM II/A-LL 42,5 N (Fluvio 4), CEM I 42,5 N (Normo 4) or CEM II/B-M (S-T) 42,5 R (Robusto 4R). Cement contents used varied in the range of 305 -375 kg/m^3 and for SCC 450 kg/m^3, water-cement ratios in the range of 0.39 – 0.60.

2.2 Experimental

The procedure according to SIA 262-1/Appendix I [1] was used. Prisms (120mm x 120mm x 360mm) were produced in the ready-mix plants according to SN EN 12390-2. After demoulding the prisms were stored under water up to an age of 72h ($\pm$ 4h) * and were pre-conditioned from day 10 up to day 28 at 20 ± 2°C, 57 ± 3% relative humidity and ≤ 0.15 vol.% CO_2. Between day 4-10 the specimens were stored for 7 days under sheltered, dry indoor conditions (<70% rh., 18-24°C).

The intention is to approach job-site conditions and not allow an optimum curing as in many other (performance) tests.

In a corrigendum of 2015 alternatively to the under water storage a 'protected from drying' condition (e.g. applying a plastic film) was allowed, mainly if samples produced on a job side could not be stored under water. Additionally the intermediate conditioning had to be refined.

In the 2013 published original standard this intermediate conditioning phase was described without specifying humidity and temperature. It turned out that variations in this preconditioning could have a strong impact on the final carbonation resistance; therefore humidity and temperature were additionally specified in a corrigendum [5].

This schedule causes a certain difficulties if the concrete is produced Wednesday or Thursday, resulting in a relocation during the week-end.

At 28d the prisms were exposed to accelerated carbonation conditions (20 ±2 °C, 57 ± 3% rh. and 4.0 ± 0.1 vol% CO_2) . After exposure periods of 0d, 7d, 28d and 63d an approximately 50 mm thick slice was broken off the prism (Fig. 1). The remaining prism was then put back into the carbonation chamber. The depth of carbonation on the freshly broken surface of the split slice was determined using a 1% phenolphthalein solution. After applying the indicator solution and allowing for drying, the carbonation depth was determined with a ruler (N≥5 points on each of the four sides of the slices) to 1mm resulting in an overall accuracy for the calculated mean carbonation depth of ± 0.1mm.

2.3 Calculation of the carbonation coefficients

With the measured average carbonation depths d_{KM} [mm] after 0d, 7d, 28d and 63d an accelerated carbonation coefficient K_S is determined assuming a linear √time law:

$$d_{KM} = A + K_S\sqrt{t} \qquad (1)$$

A: constant, [mm]

t: exposure period [day]

K_S : accelerated carbonation coefficient [mm/√day]

With this value of K_S and assuming a natural CO_2 content in the air of 0.04 vol.% CO_2 a natural carbonation coefficient K_N is estimated applying the following equation [1]:

$$K_N = a \cdot b \cdot c \cdot K_S = 2.6 \cdot K_S \qquad (2)$$

K_N: natural carbonation coefficient (0.04 Vol.% CO_2) [mm/√year]

a: conversion 1day to 1 year	$\sqrt{(365/1)}$	= 19.10
b: conversion from 4.0 to 0.04 Vol.% CO_2	$\sqrt{(0.04/4.0)}$	= 0.10
c: correction factor for accelerated carbonation		= 1.36

Figure 1: Testing procedure according to SIA 262-1/Appendix I [1]

3. NEW PERFORMANCE BASED SPECIFICATIONS FOR CONCRETE FOR THE EXPOSURE CLASSES XC3, XC4

With the introduction of the accelerated carbonation test method new limiting values for XC3 and XC4 concrete were defined. For an assumed service life of 50 years a limiting value of K_N<5.0 mm/√year was defined for both exposure classes XC3 and XC4. Differing values for the exposure classes XC3 and XC4 were defined for an assumed service life of 100 years: K_N<4.0 mm/√year for exposure class XC3, and K_N<4.5 mm/√year for exposure class XC4.

The defined limits are based on the carbonation progress being highest at dry humidities and lower under cycling drying-wetting, and corrosion progress requiring a certain high humidity level but slowed down considerably under saturation conditions due to lack of oxygen. Additionally, reinforcement cover requirements are less severe for XC3 than for XC4 exposure classes and carbonation resistance takes this into account.

Frequency of values within the tolerance (resp. limit+0.5 mm/y) is driven by table 19a of SN EN 206-1:2000), where the acceptance numbers for conformity criteria for properties other than strengthare listed.

4. CHARACTERISTICS OF THE METHOD

4.1 Repeatability and reproducibility

So far two round robin tests were organized to determine characteristic properties of the carbonation test method according to SIA 262-1/Appendix I [1]. Results of the first limited round robin test organized in 2013 and involving 4 pilot laboratories and 8 concretes are reported in [6], the main results are shown in Table 1.

Table 1: Precision parameters for the natural carbonation coefficient K_N in [mm/√year]

Repeatability standard error σ_E	Repeatability r	Reproducibility standard error σ_R	Reproducibility R
0.292	0.825	0.356	1.008

A second round robin organized by the VAB (association of accredited building-materials laboratories) and involving 18 laboratories and 4 concretes are reported in [7], the main results are shown in Table 2.

Table 2: Precision parameters for the natural carbonation coefficient K_N in [mm/√year] determined in [7].*In row 3 the precision parameters determined on all but one concrete having a very low carbonation coefficient is shown.

Repeatability			Reproducibility		
Repeat. standard error σ_E	Repeat. CoV [%]	Repeat. r	Reprod. standard error σ_R	Reprod. CoV [%]	Reprod. R
0.09 - 0.23	2.2 - 10.1	0.25 - 0.64	0.29 - 0.59	7.8 - 19.5	0.81 - 1.63
0.09 - 0.23*	2.2 - 4.9*	0.25 - 0.64*	0.30 - 0.59*	7.8 - 8.8*	0.84 - 1.63*

The determined carbonation resistance K_N of one of the concretes was 1.49 mm/√year, a very low value. Correspondingly, the coefficient of variations so-determined are relatively high. It is related to the lower relative precision when measuring low carbonation depths and consequently carbonation coefficients. For common conformity assessment the values in row 3 are more meaningful. The reproducibility CoV of 8.8% when compared to the CoV of other concrete performance test methods (e.g. chloride migration tests) is very low. The test method is therefore well suited for qualifying concretes.

5. IMPLICATION FOR THE CONCRETE PRODUCER

As expected, some of the concretes for exposure classes XC3 and XC4 conforming to the prescriptive standard used so far did not conform to the new performance-based requirements. Therefore, some measures had to be taken in order to meet the requirements. Surprisingly, regression analysis of the data presented in this paper showed that while the water-cement ratio had a significant impact on the resulting carbonation resistance, cement content was not significant. This conclusion is shown on the following two scatterplots (Fig. 2), where the measured carbonation coefficients are plotted versus water-cement ratio(left) and cement-content(right).

Due to the few concrete produced with other cements than CEM II/B-M (T-LL) 42,5 N the influence of the cement type cannot be conclusively derived from these data, but as reported also in [6] the choice of cement type significantly influences carbonation resistance. The mixes made with CEM I cement show generally a lower carbonation rate than those made with the other cement types. In this case, the carbonation coefficients K_N of all concretes produced with a CEM I 42,5 N are below 3 mm/√year.

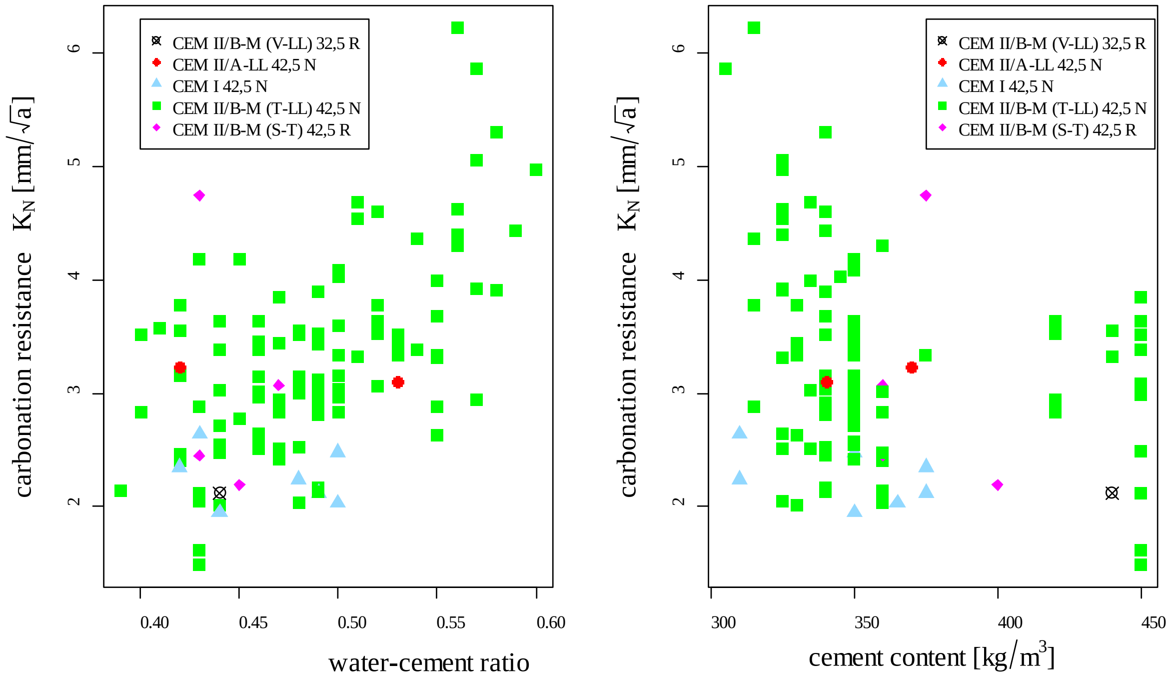


Figure 2: Measured carbonation coefficients K_N versus water-cement ratio (left) and cement content (right)

The duration of the carbonation test is 28d pre-conditioning + 63d test duration = 91 days. This long period of time between production and final verification delays any interventions if needed.

This limits the effectiveness of the test for quality control purposes. But it has been established that the final carbonation resistance could be predicted with satisfactory precision using the measured carbonation depths after 0d and 7d accelerated carbonation exposure (figure 3). On one hand, the values calculated using the carbonation depths 0-7d are almost all higher than the carbonation coefficients obtained at the end of the test. The probability that concretes complying using the estimated carbonation coefficient using only 0-7d values, but not complying at the end of the test is very low. On the other hand, an even more precise prediction of the final carbonation coefficient is obtained by fitting a linear model using 0-7d carbonation coefficients and water-cement ratio.

In this manner 35 days after production, just 7 days after determination of the 28d compressive strength, a reliable indication of the final carbonation resistance is obtained, allowing for corrective measures to be applied much earlier.

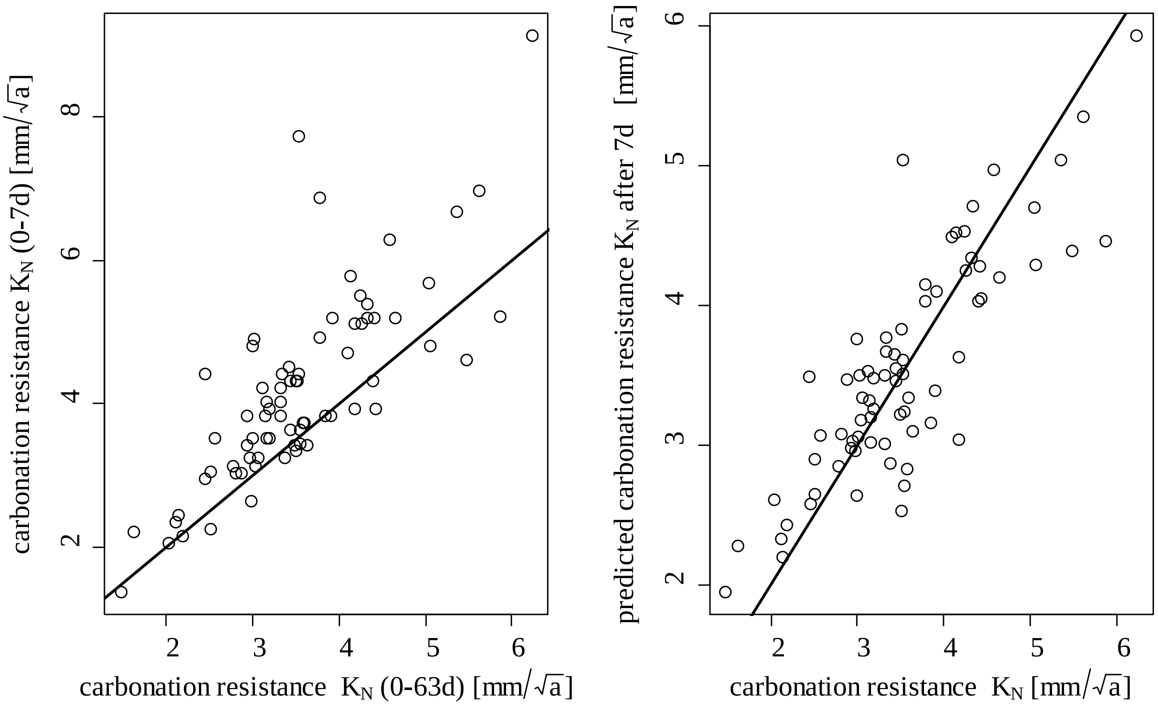


Figure 3: In the left figure the calculated carbonation resistances using 0-7d vs 0-63d values are shown. In the right figure the predicted carbonation coefficient K_N (with a model using 0-7d accelerated carbonation depth values and water-cement ratio) versus the standard carbonation coefficient K_N (using as described 0d, 7d, 28d and 63d carbonation depth values)

6. CONCLUSION

Two years after its introduction the accelerated carbonation resistance test according to Swiss standard, the related limits for concretes for XC3 and XC4 exposure classes and the production control of concrete in the ready-mix plants for these concretes are established. In some cases concrete producer had to adapted their mix-design in order to meet the requirements. The test method has been shown to be reliable.

REFERENCES

[1] SIA 262/1:2013 ‘Concrete Structures – Supplementary specifications’, Annex I "Carbonation resistance
[2] F. Hunkeler, Einfluss des CO2-Gehaltes, der Nach- und Vorbehandlung sowie der Luftfeuchtigkeit auf die Karbonatisierungsgeschwindigkeit von Beton, Beton- und Stahlbetonbau, Vol.107, Heft 9, 2012, S. 613-624.
[3] F. Hunkeler und L. Lammar, Anforderungen an den Karbonatisierungswiderstand von Betonen. Forschungsauftrag AGB 2008/012 auf Antrag der Arbeitsgruppe Brückenforschung (AGB), Bericht VSS Nr. 649, November 2012.
[4] SN EN 206-1/NE:2013 ‘Beton – Teil 1: Festlegung, Eigenschaften, Herstellung und Konformität – Nationale Elemente NE zur Norm SN EN 206-1:2000’
[5] SIA 262/1:2015 ‘Concrete Structures – Supplementary specifications’, Corrigenda C1 to [1], document Nr. 505 262/1-C1

[6] Moro, F., Hunkeler, F., Nygaard, P. and Cuchet, S. ‘Carbonation resistance: accelerated determination and limit values based on Swiss standard method’, Proceeding of the RILEM International workshop on performance-based specification and control of concrete durability, Zagreb, June, 2014 85-92
[7] Loser, R ‘VAB-Ringversuch Karbonatisierungswiderstand von Beton nach SIA 262/1, Anhang I, Schlussbericht’, VAB/ALA Bericht 2-1-040-02.15a, February, 2015

THE PROS AND CONS OF SPECIFYING STRENGTH AT 91 DAYS

T A Harrison

Professor, Chairman of the ERMCO Environmental and Technical Committee

Abstract

The drive to produce more sustainable concrete has led to the wider use of cements with lower clinker contents. Such cements develop their strength more slowly but in the longer term give higher strengths provided water is available for continued hydration. Specifying strength at 91 days would take account of this slower strength gain and lead to more sustainable concretes due to lower Portland cement clinker content. The advantages in terms of structural performance have to be balanced against possible delays to the construction process due to a lower early strength and the technical issues and commercial risks associated with conformity at 91 days.

This paper describes the various issues associated with specifying strength at 91 days and offers solutions to the downside issues. Provided the downside issues are correctly handled, there is a strong sustainability case for specifying strength at 91 days.

Keywords: Specification, compressive strength, sustainability, conformity, production control, formwork striking times, early-age thermal cracking.

1. INTRODUCTION

The European concrete structures committee (CEN/TC250/SC2) and the concrete committee (CEN/TC104/SC1) have held initial discussions on basing the characteristic strength on the 91 day strength as the norm. This proposal is being pushed by the concrete designers [1] as they see a number of structural and environmental benefits; however, before any decision can be taken, the durability, construction and control issues need to be considered. This paper is a contribution to those discussions.

2. ENVIRONMENTAL ASPECTS

With respect to the material concrete, the Portland cement clinker content of concrete is the main contributor to global warming potential [2], see Figure 1. Consequently the cement industry is developing new cements with lower clinker contents [3] and the concrete industry is reducing the impact of concrete by selecting such cements or using additions, e.g. fly ash, directly in the concrete mixer. One impact of the secondary cementitious materials and additions (limestone excepted) is that compared with CEMI concrete they enhance strength development after 28 days provided there is water for hydration. If the characteristic strength were to be based on 91 day strength and not 28 day strength, there is the potential to reduce the proportion of Portland cement clinker in the cement or concrete and/or reduce the cement content provided the concrete retains a closed structure. This would further reduce the environmental impact of concrete structures.

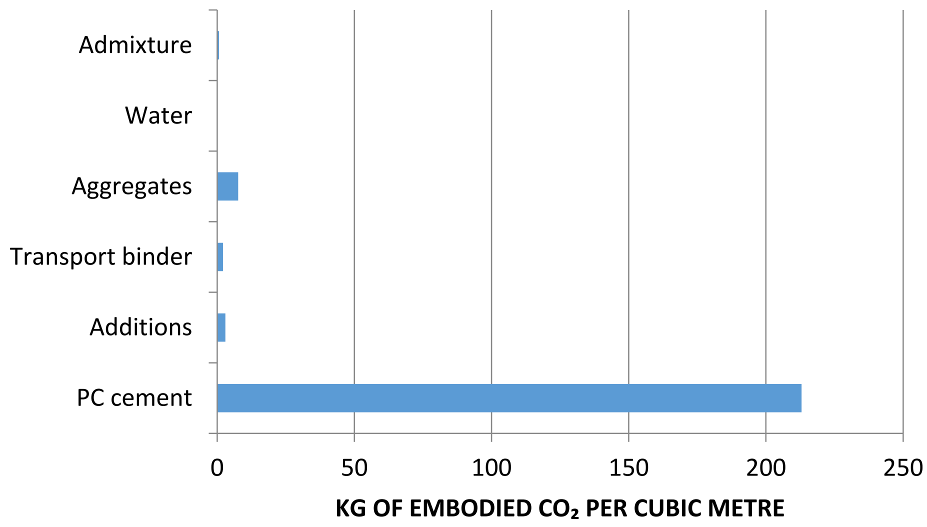


Figure 1: Embodied carbon dioxide per cubic metre of average UK concrete

3. STRUCTURAL ASPECTS

Leivestad [1] has identified a number of structural benefits resulting from basing the characteristic strength on the 91 day strength. He also questions why the full potential strength of concrete is not utilised when no account is taken of the strength development after 28 days, which may be in the order of 35 to 50% [1]. Furthermore, high early strengths lead to higher temperature rises and to a higher risk of early-age thermal cracking. In addition higher early concrete strengths lead to the need for higher proportions of crack control reinforcement. Thus, from a structural viewpoint there are several benefits and some safety issues (over-strength) are taken care of by basing characteristic strength on 91 days and no downside.

Concern will be raised as to whether concrete in rapid multi-storey construction will achieve the expected in-situ strength. These concerns might originate from data on the strength development of test specimens exposed to indoor air. Marsh and Ali [4] showed that data from test specimens over-estimate the impact on structures. CIRIA [5] showed that a 25% reduction in the outer 25mm of a section had the following impacts:

- flexural strength 93%;
- shear strength 98%;
- bond strength no change.

4. DURABILITY

There are those who argue that compressive strength has nothing to do with durability and within specific boundaries they are correct; however, it is too simplistic to ignore the impact of basing characteristic strength on 91 days and not 28 days. Assuming that the traditional method of specification of durability is applied, the concrete producer will check the concrete for conformity to the maximum w/c ratio for durability and the w/c ratio needed to achieve the target strength. If the w/c ratio needed to achieve the target strength is higher than the maximum w/c ratio specified, the producer may increase the proportion of additions so that the w/c ratio to achieve the target strength is the same as the maximum w/c ratio provided that the proportion of addition does not exceed the permitted value. As most national limiting values permit a wide range of cement/binder types, the net effect is likely to be an increase in the proportion of additions. This increase in proportions will in general:

- reduce the resistance to carbonation;
- reduce the freeze-thaw resistance;
- with GGBS and fly ash, increase the chloride resistance;
- with limestone, decrease the chloride resistance;
- increase the resistance to sulfate attack, limestone excepted.

In addition, abrasion resistance is a function of the concrete strength and so a reduction in the 28 day strength will lead to a reduction in abrasion resistance. Whether such changes in performance are significant is an open question. The author suspects that some national limiting values are based on cements that are available in the marketplace and do not take account of cements or binders at the limits of composition when combined with aggregates that are permeable to aggressive species, e.g. carbon dioxide, chloride ions, or poorly shaped. For example a cement with 40% fly ash combined with 50% of the coarse aggregate being recycled concrete gives a rate of carbonation that is significantly higher than ‘normal’ [6].

NOTE: There may be issues with respect to creep and drying shrinkage with such high proportions of recycled concrete aggregate and CEN/TC250/SC2 is currently considering the impact on design when the concrete contains recycled aggregates.

The solution is to specify durability by performance as in this case adequate durability performance will be proven.

5. CONSTRUCTION

There are several aspects of construction that need to be considered. Firstly a reduced 28 day strength also results in lower early strengths and potentially longer times before formwork may be stripped. In large sections this is unlikely to be significant but it will impact on striking times to soffit formwork of suspended slabs. Using rates of strength development data from reference [1] and assuming a compressive strength class of C30/37 and a target 2:1 cylinder strength of 38 N/mm^2, the rates of strength development is given in Figure 2 for equal 28 day strength and in Figure 3 for equal 91 day strength. The estimated time to achieve 10 N/mm^2 is given in Table 1.

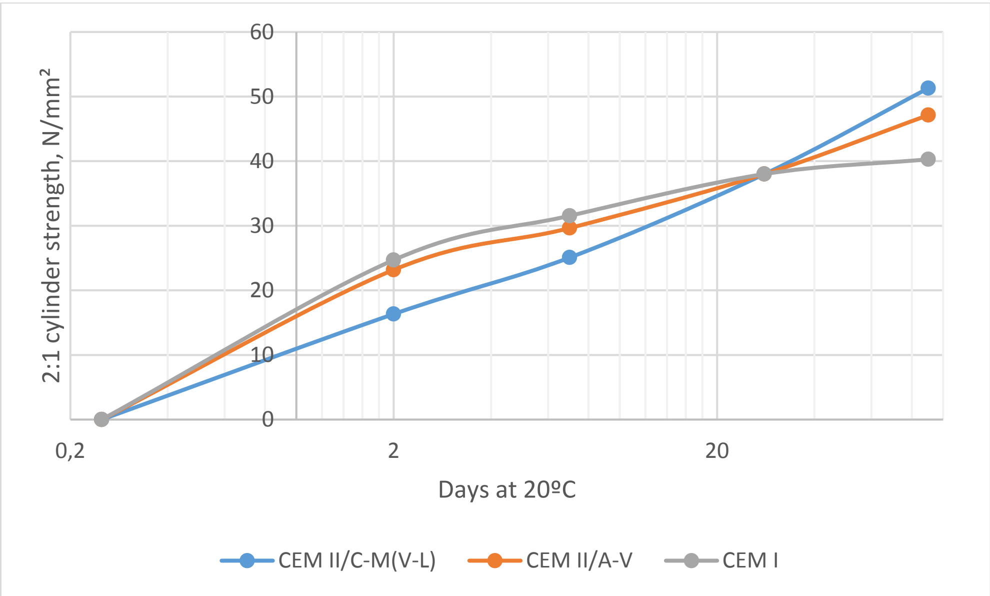


Figure 2: Rate of strength development for a C30/37 at equal 28 day strength

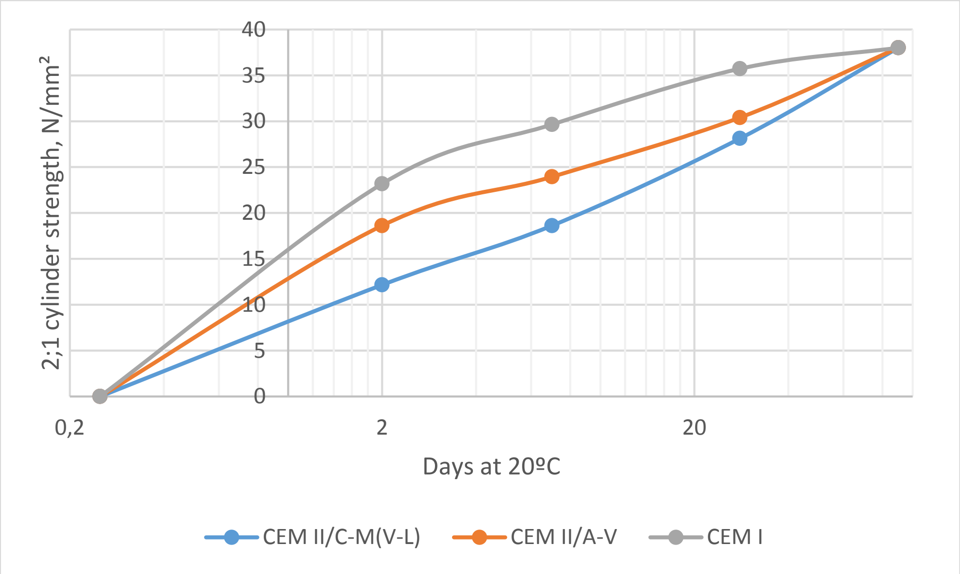


Figure 3: Rate of strength development for a C30/37 at equal 91 day strength

Table 1: Estimated time in days to 10 N/mm^2 for C30/37 concretes

Cement type	Equal 28 day strength				Equal 91 day strength			
	Average concrete temperature, ºC				Average concrete temperature, ºC			
	20	15	10	5	20	15	10	5
CEM II/C-M(V-L)	0.91	1.21	1.73	2.64	1.35	1.82	2.59	3.97
CEM II/A-V	0.61	0.82	1.17	1.79	0.75	1.01	1.44	2.20
CEM I	0.58	0.78	1.11	1.70	0.60	0.81	1.15	1.76

These estimates show that at an average concrete temperature of 10ºC, which is a reasonable value for the spring and autumn in northern Europe, the change from conformity at 28 days to conformity at 91 days using the same cement type at worse increases the formwork striking time by 1 day. A change from a CEM II/A-V cement to one of the new CEM II/C-M (V-L) cements increases the formwork striking time by about 0.5 days and, if the change in the conformity period led to the use of these new cement types, the formwork striking time are almost doubled. A slower strength gain may also impact on subsequent operations, but usually this can be resolved by re-propping, which is the process whereby as the soffit formwork is removed, props are placed to take the load.

On the other hand the use of the 91 day strength will reduce the early-age temperature rise and the risk or extent of early-age thermal cracking and also the risk of delayed ettringite formation.

In order to get a feel for the magnitude of these changes, if we take a 700mm wide section in plywood formwork and an assumed cement content of 291 kg/m^3, the temperature fall from peak temperature to mean ambient temperature is 33ºC for the CEMI, 28 to 29ºC for the CEM II/A-V and 25 to 26ºC for the CEM II/C-M [7]. This assumes that the CEM II/C-M would have a heat output that would qualify it as a very low heat cement. Even without a change in cement type, a 91 day strength would require a higher w/c ratio and a lower cement content (if permitted by the specification) and the lower cement content will give a lower temperature rise in the concrete. A reduction in the temperature fall leads to a reduction in the crack control reinforcement and a further improvement in the sustainability.

6. PRODUCTION CONTROL AND CONFORMITY

It is not a satisfactory solution to any of the parties involved to have to wait for 91 days before conformity of the concrete is proven. EN 206:2013 offers a possible solution as it permits conformity to be based on the use of control charts such as CUSUM [8]. CUSUM uses the measured 7 day strengths to predict the 28 day strength until the real 28 day strengths are available. There is no reason why this principle could not be applied to predicted 91 day strengths. Practice has shown that the ratio of 7 to 28 day strength is not constant and a second CUSUM assessment is made to detect changes in this ratio and corrections are applied when a change is detected. As it is unlikely that the ratio of 7 to 91 day strength will be constant, this procedure could be extended to the ratio of 7 to 91 day strength.

In general producers do not want to operate two control systems and they would like either 28 days or 91 days to be the norm. There would also be significant 'education' issues resulting from a change to the norm being 91 days. As this would be such a major change, there would be widespread publicity about the change, but there will still be some specifiers who would not be aware of this change. While ERMCO does not believe identity testing is needed for concrete from companies with third party certification, it is the reality in some places. If there is agreement to adopt 91 day strength for conformity, ERMCO needs to discuss whether they would support identity testing being based on 7-day strength only.

7. CONCLUSION

There are structural and environmental benefits for changing the basis of characteristic strength from one based on testing at 28 days to one based on testing at 91 days. From a construction viewpoint there are potential benefits (reduced temperature rise and reduced risk of early-age thermal cracking) but there is also a downside in that formwork striking times would be longer.

Concrete producers can cope with the characteristic strength being based on 91 days if they use the option in EN 206:2013 of using control charts, but having to run two control systems will result in problems.

REFERENCES

[1] Leivestad S, *Consider redefining the strength classes by increasing the hardening period from 28 days to 91 days*, draft memo dated 2014-08-25.

[2] Harrison T A, Jones R, Kandasami S and Khanna G, *Effectiveness of the traditional parameters for specifying carbonation resistance*, Magazine of Concrete Research, 2012, 64(1) p1 -11.

[3] European Standardization Committee, *Cement- Part1: Composition, specifications and conformity criteria for common cements*, draft revision EN 197-1 sent for public comment, June 2014.

[4] Marsh B K and Ali M A, *Assessment of the effectiveness of curing on the durability of reinforced concrete*, 3rd CANMET/ACI Int. CONF. on the durability of concrete, ACI SP-145, pp1161-1176.

[5] Ciria Project Report 49, *On-site curing – Influence on the durability of concrete: A review*, 1997.
[6] Harrison T A, *Meeting the challenge of efficient and sustainable resource use,* University of Dundee Conference, 2012, Conference theme: Efficient and sustainable use of resources, Keynote paper.
[7] Dhir R K, Paine K A and Zheng L, *Design data for use where low heat cements are used*, University of Dundee, DTI research Contract No. 39/3/680 CC2257), March 2004.
[8] Gibb I and Harrison T, *Use of control charts in the production of concrete,* October 2010, available for free download from the ERMCO website.

CONCRETE FOR A BETTER SUSTAINABILITY OF BUILDING

Vincent Waller (1, 2), Olivier Collin (1, 3) and Jean-Marc Potier (1)

(1) SNBPE

(2) Unibéton

(3) Lafarge Bétons

Abstract

As the building sector is one of the biggest "energy and resources consumers", to include environmental aspects into building design is a key issue. To fulfil this objective, many actions are followed by SNBPE and its members:

- Communicate environmental information on products
  - BETie, the French internet tool to create concrete EPD was modified to take into account EN 15804
- Promote recycling of concrete into concrete
  - SNBPE is strongly involved in The National Project for Research and Development "RECYBETON"
  - The aim of RECYBETON is to promote re-using Construction and Demolition Waste (CDW) and especially the concrete part (including the fine particles) as components of new concrete or of hydraulic binders.
- Develop concrete with thermal insulation properties
  - To minimize "thermal bridges"
  - To fulfil the French thermal regulation with traditional construction methods
  - These concretes are designed to obtain a thermal conductivity "λ" $\leq$ 0.6 W/(m.K) compared to a thermal conductivity of 2 for normal concrete
  - Concrete in line with EN 206 with compressive strength classes between LC 20/22 and LC 30/33, dry density $\leq$ 1.4, consistence (Slump test) S4 or S5
  - Design of the structure performed according to Eurocode 2 and Eurocode 8

Keywords: EPD; recycling; insulation; energy efficiency

1. INTRODUCTION

The building sector is one of the biggest "energy and resources consumers":

- buildings and the built environment uses 50% of the materials taken from the Earth's crust;
- during their life cycles buildings comprise the largest energy consuming sector with the almost half of the primary energy used and generating about 40 % of all greenhouse gas emissions in Europe;
- waste produced from the building process is the source of 25 % of all waste generated.

So, to include environmental aspects into building design is a key issue. To fulfil this objective, many actions are followed by SNBPE and its members.

2. COMMUNICATE ENVIRONMENTAL INFORMATION ON PRODUCTS

2.1 The BETie tool

BETie stands for **BET**on (Concrete) **i**mpact **e**nvironmental (Environmental Impact). BETie creates an EPD (Environmental Product Declaration) of a concrete element taking into account:

- the distance and method of transport of constituents from their production site to the concrete plant (cement, aggregates, additions, admixtures…);
- the exact concrete composition;
- the distance between the plant and the job site;
- the dimension of the functional unit:
    - thickness of a wall or a slab (functional unit = 1 m²);
    - section of a column or a beam (functional unit = 1 linear metre);
    - reinforcement quantity;
- the method of placing concrete (concrete pump, crane or directly from the truck mixer).

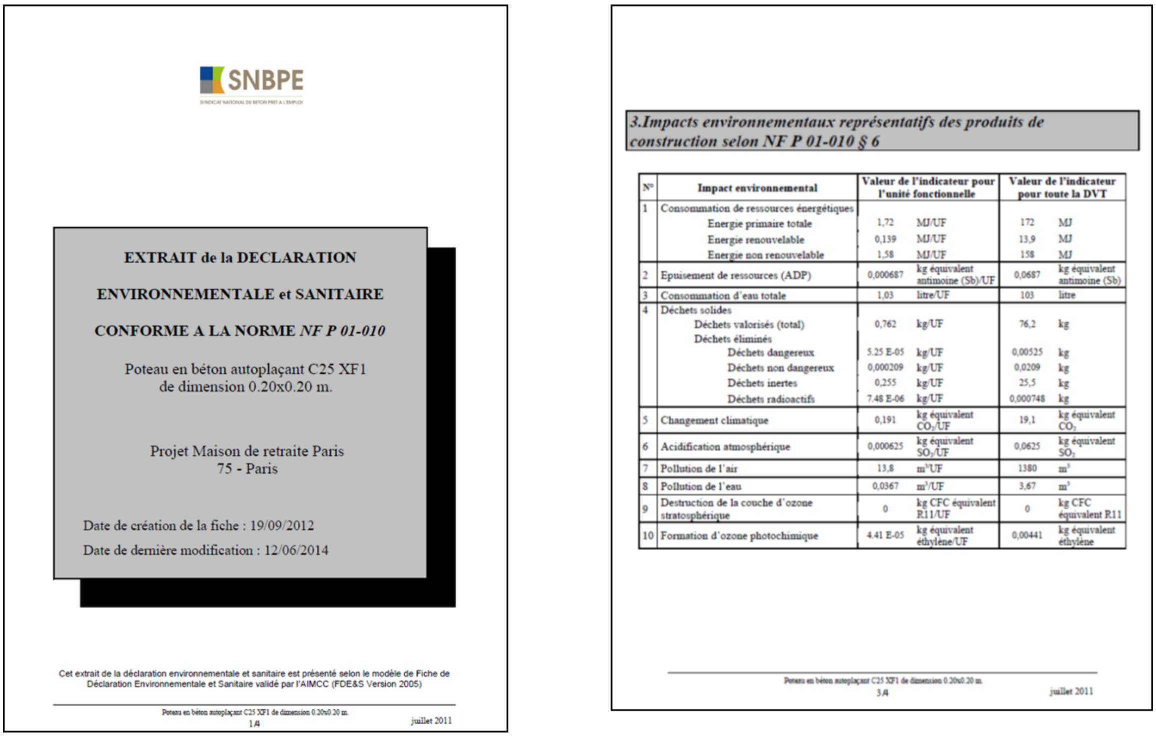
BETie was created in 2011 according to the French standard on EPDs, NF P 01-010, in use at this time.

2.2 The « new BETie »

In 2014, NF EN 15804 (+A1) was published in France, replacing NF P 01-010. BETie was modified accordingly.

BETie was also modified to take into account the modification of NF EN 206:

- possibility of using additions with CEM II/A;
- use of fibres (metallic and organic);
- use of recycled aggregates.



SNBPE

EXTRAIT de la DECLARATION

ENVIRONNEMENTALE et SANITAIRE

CONFORME A LA NORME *NF P 01-010*

Poteau en béton autoplaçant C25 XF1
de dimension 0.20x0.20 m.

Projet Maison de retraite Paris
75 - Paris

Date de création de la fiche : 19/09/2012

Date de dernière modification : 12/06/2014

Cet extrait de la déclaration environnementale et sanitaire est présenté selon le modèle de Fiche de Déclaration Environnementale et Sanitaire validé par l'AIMCC (FDE&S Version 2005)

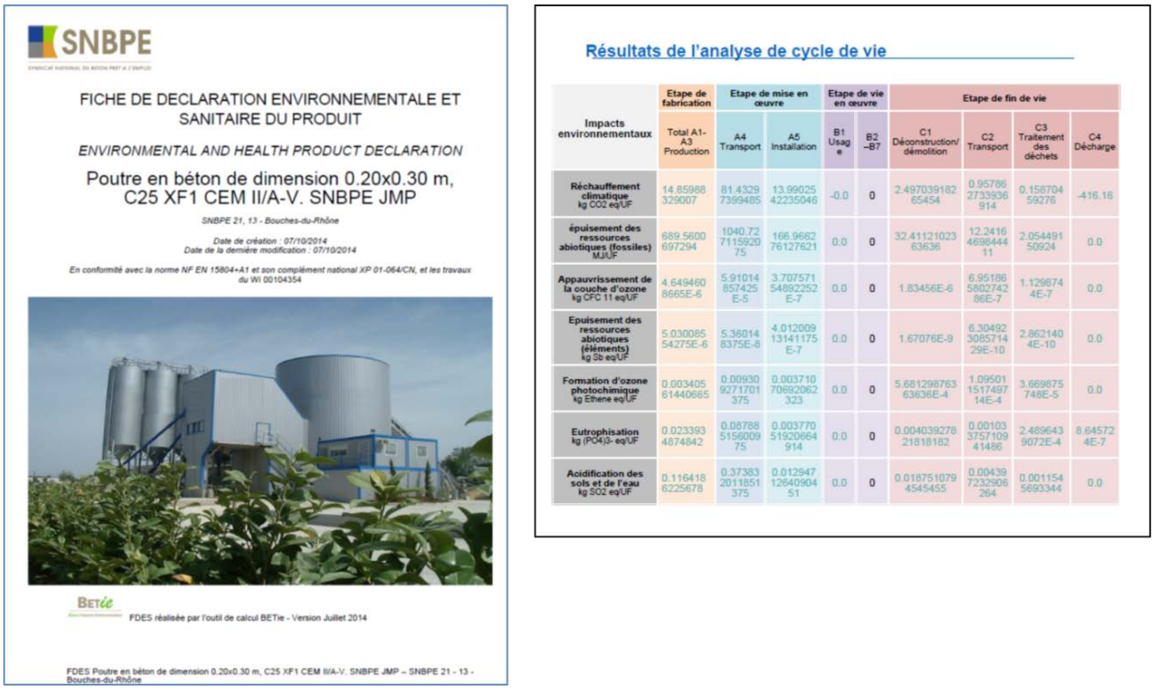
Poteau en béton autoplaçant C25 XF1 de dimension 0.20x0.20 m.
1/4 juillet 2011

3.Impacts environnementaux représentatifs des produits de construction selon NF P 01-010 § 6

N°	Impact environnemental	Valeur de l'indicateur pour l'unité fonctionnelle		Valeur de l'indicateur pour toute la DVT	
1	Consommation de ressources énergétiques				
	Energie primaire totale	1,72	MJ/UF	172	MJ
	Energie renouvelable	0,139	MJ/UF	13,9	MJ
	Energie non renouvelable	1,58	MJ/UF	158	MJ
2	Epuisement de ressources (ADP)	0,000687	kg équivalent antimoine (Sb)/UF	0,0687	kg équivalent antimoine (Sb)
3	Consommation d'eau totale	1,03	litre/UF	103	litre
4	Déchets solides				
	Déchets valorisés (total)	0,762	kg/UF	76,2	kg
	Déchets éliminés				
	Déchets dangereux	5.25 E-05	kg/UF	0,00525	kg
	Déchets non dangereux	0,000209	kg/UF	0,0209	kg
	Déchets inertes	0,255	kg/UF	25,5	kg
	Déchets radioactifs	7.48 E-06	kg/UF	0,000748	kg
5	Changement climatique	0,191	kg équivalent CO_2/UF	19,1	kg équivalent CO_2
6	Acidification atmosphérique	0,000625	kg équivalent SO_2/UF	0,0625	kg équivalent SO_2
7	Pollution de l'air	13,8	m^3UF	1380	m^3
8	Pollution de l'eau	0,0367	m^3/UF	3,67	m^3
9	Destruction de la couche d'ozone stratosphérique	0	kg CFC équivalent R11/UF	0	kg CFC équivalent R11
10	Formation d'ozone photochimique	4.41 E-05	kg équivalent éthylène/UF	0,00441	kg équivalent éthylène

Poteau en béton autoplaçant C25 XF1 de dimension 0.20x0.20 m.
3/4 juillet 2011

Figure 1: Example of an EPD edited with BETie in 2011



SNBPE

FICHE DE DECLARATION ENVIRONNEMENTALE ET SANITAIRE DU PRODUIT

ENVIRONMENTAL AND HEALTH PRODUCT DECLARATION

Poutre en béton de dimension 0.20x0.30 m, C25 XF1 CEM II/A-V. SNBPE JMP

SNBPE 21, 13 - Bouches-du-Rhône

Date de création : 07/10/2014
Date de la dernière modification : 07/10/2014

En conformité avec la norme NF EN 15804+A1 et son complément national XP 01-064/CN, et les travaux du WI 00104354

BETie

FDES réalisée par l'outil de calcul BETie - Version Juillet 2014

FDES Poutre en béton de dimension 0.20x0.30 m, C25 XF1 CEM II/A-V. SNBPE JMP – SNBPE 21 - 13 - Bouches-du-Rhône

Résultats de l'analyse de cycle de vie

Impacts environnementaux	Etape de fabrication	Etape de mise en œuvre		Etape de vie en œuvre		Etape de fin de vie			
	Total A1-A3 Production	A4 Transport	A5 Installation	B1 Usage	B2 –B7	C1 Déconstruction/ démolition	C2 Transport	C3 Traitement des déchets	C4 Décharge
Réchauffement climatique kg CO2 eq/UF	14.85988329007	81.43297399485	13.9902542235046	-0.0	0	2.49703918265454	0.957862733936914	0.15870459276	-416.16
épuisement des ressources abiotiques (fossiles) MJ/UF	689.5600697294	1040.72711592075	166.966276127621	0.0	0	32.4112102363636	12.2416469844411	2.05449150924	0.0
Appauvrissement de la couche d'ozone kg CFC 11 eq/UF	4.6494608665E-6	5.91014857425E-5	3.70757154892252E-7	0.0	0	1.83456E-6	6.95186580274286E-7	1.1298744E-7	0.0
Epuisement des ressources abiotiques (éléments) kg Sb eq/UF	5.03008554275E-6	5.360148375E-8	4.01200913141175E-7	0.0	0	1.67076E-9	6.30492308571429E-10	2.8621404E-10	0.0
Formation d'ozone photochimique kg Ethene eq/UF	0.00340561440665	0.009309271701375	0.00371070692062323	0.0	0	5.68129876363636E-4	1.09501151749714E-4	3.669875748E-5	0.0
Eutrophisation kg (PO4)3- eq/UF	0.0233934874842	0.08788515600975	0.00377051920664914	0.0	0	0.00403927821818182	0.00103375710941486	2.4896439072E-4	8.645724E-7
Acidification des sols et de l'eau kg SO2 eq/UF	0.1164186225678	0.373832011851375	0.0129471264090451	0.0	0	0.0187510794545455	0.004397232906264	0.0011545693344	0.0

Figure 2: Example of an EPD edited with BETie in 2015

3. PROMOTE RECYCLING OF CONCRETE INTO CONCRETE

3.1 French national research & development project RECYBETON

SNBPE is strongly involved in The National Project for Research and Development "RECYBETON". This National Project federates 44 French partners (contractors, producers, users, technical centres, universities, administration).

The aim of RECYBETON is to promote re-using all the materials of deconstructed concrete (including the fine particles) as components of new concrete or of hydraulic binders.

In France, nearly 20 Mt of concrete waste material could be recovered by its integral recycling, which would result in:

- limiting or even eliminating sending waste concrete to landfill;
- use of natural resources rationally;
- reduced material transport, as "recycled aggregate" resources often exists near construction sites, in particular in large urban centres.

This will help in reducing CO_2 emissions and energy consumption. The program of the PN RECYBETON contributes to this.

Figure 3: The partners of RECYBETON

Organization of the project

The project is subdivided in 5 main themes:

Theme 1: Technologies and Processes

This theme deals with three subjects:

- T1.1 – Crushing and Sorting: to improve the technologies used to separate concrete from other construction waste materials as well as the sorting processes for the different concrete components

- T1.2 – Cement Incorporating Material from Recycled Concrete: to optimize the processes for incorporating the fine particles obtained by crushing concrete into the manufacture of cement
- T1.3 – Concrete Incorporating Material from Recycled Concrete: to optimize the processes for using recycled concrete aggregates in the production of new concrete.

Theme 2: Materials and Structures

In this theme, five subjects are included:

- T2.1 – Recycled Aggregates and Fines: to extend the use of all particle sizes obtained from crushed concrete into traditional concrete
- T2.2 – Recycled Concrete at Early Age and during Hardening: to control the changes induced by the use of recycled aggregates during the early age and hardening of concrete
- T2.3 – Hardened Recycled Concrete – Mechanical behavior: to evaluate the changes in the mechanical properties of recycled concrete in order to minimize any adverse impacts
- T2.4 – Hardened Recycled Concrete – Durability: to determine the parameters with an impact on its durability
- T2.5 – Recycled Concrete – Fire and Thermal Behavior: to study its behavior at high temperatures

Theme 3: Sustainable Development

This theme is divided into two important subjects:

- T3.1 – Social and Economic Aspects: to evaluate the economic and social impacts of recycled concrete and of developing the concrete recycling sector
- T3.2 – Environmental and Health Aspects: to determine and control the possible impacts of recycling concrete on the environment and on health

Theme 4: Standard and Normative Aspects (Transversal theme)

This theme deals with the identification of incentives and barriers to the use of recycled concrete at French and European levels, in order to propose adaptations or modifications to existing standards. It will prepare a recommendation guide, to give a framework for the use of recycled concrete aggregates in new concretes

Theme 5: Communication and Promotion (Transversal theme)

This theme has set up a web site (www.pnrecybeton.fr) with both a public access and a private collaborative platform for the partners. It will also communicate via conferences, publications and vocational training actions.

An additional theme deals with the possibility of using composite aggregates containing a small certified part (around 10%) of recycled aggregates mixed with natural aggregates.

The first results

Theme 1:

- Influence of crusher type (jaw, gyratory or impact crusher) on the quality of recycled concrete aggregates: in laboratory tests, the best results are obtained with gyratory and impact crushers
- Incorporating recycled concrete sands into cement raw meal: the cement potential of recycled sands is proven, with substitution rates that could be higher than 30 %,

depending on the best compromise on "recycled sands – virgin raw materials". Some remaining drawbacks should be solved through industrial tests

- Incorporating recycled concrete sands with the clinker to manufacture blended cement: the compatibility of recycled concrete sands reactivity with that of cement was assessed. Some remaining drawbacks should be solved through industrial tests

Theme 2:

- Identification of the variability of recycled concrete aggregates due to different geographical locations of the resources: 16 different productions from 13 platforms were tested and the possibility of using them has been confirmed
- Checking the validity of existing test standards for natural aggregates (on mechanical, physical and chemical properties) on recycled concrete aggregates: the results of tests are less conclusive when testing recycled aggregates rather than natural ones. Necessity of adapting the tests and then the standards? The testing procedures and standards need to be revised accordingly
- Some partial results are obtained on recycled concrete early-age rheology, tensile and compressive behavior, and on durability: the general trend is better for mechanical properties than for rheology and durability. Awaiting further results to confirm the preliminary trend

Theme 3:

A data inventory of the areas around the city of Lyon where CDW (Concrete Demolition Waste) could be recycled has been made using the GIS (Geographical Information System) method: the eligible, difficult and ineligible areas have been identified. The first study for Life Cycle Assessment (LCA) of recycled concrete has been launched.

Figure 4: The first experimental project

Normally, the program will be completed at the beginning of 2016, as planned. Only some communication and promotion operations should continue after that time.

4. DEVELOP CONCRETE WITH THERMAL INSULATION PROPERTIES

4.1 Environmental background

Building sector is consuming more than 40% of the total energy in France. New environmental regulation for buildings had been implemented by French government. To fulfil the new French thermal regulation, still using traditional construction methods, it is necessary to use material with better insulation ability.

4.2 Thermal conductivity

Thermal insulation properties are characterized by thermal conductivity of the material "λ" This "λ" is the ability of the material to transmit heat, depending of time and surface. It is expressed inW/(m.K). The lower "λ" is, the more insulating properties the material has.

For a "normal" concrete, the value of λ is around 2; for concrete with thermal insulation properties, the value of λ is around 0.6.

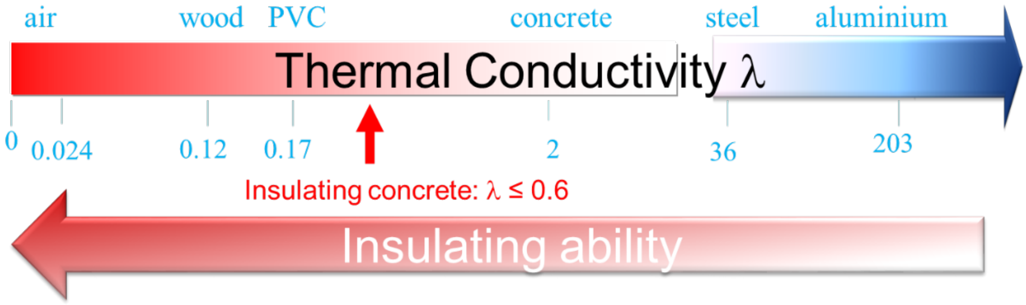


Figure 5: Thermal conductivity of materials

4.3 Thermal bridges

Thermal bridges are one of the main causes of heat losses in buildings. These heat losses generate a need for more heating of the building. Furthermore, if the impact of thermal bridges is not taken into account, the heating installation may be undersized.

Some years ago, thermal bridges generated 10 to 20% of total heat losses of the building, but nowadays as wall insulation has progressed, this percentage has increased.

To give an order of magnitude, the heat loss through a thermal bridge at the junction slab/wall may be 5 times the heat loss through the wall, and even twice the heat loss through the window!

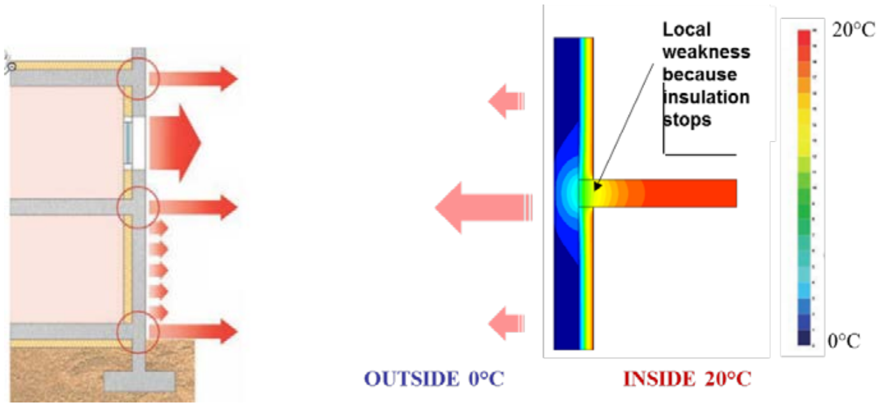


Figure 6: Thermal bridges

4.4 The concrete answer

To answer this problem, the French ready mixed concrete producers developed a new family of insulating concrete

- With a λ < 0.6 W/(m.K)
- Conforming to EN 206
- Structural concrete from LC20/22 to LC30/33

- Consistence S4 or S5
- Conforming to EC 2 and seismic rules
- Aesthetic properties similar to traditional concrete

The SNBPE is actually working on an evolution of the certification scheme of concrete plants to introduce the certification of λ.

LIFE CYCLE ASSESSMENT OF RESIDENTIAL BUILDINGS: A COMPARISON BETWEEN REINFORCED CONCRETE, STEEL AND WOOD STRUCTURES

Borroni Marco

Operations and Plant Design Director for Unical S.p.A.

Abstract

Buildings using a significant share of our resources - materials, energy, water and land - play an important part in a resource efficient economy. If we want the European building sector to become more competitive in the future, we need to obtain higher resource efficiency levels. Atecap (the Italian technical economic association for ready-mixed concrete) has conducted a LCA study in cooperation with the Department of Structural Engineering of the University of Naples “Federico II” to prove how concrete can strongly improve the whole-life performance of buildings.

The study compares the environmental impacts of a reinforced concrete (RC) building assumed to be built in Rome, Italy with the impacts of equivalent steel or wood buildings. In particular, the environmental impact has been assessed for the entire life cycle of the structural parts of the three residential buildings, having the same volumetric and architectural features and structural performances. For this purpose, the three structures have been designed on the basis of the seismic limit states of collapse and damage in accordance with the Italian Code (NTC 2008). Then the environmental impacts of the three different solutions have been assessed according to the four steps of the life-cycle assessment procedure (ISO 14040): extraction and processing of raw materials; construction, including transportation of the structural elements and materials to the building site; operation, including the maintenance phases during the building life; end of life, including structural demolition and recycling of building waste materials. The methodology used for the assessment of environmental impact was IMPACT2002+, with data about the impacts of wood and steel and their processes provided by the database Ecoinvent 3.0. In the case of concrete more refined data have been collected by using specific Environmental Product Declarations.

Keywords: Comparative life-cycle assessment, reinforced concrete structure, climate change, concrete, sustainability

1. INTRODUCTION

Atecap has promoted a comparative LCA (Life Cycle Assessment) of the environmental impacts of reinforced concrete (RC), steel and wood buildings having the same geometry and volume and subject to the same actions (accidental actions, snow and earthquake) [1]. The scope was to assess the environmental impacts of the three solutions during the construction, maintenance and end-of-life phases.

2. STRUCTURAL DESIGN OF THE BUILDINGS

The building is a civil house ideally located in the municipality of Rome, Italy (Lat. 41.9075°; Long. 12.49°; h 20 m a.s.l.). The structures were designed for the seismic limit states of collapse and damage according to the Italian Code (NTC 2008 [2]) using the design program Edilus v.26.00 [3].

The building characteristics are:

- rectangular plant (12x25) m
- three floors 300 m^2 each
- two apartments for floor, 130 m^2 each
- four hollow-core balconies (20 cm thick) for each side of the building
- (0,30x0,50) m columns, except for the first floor columns (0,40x0,60) m
- (0,30x0,50) m beams
- hollow-core concrete roof
- 30 cm thick double liner infill.

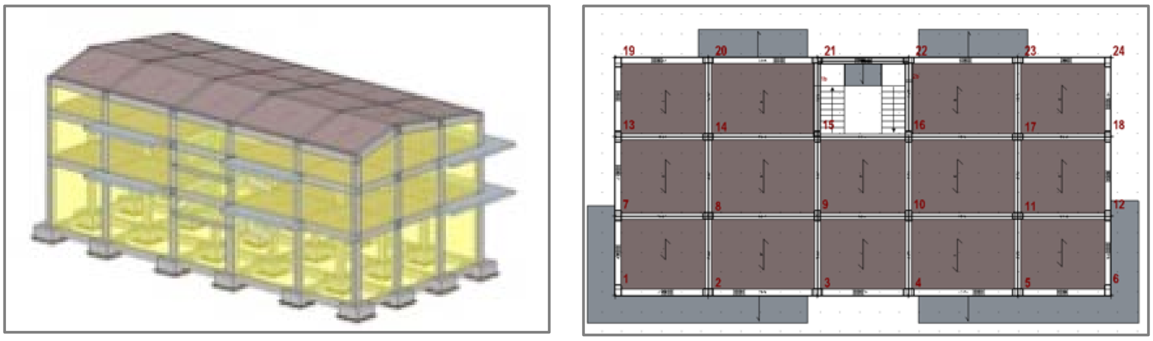


Figure 1: Edilus screen-shots of the building

For the reinforced concrete structure the use has been assumed of:

- ready-mixed concrete with strength class C 25/30, exposure classes XC1 for the out of ground structures and XC2 for the foundation;
- B450C class steel reinforcing bars.

3. THE LCA STEPS

Three different steps have been followed to develop the comparative LCA:

- the building structural design on the basis of the Italian seismic Code (NTC 2008 [2]);
- the calculation of the amount of materials used for each solution;
- the LCA analysis.

The environmental impacts have been assessed for the whole service life of the different structural parts (**from cradle to grave**), using the Swiss methodology IMPACT2002+ [4] that sums up a number of impacts into four main damage categories (see fig. 2):

- human health
- ecosystem quality
- climate change
- use of resources.

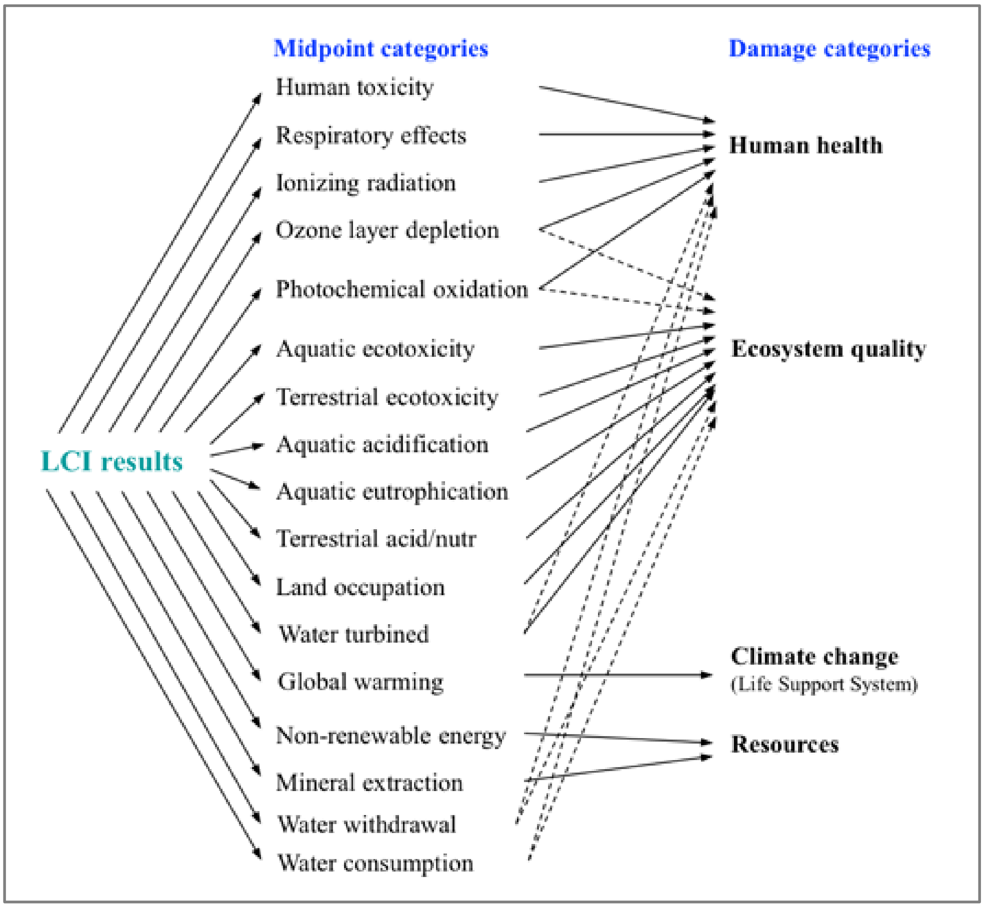


Figure 2: Impacts and damage categories scheme

The LCA has been modelled using software SimaPro 7.3 [5] and environmental impact data for the three construction materials from the Ecoinvent 3.0 database [6].

3.1 Boundary conditions

The following conditions have been assumed both for building design and impact assessment:

- 50 years Estimated Service Life for the building;
- LCA for the structural parts of the building only;
- LCA functional unit is assumed to be the whole building structure;

- building maintenance of the ordinary type only;
- “conventional” demolition of the building, with wastes recycled, incinerated or dumped according with the Italian and European percentages.

4. THE COMPARATIVE LCAs

The environmental impacts of the three structures (reinforced concrete, wood and steel) have been assessed according to the four phases of the life-cycle assessment procedure (ISO 14040 [7]): 1) extraction and processing of raw materials (production); 2) construction, including transportation of the structural elements and materials to the building site; 3) use, including maintenance during the building life; 4) end-of-life, including structural demolition and recycling of building waste materials.

In addition to the environmental impacts, SimaPro [5] calculates the so called “standardised” and “weighted” impacts of the damage categories. The standardization method (IMPACT200+ [4]) consists of comparing all the impact categories with some “reference” values - the average European data over a specific period of time. Then the standardized damage categories are multiplied by "weighting factors", which represent the critical issues of the different environmental impacts. These standardized and weighted impacts of the damage categories may be used to compare the different impacts among themselves and for the different life phases of the three buildings.

The most import issues of each phase are summarized below.

Production phase. The Ecoinvent database [6] has been partly customized to adapt it to the Italian scenario: for concrete transportation by rail and barge inputs were not considered and the type of heavy fuel oil burned for plant heating was modified. Moreover the Portland cement data were replaced by more refined data collected from specific Environmental Product Declarations (EPDs) of some Italian cement plants, provided by Aitec, the Italian cement association.

Construction process phase. This includes transport to site, and for this, a concrete production plant, a reinforcing steel production plant and a brick production plant 30 km distant from the construction site have been chosen. The steel factory and the laminated wood (glulam) supplier nearer to the construction site are respectively at a distance of 87,5 km and 249 km. The construction process phase has lower environmental impacts than the other phases.

Use phase. Building maintenance of the ordinary type has been assumed. The building resources consumption (energy, water, etc.) has not been considered within the study, because the analysis has been concerned the structural parts of the building only. About the maintenance of the reinforced concrete structure, 5% of the surface of the beams and columns exposed to air is treated with a protective varnish and the cover to reinforcement was increased with an additional 3 cm of cementitious mortar [8].

End-of-life phase. It consists of the following operations:

- mechanical demolition;
- separation of reinforcing steel from concrete and stockpiling of the waste;
- waste recycling/incineration/dumping according to the following percentages:
  - concrete: 65% recycled, 35% to landfill (data from Anpar, the Italian recycled aggregates producers association [9]);

- wood: 16% recycled, 4% to incineration, 80% to landfill (data from Trada, the Timber Research and Development Association [10]);
- reinforcing steel: 65% recycled, 35% to landfill (data from ArcelorMittal [11]);
- steel section bars: 98% recycled, 35% to landfill (data from ArcelorMittal [11]).

It has also been assumed that steel composition is 63% new iron and 37% recycled. Therefore the percentage of new steel not consumed is: 63%x – 37%x, here x is the steel's total weight (kg). This scenario, called "open loop with closed loop recycling procedure" according to ISO 14049:2000 [12], avoids the double-accounting of the recycled steel.

5. RESULTS

The Life-Cycle Assessment impacts are presented in figures 3 to 6 for the four selected damage categories. Figure 7 gives a total overview of the results.

The production process phase has the highest impact, as transportation, construction and maintenance phases' impacts are less than 10% of the production one (fig. 3).
For the three materials, the four impact categories assessment in the production phase shows that, globally, reinforced concrete has the lowest impact.
In detail, the steel structure shows the highest impact on human health, climate change and resources consumption, while the wood structure affects the ecosystem quality.
For the steel building the impacts are definitely due to the steel itself; in the case of the wood building the impact on ecosystem quality is due to the use of laminated wood.

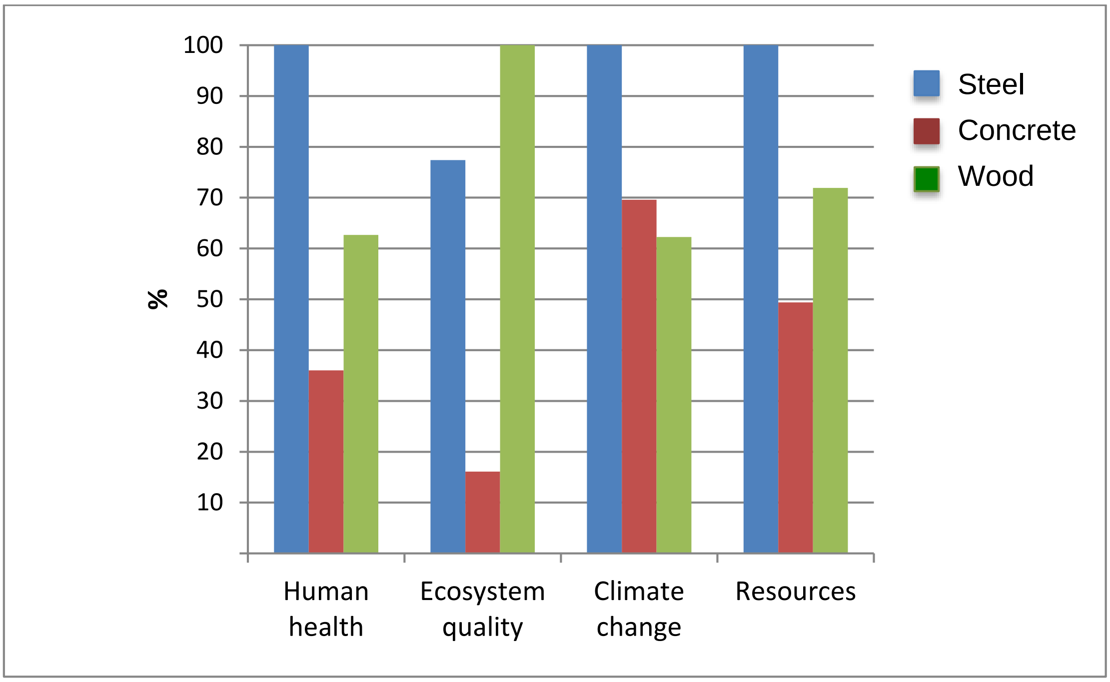


Figure 3: Production phase - impact categories

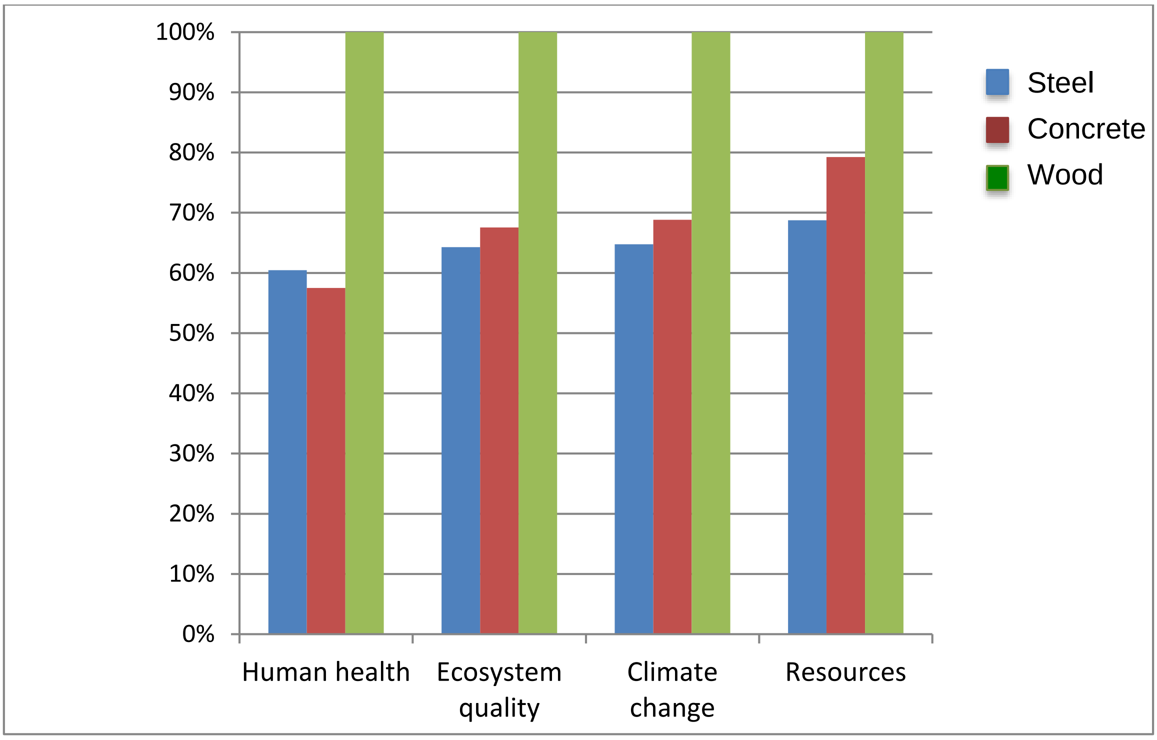


Figure 4: Transportation and construction phase - impact categories

On the whole the transportation, construction and maintenance phases show similar results as the production phase (fig. 4 – 5). The wood structure shows the highest impacts on the human health, ecosystem quality and climate change categories. This is due to the distance between the wood supplier and the construction site. In addition, the maintenance of the wood building has greater impact than that of concrete and steel, because of the high frequency of application of a wood primer.

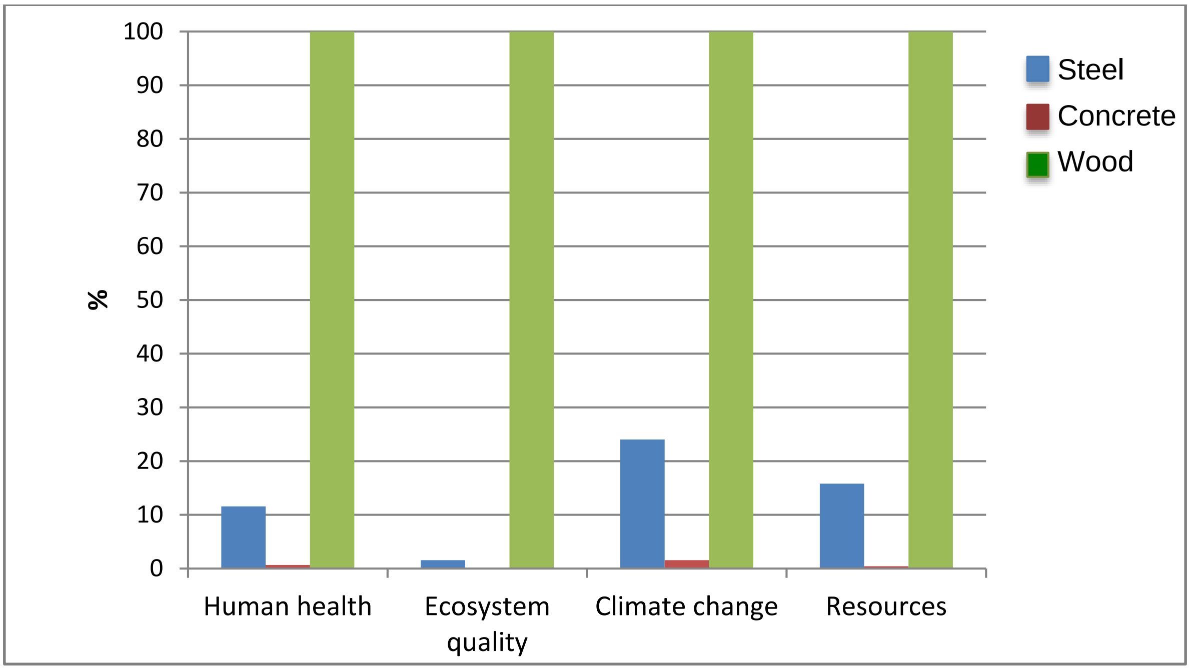


Figure 5: Use phase - impact categories

The end-of-life phases' impacts are 20-50% of those of the production phase (fig. 6). The figure below shows negative impacts for this phase, because recycling avoids the consumption of new resources. The lowest impact for all the categories in the end-of-life phase is shown by steel, because of the greater quantity of this material that can be recycled in comparison with concrete and wood.

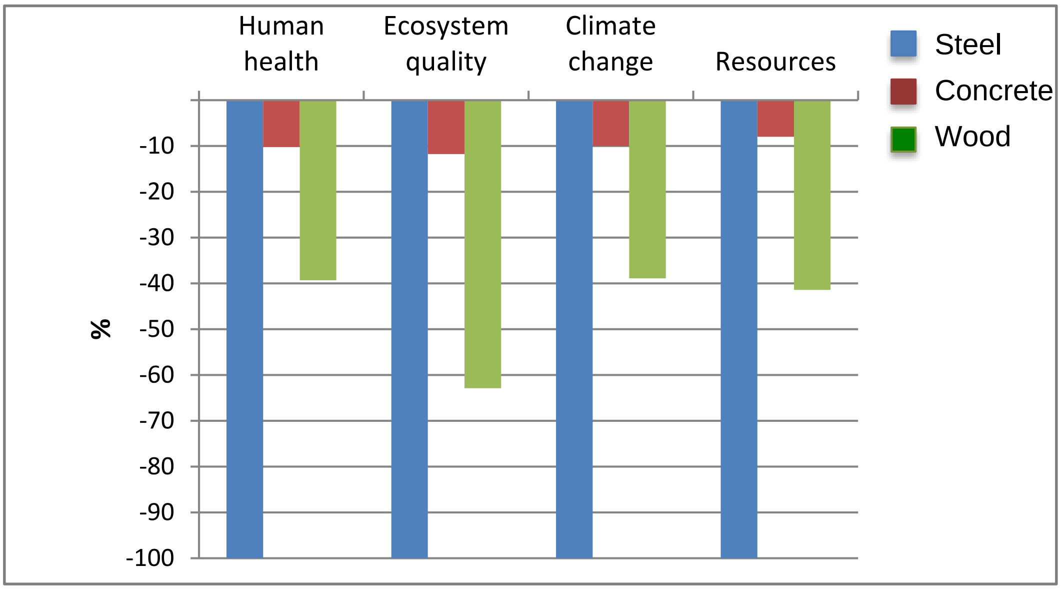


Figure 6: End-of-life phase - impact categories

Figure 7 shows the assessment of the four impact categories over the whole service life (the so-called “total impacts”) for the three solutions. The total impact values are in table 1. The reinforced concrete structure has the highest impact on climate change only; steel has the highest impact on the human health - and a high impact on climate change also; wood has the highest impact on ecosystem quality and a high impact on human health.

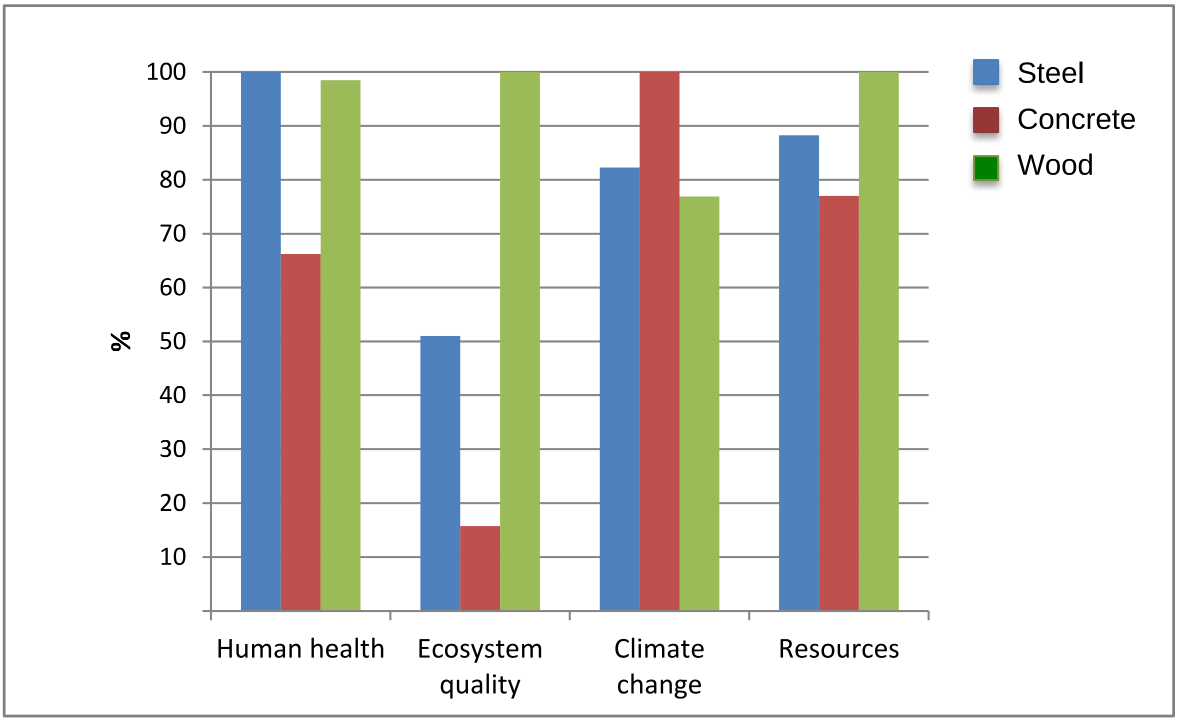


Figure 7: Building service life - impact categories

Table 1: Building service life - impact categories values

Damage category	Unit of measure	Steel	Reinforced Concrete	Wood
Human health	DALY	1,05E-01	6,98E-02	1,04E-01
Ecosystem quality	PDF*m^2*yr	4,54E+04	1,40E+04	8,90E+04
Climate change	kg CO_2 eq	9,87E+04	1,20E+05	9,23E+04
Resources	MJ primary	1,45E+06	1,26E+06	1,64E+06

6. CONCLUSIONS

There is a strong and growing interest in the environmental performance of the construction industry worldwide. As a sound, well established and cost effective structural material, reinforced concrete was and remains the preferred option for many types of buildings, but often concrete is selected exclusively for its mechanical properties, rather than for its environmental sustainability.

The comparison between three alternatives - reinforced concrete, wood and steel buildings made in the present study shows that concrete offers an optimum whole-life performance.

During the 50 years building service-life concrete has the lowest environmental impact on human health, ecosystem quality and resources consumption. These results depend in particular on concrete durability and the local availability of its raw materials (including a large proportion of secondary materials).
The LCA study promoted by Atecap concerns a specific structure, located in a given area with particular weather conditions. The results of the study look interesting and should be followed by other studies within different scenarios.

For future development it could be interesting to collect more specific (Italian) recycling data, the data used in the present analysis mainly coming from European databases. A prospective analysis could involve "green" and innovative materials also, to assess their influence on the human and environmental impact reduction.

REFERENCES

[1] "Valutazione comparativa di sostenibilità basata sull'analisi del ciclo di vita – Confronto tra gli impatti ambientali di tre edifici progettati in calcestruzzo armato, acciaio e legno", Atecap (Italian Technical Economic Association for Ready-mixed Concrete) ,Department of Structural Engineering of the University of Naples "Federico II", under publication;

[2] D.M. 14.01.2008, Norme Tecniche per le Costruzioni, Ministero delle Infrastrutture, Supplemento Ordinario alla G.U. del 04.02.2008;

[3] "Tutorial e manuale operativo Edilus", Acca Software;

[4] IMPACT2002+ User Guide for vQ2 21, Humbert S., De Schryver A., Bengoa X., Margni M., Jolliet O., 1 November 2012;

[5] SimaPro 7Introduction to LCA, Product Ecology Consultants;

[6] The Life Cycle Inventory Data Version 2.2, Ecoinvent Centre, 2007;

[7] EN ISO 14040 Environmental Management. Life cycle assessment. Principles and Framework, ISO (2006);

[8] "Analisi del ciclo di vita per la fase di posa in opera di un edificio per uso abitativo", Vitale P., Torchia A., Menna C., De Gregorio F., Asprone D, Prota A., Arena U., VIII Convegno della Rete Italiana LCA, Florence, 19-20 June 2014;

[9] L'Italia del Riciclo 2012 - Rapporto annuale sul riciclo ed il recupero dei rifiuti, Cap.13: Rifiuti inerti da costruzione e demolizione, Rapporto Anpar (Italian Association for Recycled Aggregates) 2012;

[10] "Recovering and minimising wood waste", TRADA (the Timber Research and Development Association)), TRADA Technology wood information sheet 2/3 Sheet 59', revised June 2008, 2008;

[11] Acciaio – il materiale più riciclato al mondo, Constructalia - The Steel construction website, ArcelorMittal;

[12] PD ISO/TR 14049:2000 ISO/TR 14049:2000 Environmental management - Life cycle assessment - Examples of application of ISO 14041 to goal and scope definition and inventory analysis.

CONCRETES WITH HIGH DURABILITY PERFORMANCE CONTAINING CEM III/B TYPE OF SLAG CEMENT

Mehmet Tahir Turgut (1) and Bora Yağlı (1)

(1) Oyak Beton, Turkey

Abstract

Special concrete types are being utilized in special projects designed with service lives of minimum 100 years. In this regard, concretes are designed with the same service life which is directly related with their durability properties.

In this paper, concretes with slag cement are stated and their rheological and mechanical properties are given according to the determined performance levels. In order to prevent the early age cracks caused by hydration heat development and to keep the Cl^- migration value under a very low level, it has been chosen the CEM III/B type of cement. First type of concrete is designed with the standard consistency class and the second one is designed as Self Compacting Concrete.

In addition to the stated advantages on concrete side, it can be easily mentioned about the environmental benefits of slag cement utilization in the name of sustainable development.

Keywords: Slag cement, sustainability, SCC, CO_2 emission, chloride migration.

1. INTRODUCTION

Blast furnace slag is the by-product of iron and steel production and show a perfect binding property when it is grinded separate or together and blended with Portland clinker and gypsum. The binding property of blast furnace slag was discovered by Emil Langenin Germany in 1862. Blast furnace slag production activated by lime was started in 1865 and grinding together with Portland clinker was started in 1880 in Germany. The amazing potential of this compound was recognized after its resistance to abrasive conditions. High blast furnace cement was utilized in French underground metro in 1889. [1]

Traditionally, high performance concrete (HPC) may be regarded as synonymous with high strength concrete. It is because lowering of water-to-cement ratio, which is needed to attain high strength, also generally improves other properties. However, it is now recognized that with the addition of mineral admixtures HPC can be achieved by further lowering water-to-cement ratio, but without its certain adverse effects on the properties of the material. In fact, performance can be related to any properties of concrete. It can mean excellent workability in fresh concrete, or low heat of hydration in case of mass concrete, or very quick setting and hardening of concrete in case of sprayed concrete. However, from a structural point of view, it can be understood usually that high strength, high ductility and high durability, which are regarded as the most favourable factors of being a construction material, are the key attributes to HPC. [2]

During the Izmit Bay Suspension Bridge Project Caisson and Anchorage Concrete Works implementation, it has been studied to achieve a High Performance Concrete mix design with the service life of minimum 100 years. During the mix design development, it has been decided to utilize 100% of CEM III/B type of slag cement as binder. Two types of mix designs have been performed and all the required targets were achieved with this cement type.

On the other hand, slag cements are more economical and environment friend regarding to their high additive content and low clinker utilization. They show a high performance saving from the fuel and electrical energy consumption and reduce the CO_2 emission by fuel saving in terms of sustainability. It can be seen in the following table that with the utilization of CEM III/B type of cement having a 70% of blast furnace slag has a 78% CO_2 emission reduction.

Cement Type	**CEM I**	**CEM III/A (50% GGBS)**	**CEM III/B (70% GGBS)**
Clinker ratio	90%	40%	20%
CO_2 emission content for 1 ton of cement production	900 kg	400 kg	200 kg

Transformation rate: 1 metric ton CO_2 emission for 1 metric ton of clinker

2. LABORATORY WORKS

2.1 Constituent materials

Constituent material types and sources utilized for both concrete types are given below in Table 1.

Table 1: Constituent Materials

Material	Type	Source
Cement	CEM III/B 32.5 N	OYAK Bolu Cement
Natural sand	0-1 mm	Kemerburgaz, Istanbul
Crushed aggregates	0-5 mm, 5 – 12 mm, 12-22 mm	Yalova, Turkey
Chem. Admixture	Superplasticizer	Chryso, Optima 100
Chem. Admixture	Superplasticizer	Chryso, HP 2516

2.1.1 Cement

CEM III/B 32.5 N type slag cement is utilized during all the laboratory tests, industrial tests and castings of the permanent structures. Cement is being produced by OYAK Bolu Cement Ereğli Grinding Terminal by separate grinding method. Chemical and Physical properties performed by Turkish Cement Manufacturers' Association R&D Laboratory are given in Table 2.

Table 2: Cement Analysis

Test Name	Test Method	Units	Results
Loss on Ignition	TS EN 196-2	%	1,53
$Si0_2$	XRF	%	32,38
Al_2O_3	XRF	%	8,36
Fe_2O_3	XRF	%	2,62
Ca0	XRF	%	47,53
MgO	XRF	%	4,02
SO_3	TS EN 196-2, spec.fotometer	%	1,87
Na_2O	XRF	%	0,40
K_2O	XRF	%	0,68
Na_2O Equiv. Total Alkali	-	%	0,85
Cl^-	TS EN 196-2_XRF	%	0,0042
Insoluble Residue	TS EN 196-2	%	1,13
Total Additives	TS EN 197-1	%	66,60

Table 2(cont.): Cement Analysis

Test Name	**Test Method**	**Units**	**Results**
Heat of Hydration (7 days)	TS EN 196-8	Cal/g (Joule/g)	47,3 (198)
Heat of Hydration (28 days)	TS EN 196-8	Cal/g (Joule/g)	56,4 (236,1)
Density	Digital Pycnometer	g/cm³	2,99
Specific Surface (Blaine)	Auto Blaine Instrument	cm²/g	5575
Comp. Strength_2 Days	TS EN 196-1	MPa	12,9
Comp. Strength_7 Days	TS EN 196-1	MPa	22,8
Comp. Strength_28 Days	TS EN 196-1	MPa	37,5
Normal Consistency	TS EN 196-3	%	31,8
Initial Setting Time	TS EN 196-3	min	165
Final Setting Time	TS EN 196-3	min	235
Soundness	TS EN 196-3	mm	1,0

2.1.2 Fine and coarse aggregates

Tests on aggregates for both designs have been shown in Table 3 and Table 4. Natural sand is from Kemerburgaz region of Istanbul and other crushed aggregates belong to Yalova region and type of limestone.

Table 3: Aggregate Sieve Analysis

Test Name	**Test Method**	**Units**	**Results**			
Sieve Analysis			**N.Sand**	**C.Sand**	**C.A 1**	**C.A 2**
22 mm	TS EN 933-1	%	100	100	100	100
16 mm		%	100	100	100	72
8 mm		%	100	100	61	1
4 mm		%	100	92	2	1
2 mm		%	99	62	1	1
1 mm		%	98	42	1	1
0,5 mm		%	97	34	1	1
0,25 mm		%	24	25	1	1
0,125 mm		%	1	20	1	1
0,063 mm		%	0,9	13,4	0,8	0,7

Table 4: Aggregate Properties

Test Name	Test Method	Units	Results			
			N.Sand	C.Sand	C.A 1	C.A 2
Organic Impurities	TS EN 1744-1	-	Lighter	Lighter	Lighter	Lighter
Methylene Blue Test	TS EN 933-8	MB	0,5	0,4	-	-
Drying shrinkage	TS EN 1367-4	%	0,02	0,01	0,01	0,01
Particle density	TS EN 1097-6	Mg/m^3	2,63	2,69	2,71	2,72
Water absorption	TS EN 1097-6	%	1,0	0,8	0,5	0,4
Flakiness index	TS EN 933-3	%	-	-	10	8
Shape index	TS EN 933-4	%	-	-	1	3
Los Angeles coefficient	TS EN 1097-2	%	-	-	27	
Water Sol. Cl ions	TS EN 1744-1	%	0,02	<0,001	<0,001	<0,001
Water Sol.Alkali Content_(eqv. Na_2O)	ASTM C114-05	%	0,01	<0,01	<0,01	<0,01
Magnesium Sulfate Loss Value	TS EN 1367-2	%	-	-	1,0	1,0
Acid soluble sulphate as SO_3	TS EN 1744-1	%	0,04	0,02	0,01	0,01
Pot. Reactivity_(14 days) (50%blended)	CSA A23.2-25A	%	0,08		0,01	0,01

2.1.3 Chemical admixtures

In both designs slump keeping superplasticizer type of chemical admixtures have been used. It has been requested minimum 120 minutes slump retention. As a result of Self Compacting Concrete needs different chemical admixtures were chosen for designs. The chemical admixtures Chryso Optima 100 is chosen for DURABET® PLUS and Chryso HP 2516 for DURABET®SELF. These superplasticizers work as new generation superplasticizers based on modified phosphonate.

2.1.4 Concrete mix designs

During the preliminary laboratory trials, it has been decided to be utilized one type of slag cement in both concrete types. Regarding the employer's requirements the w/c ratio was kept 0,38 for conventional consistency class (target slump value: 210 mm) concrete and 0,35 for self-compacting concrete (target flow value: 720 mm). Mix design properties are stated in Table 5.

Table 5: Mix design properties

Property/Material	**DURABET® PLUS**	**DURABET® SELF**
Concrete strength class	C45/55	C45/55
Max. aggregate size*	22 mm	12 mm
Consistency, Slump	180mm<…<240mm	-
Consistency, Slump Flow	-	660mm<… <780 mm
t_{500}	-	VS2, $\mathbf{t_{500}} \geq 2,0$ s
w/c ratio	0,38	0,35
Cement Content	380 kg/m^3	475 kg/m^3
Natural Sand, 0-1 mm	427 kg/m^3	515 kg/m^3
Crushed Sand, 0-5 mm	474 kg/m^3	441 kg/m^3
C.Aggregate, 5-12 mm	382 kg/m^3	799 kg/m^3
C.Aggregate, 12-22 mm	614 kg/m^3	-
Superplasticizer	1,5%, on cement	0,95%, on cement
Water	140 kg/m^3	163 kg/m^3

3. TEST RESULTS

3.1 Fresh concrete test results

In this stage of testing process, target concretes with required properties have been produced at concrete batching plant and the fresh concrete properties are stated in Table 6 and Table 7. Industrial testing method was chosen in order to simulate the real casting ambient and conditions.

Table 6: DURABET® PLUS Fresh Concrete Tests

Test	$\mathbf{T_0}$	**100 min**	**180 min**
Fresh Conc. Temp.	25,4ºC	26,6 ºC	30,5 ºC
Slump	200 mm	230 mm	230 mm
Density	2438 kg/m^3	2462 kg/m^3	2451 kg/m^3
Air Content	2,0%	1,8%	2,1%

Table 7: DURABET® SELF Fresh Concrete Tests

Test	$\mathbf{T_0}$	**60 min**	**120 min**
Fresh Conc. Temp.	25,1 ºC	24,7 ºC	23,7 ºC
Slump Flow	670 mm	690 mm	720 mm
T_{500}	5,9 s	6,2 s	5,3 s
U box	15 sec- 34 cm		15 sec- 33,5 cm
Density	2435 kg/m^3	2442 kg/m^3	2420 kg/m^3
Air Content	1,8%		1,9%

3.2 Hardening and hardened concrete test results

It can be easily seen in Table 8 and Table 9 that the compressive strength values are reflecting much more than C45 class of concrete related to the limited w/c ratio and cement content in order to gain the required durability properties. All the compressive strength, tensile strength and E-modulus test results are derived from 15*30 cm cylinder samples. The main criteria for the minimum 100 years of service life of concrete mix design is limiting the Chloride migration coefficient with the value of $3x10^{-12}$ m^2/s with CEM III/B type of cement utilization and reducing the early age cracking related to the adiabatic heat development.

Table 8: DURABET® PLUS Hardening and hardened concrete test results

Test	Unit	Days	Value
Compressive Strength, *TS EN 12390-3*	**MPa**	0,5	4
		1	17
		2	30
		7	53
		14	72
		28	78
Splitting Tensile Strength, *TS EN 12390-6*	**MPa**	0,5	0,5
		1	2,1
		2	2,8
		7	4,5
		14	5,4
		28	6,1
E-Mod., E_o, *NT BUILD 205*	**GPa**	0,5	11
		1	25
		2	30
		7	40
		14	46
		28	48

Test	Unit	Days	Value
Adiabatic Heat Development *NT BUILD 388*	**(kJ/kg)**	0,5	2,3
		1	23,7
		2	76,7
		3	107
		7	167
		14	205
Thermal Expansion Coefficient, *TI-B 101*	**[$*10^{-6}/^oC$]**	7	7,27
Chloride Migration Coefficient *(Core Nominal Diameter 100 mm),* *NT BUILD 492*	**m^2/s**	28	$1,14x10^{-12}$
		56	$0,70x10^{-12}$

Table 9: DURABET® SELF Hardening and hardened concrete test results

Test	Unit	Days	Value
Compressive Strength, *TS EN 12390-3*	**MPa**	0,5	3
		1	16
		2	30
		7	60
		14	77
		28	83
		56	93
Splitting Tensile Strength, *TS EN 12390-6*	**MPa**	0,5	0,3
		1	1,8
		2	2,9
		7	4,2
		14	6
		28	6,5
		56	6,8
E-Mod., E_o, *NT BUILD 205*	**GPa**	0,5	19,5
		1	22
		2	27
		7	42
		14	44
		28	47
		56	48
Adiabatic Heat Development *NT BUILD 388*	**(kJ/kg)**	0,5	19
		1	93
		2	138
		3	165
		7	212
		14	216
Thermal Expansion Coefficient *TI-B 101*	**[$*10^{-6}$/ºC]**	7	7,2
Chloride Migration Coefficient *(Core Nominal Diameter 100 mm), NT BUILD 492*	**m^2/s**	28	$0,68x10^{-12}$
		56	$0,51x10^{-12}$

4. CONCLUSION

- Slag addition (substitution with clinker 70%) has a high contribution on 28 days and 56 days compressive strength values. It is achieved to overcome the 90 MPa 56 days compressive strength value with Self Compacting concrete and to overcome the 75 MPa value at 28 days with conventional consistency concrete.

- Depending on the w/c ratio and cement content, Chloride migration values are controlled and vary from $0,51x10^{-12}$ m^2/s to $0,70x10^{-12}$ m^2/s under the expected durable behaviour.

- It has been achieved to reduce the 14 days adiabatic heat development values under the 225 kj/kg value for both mixes with the positive contribution of high slag content.

REFERANCES

[1] D.Higgins, D., “Ground Granulated Blast Furnace Slag”, World Cement, N.73, 1995.

[2] Oral, B., Denvid, L.; “High Performance Concrete: Fundamentals and Application”; Department of Civil and Environmental Engineering, Massachusetts Institute of Technology.

EXAMINATION OF CHLORIDE MIGRATION OF DIFFERENT CONCRETE MIX DESIGNS WITH DIFFERENT BINDER COMBINATIONS AND WATER TO BINDER RATIOS OVER TIME

Göktuğ Aktaş (1), Arda Işık (2), Cenk Kılınç (2), Tümer Akakın (2), Seyhan Satılmış (3)

(1) Akçansa San. ve Tic. A.Ş., Turkey

(2) Türkiye Hazır Beton Birliği (THBB), Turkey

(3) İC İçtaş İnşaat ve Astaldi Ortak Girişimi (İCA), Turkey

Abstract

Durability based designs have gained priority at infrastructure and superstructure construction projects nowadays. Low permeability concrete, corrosion inhibitors, protective coating to the steel reinforcement, waterproof coatings, cathodic protection and even combinations thereof can be used for enhancing the durability properties of the structure. As important examples, in The Third Bosphorus Bridge and The Izmit Bay Bridge Projects, along with other precautions, chloride migration level of concrete has been limited for assuring a service life of 100 years, which has been predicted using various simulation programs.

In this study, chloride migration, which is one of the most important criteria for concrete durability will be examined at different ages of concretes containing different combinations of mineral binders and water/binder ratios with also concentrating on the principles of chloride migration theory.

1. INTRODUCTION

While designing structures, strength and durability of materials should be considered. In addition, based on exposure conditions, service life should be calculated for a good life cycle estimation. A long service life requires a concrete that can resist ambient conditions in other terms, a durable concrete. One of the main components of the durability is impermeability. Whenever the concrete is less permeable, it is less prone to deterioration due to environmental conditions. Concrete permeability can be affected by binder content, water to binder ratio, combined aggregate grading, compactability ratio, curing conditions and several other parameters.

One of the test methods that can be used for permeability determination is rapid chloride migration test. Rapid chloride migration test provides valuable information on service life determination. Softwares such as STADIUM or LIFE365 use rapid chloride migration as a tool for prediction.

Aim of this study is to determine effect of binder compositions and water to cement ratio on rapid chloride permeability.

2. LITERATURE REVIEW

In addition to ensuring impermeability and lowering the water to cement ratio, using pozzolanic materials is also an effective method for enhancing the durability of concrete subjected to the sea water. Pozzolanic additions also increase the amount of aluminates in the binder matrix. Since chloride ions are chemically binded with aluminate components, increasing the pozzolans prevents the migration of chloride ions deeper into the concrete [1].

Some researchers relate the finer particle distribution of pozzolans to a better hydration development. Han et al. (2007) found that increasing the C_3A content increases Friedel Salt formation capacity and that decreases the migration of chloride ions. It should also be noted that the chloride ion binding capacity of slag is provided by the hydration of aluminate containing components of the glass phases [2].

Chindaprasirt and others found that fly ash decreases the rapid chloride permeability of concrete and related that to the decrease in average pore diameter and denser interfacial transition zone characteristics [3].

Dhir et al. suggested that high binding capacity of cement-slag paste may be due to the high alumina content in slag, resulting in the formation of more Friedel’s salt. In addition, amount of C-S-H gels is usually found to be higher than ordinary Portland cement mixes therefore a denser and less permeable structure is formed. In addition to that, C-S-H gels physically bound some amount of chloride into their structure [4].

Addition of silica fume to concrete decreases the permeability by filling capillary voids due to very small particle size [5].

3. EXPERIMENTAL STUDIES

3.1 Materials

Different types of binders and two different supplementary cementing materials (SCM) have been used for this study. Properties of the binders are given in the Table 1. Material type and the related standard are given in the Table 2.

Table 1: Properties of binders, slag and supplementary cementing materials (SCM)

CEMI 42.5R		CEMIV 42.5R		SLAG	
LOI (%)	2.1	LOI (%)	5.3	LOI (%)	0.3
MgO (%)	1.2	MgO (%)	1.6	MgO (%)	6.3
SO_3 (%)	3.1	SO_3 (%)	2.8	SO_3 (%)	0.3
Cl^- (%)	0.0	Cl^-(%)	0.0	CaO (%)	38.4
SiO_2	19.4	SiO_2	23.0	SiO_2	38.4
Insoluble Residue (%)	2.1	Insoluble Residue (%)	5.3	Al_2O_3 (%)	12.2
2. Day Compressive Strength(MPa)	27.5	2.Day Compressive Strength(MPa)	23.8	(CaO+MgO)/ SiO_2	1.2
28. Day Compressive Strength(MPa)	51.7	28.Day Compressive Strength(MPa)	50.5	45 micron residue (%)	-
Setting Time (Start./Finish.)	88 / 147	Setting Time (Start/Finish)	126 / 190	7. Day Activity (%)	52
Soundness (mm)	1	Soundness(mm)	1	28. Day Activity (%)	79
Blaine (cm^2/g)	3480	Blaine (cm2/g)	4030	Blaine (cm^2/g)	5350
Density (g/cm^3)	3.14	Density (g/cm^3)	3.06	Density (g/cm^3)	2.8

FLY ASH		MICROSILICA	
LOI (%)	2.4	LOI (%)	0.9
MgO (%)	0.9	Na_2O (%)	0.2
SO_3 (%)	-	K_2O (%)	0.6
CaO (%)	0.7	SO_3 (%)	0.1
$SiO_2+Al_2O_3+Fe_2O_3$	92.6	CaO (%)	0.3
Cl^-(%)	0.03	SiO_2	96.4
Na_2Oeq (%)	2.2	Na_2Oeq (%)	0.6
45 micron residue (%)	16.1	45 micron residue (%)	-
28. Day Activity (%)	74	7. Day Activity (%)	-
90. Day Activity (%)	86	28. Day Activity (%)	114
Blaine (cm^2/g)	-	Blaine (cm^2/g)	209000
Density (g/cm^3)	2.3	Density (g/cm^3)	2.2

Table 2: Material type and related standard

	Material Type	Standard
Binder	CEM I 42.5R Cement	EN 197-1
Slag	GGBS from Bolu Ereğli	EN 15167-1
SCM	Fly Ash	EN 450-1
SCM	Silica Fume	EN 13263-1
Binder	CEM IV 42.5 Cement	EN 197-1
Aggregate	Fossiliferous Limestone	EN 12620
Aggregate	Silica Sand	EN 12620
Water	Potable Water	EN 1008
Admix 1	Lignin based Water Reducer	EN 934-2
Admix 2	Naphthalene and Lignin based water reducer	EN 934-2
Admix 3	Polycarboxylate based water reducer	EN 934-2

Natural silica sand and crushed limestone are used as fine and coarse aggregate (2 types), respectively. Sieve analysis of aggregates is presented in Table 3 and physical properties of aggregates are given in Table 4.

Table 3: Sieve analysis of aggregates

% PASSING													
Material Type	31.5 mm	22.4 mm	19 mm	16 mm	11.2 mm	8 mm	5.6 mm	4 mm	2 mm	1 mm	0.5 mm	0.25 mm	0.125 mm
Crushed Sand	100	100	100	100	100	100	100	99	95	62	34	24	12
Coarse 1	100	100	100	100	92	65	34	3	1	1	1	1	1
Coarse 2	100	100	79	73	55	3	1	1	1	1	1	1	1
Silica Sand	100	100	100	100	100	100	100	100	99	90	77	26	3

Table 4: Physical properties of aggregates

Material Type	$CaCO_3$ (%)	Sand Eq. (%)	Meth. Blue (ml/g)	<0,063 mm (%)	SSD Weight g/cm^3	Water Absorption (%)	Flakiness (%)	Los Angeles Abrasion (%)
Crushed Sand	95	59	1	8	2.69	1.1	-	-
Coarse 1	98	-	-	1	2.70	0.7	11	19
Coarse 2	99	-	-	1	2.71	0.5	9	19
Silica Sand	-	90	1	1	2.64	1.6	-	-

3.2 Concrete mixture design and fresh concrete test results

For the study, twelve different mix designs have been produced with combination of four binders at three water to binder ratios. Slump, flow and unit weight measurements were made according to EN 12350-2, EN 12350-8 and EN 12390-7, respectively. Coefficient of slag, fly ash and micro silica for water to binder ratio calculations are taken as 1 – 0.4 and 2 respectively. Slump was held constant at 200 mm for all (0.45 and 0.53) water to binder ratio mix designs and 550 mm flow was observed for 0.37 w/b ratio mix designs. Fresh concrete properties and mixture designs have been presented in table 5.

Table 5: Fresh concrete properties and mixture designs

Mix ID	CEMI 42.5R (kg)	SCM (kg)	Microsilica (kg)	EQ. Binder (kg)	Total Binder (kg)	Admix Type	Admix Amount	Slump /Flow (mm)	Unit Weight (kg/m^3)
Mix S 37	280	120	-	400	400	Admix3	1.05%	550	2528
Mix S 45	259	111	-	370	370	Admix2	1.52%	200	2477
Mix S 53	224	96	-	320	320	Admix1	1.50%	200	2453
Mix F 37	345	148	-	345	493	Admix3	1.20%	550	2447
Mix F 45	320	137	-	320	457	Admix2	1.78%	200	2458
Mix F 53	275	118	-	275	393	Admix1	1.82%	200	2448
Mix F+MS 37	310	133	24	411	467	Admix3	1.40%	550	2447
Mix F+MS45	300	130	23	398	453	Admix2	1.60%	200	2372
Mix F+MS 53	250	107	18	329	375	Admix1	1.73%	200	2413
Mix CEMIV 37	-	400	-	-	400	Admix3	0.90%	550	2507
Mix CEMIV 45	-	370	-	-	370	Admix2	1.60%	200	2442
Mix CEMIV 53	-	340	-	-	340	Admix1	1.70%	200	2440

3.3 Hardened concrete properties and test results

Concrete samples for compressive strength testing were air cured for 72 hours and then cured in water at 20±2 °C for 25, 53 and 87 days according to TS EN 12390-2. 100 × 100 mm concrete cubic samples were used for the compressive strength tests. Compressive strength was determined according to EN 12390-3.

Chloride migration tests were performed according to NT Build 492 using 100 mm diameter, 50 mm height cylinders. This test is aimed to determine the tendency of concrete to transport chloride ions. Concrete samples were air cured for 72 hours and then cured in water at 20±2 °C for 25 days. After initial curing, samples were taken out of water for sample preparation. After 28 days, samples were air cured for 55 and 89 days until tests.

In order to obtain 50 ± 2 mm thick slices as required by the standard, 100 × 200 mm cylinders were cast. Test specimens were prepared by cutting the cylinders into two halves and then cutting a 50 ± 2 mm thick slice from halves. The end surface that was nearer to the first cut is the one to be exposed to the chloride solution. After cutting, test samples were placed inside a vacuum container for vacuum treatment. After maintaining 10-50 mbar vacuum for three hours, while vacuum pump was still running, container was filled with saturated $Ca(OH)_2$ solution to immerse all the specimens. After vacuuming for one more hour in the container, air was allowed into the container and samples were stored in the solution for 18±2 hours. After the initial preparation, sides of samples were tightly covered with silicone rubber and upper side of the sample was treated with 0.3 M NaOH solution. Afterwards, the sample covered with rubber and filled with NaOH solution was placed into a container filled with 10 % NaCl solution. Then, the sample was subjected to DC voltage. Based on the initial current, the test period was determined. During the test, NaOH solution was used as anolyte. The anolyte helps the transfer of chloride ions in the concrete. After the tests, samples were split into two halves, while the faces of the samples were treated with $AgNO_3$. When $AgNO_3$ and Cl^- ions chemically interact, faces of the samples change color and penetration depth of Cl^- ions is measured. [6]

3.4 Chloride migration tests

Compressive strength test results are given in Table 6. Chloride migration test results are presented in Table 7.

Table 6: Compressive strength test results

Mix Design	Compressive Strengths (MPa)				Improvement		
	Days				Days		
	7	28	56	90	7/28	28/56	28/90
Slag w/b:0.37	59.8	75.8	81.2	88.4	27%	7%	17%
Slag w/b:0.45	43.6	62.1	68.6	79.9	42%	10%	29%
Slag w/b:0.53	26.6	49.8	58.7	67.0	87%	18%	35%
Fly Ash w/b:0.37	53.7	69.8	83.5	82.9	30%	20%	19%
Fly Ash w/b:0.45	39.7	54.1	66.2	66.2	36%	22%	22%
Fly Ash w/b:0.53	31.8	50.2	62.7	64.7	58%	25%	29%
Fly Ash+Microsilica w/b:0.37	56.0	82.9	87.5	88.9	48%	6%	7%
Fly Ash+Microsilica w/b:0.45	33.7	52.2	59.9	62.0	55%	15%	19%
Fly Ash+Microsilica w/b:0.53	28.7	47.5	51.7	55.6	66%	9%	17%
CEMIV w/b:0.37	50.2	65.2	70.1	72.7	30%	8%	12%
CEMIV w/b:0.45	34.3	46.6	51.3	55.3	36%	10%	19%
CEMIV w/b:0.53	24.0	36.1	39.8	41.2	50%	10%	14%

Table 7: Chloride migration test results

Mix Design	Chloride Migration (x10^{-12} m^2/s)			Improvement	
	Days			Days	
	28	56	90	28/56	28/90
Slag w/b:0.37	3.4	1.5	1.6	55%	54%
Slag w/b:0.45	5.6	4.4	4.0	23%	30%
Slag w/b:0.53	6.0	5.1	5.0	15%	16%
Fly Ash w/b:0.37	7.2	4.0	2.7	44%	63%
Fly Ash w/b:0.45	15.7	12.1	7.8	23%	50%
Fly Ash w/b:0.53	11.5	11.7	11.8	-2%	-3%
Fly Ash+Microsilica w/b:0.37	3.7	2.0	1.6	46%	58%
Fly Ash+Microsilica w/b:0.45	10.2	8.4	8.1	18%	20%
Fly Ash+Microsilica w/b:0.53	9.6	8.5	8.1	11%	15%
CEMIV w/b:0.37	8.1	7.6	7.2	7%	12%
CEMIV w/b:0.45	14.8	15.8	25.0	-7%	-
CEMIV w/b:0.53	18.5	25.0	25.0	-	-

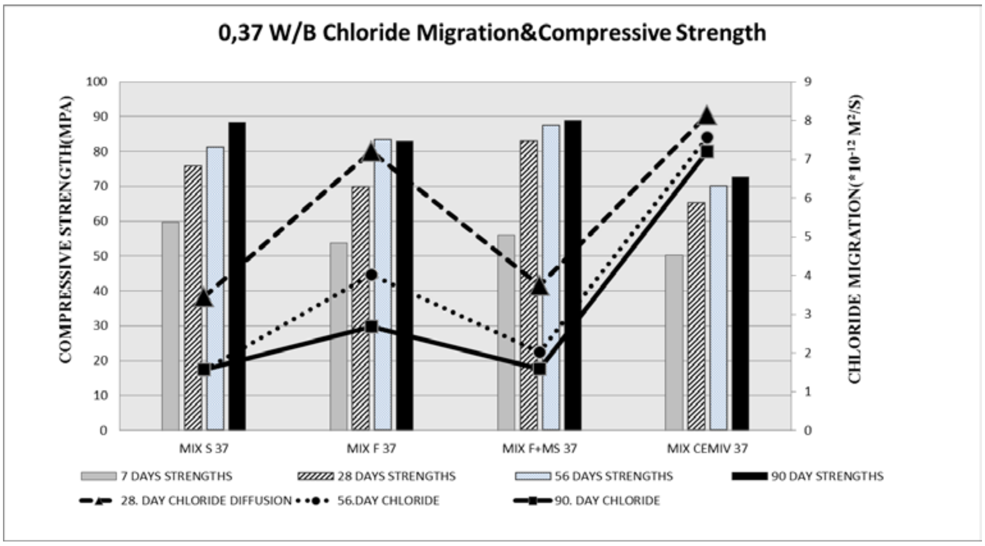


Figure 1: Chloride migration and compressive strength test results for W/B of 0.37

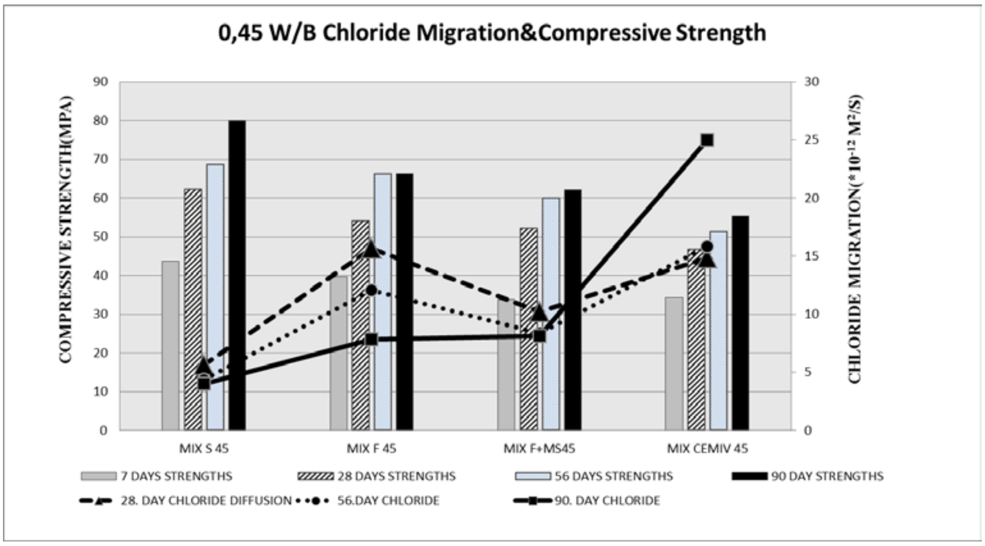


Figure 2: Chloride migration and compressive strength test results for W/B of 0.45

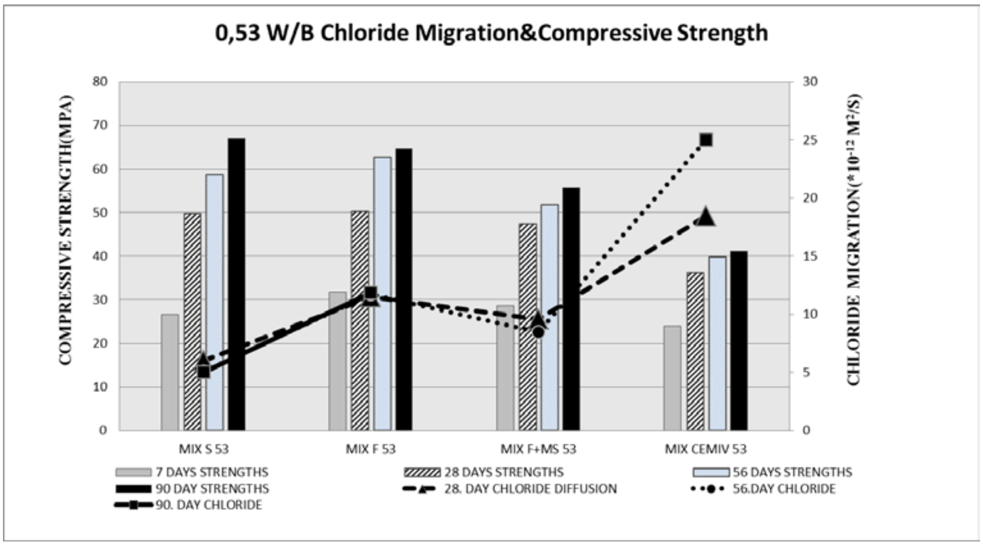


Figure 3: Chloride migration and compressive strength test results for W/B of 0.53

4. GENERAL CONCLUSIONS

- For mixtures containing slag, having W/B ratios between 0.37 and 0.45 and 0.53 chloride migration values decreased by 55%, 23% and 15%, respectively from 28 to 56 days.
- There was no significant difference between chloride migration values of slag samples for all w/b ratios from 56 to 90 days. This can possibly be due to the test method used in this study not being sensitive enough to make a distinction between those concrete designs.
- Chloride migration values decreased for fly ash samples with w/b ratios of 0.37 and 0.45 by 44% and 23%, respectively from 28 to 56 days.

- Chloride migration values for fly ash samples with 0.37 and 0.45 w/b ratios have decreased by 63% and 50% from 28 to 56 days. It can also be concluded that fly ash causes decrease in chloride permeability.
- Despite the high performance at 0.37 and 0.45 w/b at latter ages, at 0.53 w/b there is no significant change in chloride permeability.
- Chloride permeability for Fly ash+Silica fume samples at 0.37 – 0.45 and 0.53w/b samples have decreased by 58%, 20%, 15% from 28 to 90 days.
- Chloride permeability of CEMIV with 0.37 w/b decreased by 12% from 28 to 90 days. However chloride permeability for 0.45 and 0.53 mix designs were so high that chloride ions transferred directly to the anolyte solution. Therefore tests have to be stopped.It can be said that, high water to binder ratio contributed to that outcome.
- However, compressive strengths of fly ash series and fly ash+microsilica series did not improve in parallel with chloride migration as the slag series did between 56 and 90 days. Therefore, we can conclude that there is no direct correlation between strength development and chloride permeability for the tested mix designs.

REFERENCES

[1] Baradan, B., Yazıcı, H. and ÜN, H., 'Beton ve Betonarme Yapılarda Kalıcılık (Durabilite)', İstanbul, September, 2010, 189-191.
[2] Han, S. H., 'Influence of diffusion coefficient on chloride ion penetration of concrete structure', Construction and Building Materials, September, 2005, 370–378.
[3] Chindaprasirt, P., Chotithanorm, C., Cao, H. T. and Sirivivatnanon, V., 'Influence of fly ash fineness on the chloride penetration of concrete', Construction and Building Materials, September, 2005, 356–361.
[4] Yuan, Q., Shi, C., Schutter, G. D., Audenaert, K., Deng, D. 'Chloride binding of cement-based materials subjected to external chloride environment – A review.', Construction and Building Materials, March, 2008, 1-13.
[5] Düzbasan, S., Uluöz, S., Yakıt, E., 'Deniz yapılarında mikro silika kullanılması', 191-200.
[6] NT BUILD 492, 'Concrete, Mortar And Cement-Based Repair Materials: Chloride Migration Coefficient From Non-Steady-State Migration Experiments', Northtest, Finland, November, 1999.

EFFECT OF MINERAL ADMIXTURES ON SHORT AND LONG TERM ASR TEST RESULTS

Gülden Keskin (1), Yılmaz Akkaya (2), Mehmet Ali Taşdemir (2), Tahir Turgut (3) and Hasan Açık (3)

(1) Yapı Merkezi - SK Engineering & Construction Joint Venture,İstanbul Strait Road Tube Crossing Project, Turkey

(2) İstanbul Teknik Üniversitesi, İnşaat Fakültesi, Turkey

(3) Oyak Beton A.Ş., Turkey

Abstract

The petrographic properties of the aggregates have significant roles on the long term performance of concrete. Therefore physical, chemical, petrographic properties and alkali silica reactivity (ASR) of the aggregates affect the durability of concrete and service life of reinforced concrete structures.

In this research, physical, chemical and petrographic properties of the aggregates, sampled from Cendere Valley quarry and Çatalca quarry, were determined. Also, ASR risks of these aggregates were investigated by short and long term tests. Twelve different mortar mixes were designed according to ASTM C 1260 and TI B 51 and stored 14 days in 80^{o}C NaOH and 20 weeks in 50^{o}C NaCl, respectively. Change in length of the mortar bars were measured. Thin sections were prepared from the mortars and microscopic observations were carried out. Experimental expansion results were compared with ASR activity on the thin sections.

It is concluded that ASR risk investigations on the thin sections is as important as ASR length change measurements, physical, chemical and petrographic investigations. Use of mineral admixtures such as blast furnace slag in the cementations phase, has been found to be very useful in mitigation of alkali silica reactivity.

Keywords: Alkali silica reaction (ASR), durability, concrete, aggregate, mineral admixture

1. INTRODUCTION

In this research, Alkali Silica Reactivity (ASR) of Cendere quarry and Çatalca quarry aggregates were investigated. Alkali Silica Reactivity (ASR) in concrete is a particular variety of chemical reaction within alkali hydroxides, usually delivered from the alkalis present in the cement used, and reactive forms of silica present in aggregate particles. This chemical reaction also requires water for it to produce the alkali silica gel reaction product which swells with the absorption of moisture. Typical deleterious features of ASR in concrete structures include cracking, expansion consequent misalignment of structural elements, spalling of fragments of surface concrete as pop-outs and the presence of gel in fractures or associated with aggregate particles within the concrete.

ASR is a slow developing reaction. The actual determination of the potential alkali reactivity of aggregates is often only possible through laboratory tests, which ensure that all aggregates are evaluated under the same conditions. For that reason there are many methods for testing ASR of aggregates. In this research, ASTM C 1260, TI B 51, TI B 52 and ASTM C 295 are used.

One of the most important benefits of the mineral admixture is on durability and permeability of concrete. For reducing the risk of ASR, fly ash, blast furnace slag, silica fume and/or metakaolin can be used as partial cement replacement.

Among other benefits of mineral admixtures, such as blast furnace slag, in concrete is the reduction of temperature development and internal-external temperature difference in hydrating concrete. Mineral admixtures also reduce the harmful effects of sulphates and chlorides in concrete durability.

2. MATERIALS

Physical, chemical and petrographic properties of the aggregates were determined. Natural and crushed sands were sampled from Çatalca Valley quarry, and Cendere quarry, respectively.

For twelve different mortar mixes CEM I 42,5 R cement, ground granulated blast furnace slag and fly ash were used in the binder phase.

2.1 Aggregates

In Table 1 grading (TS EN 933-1) and Fineness Modulus of the aggregates are given. In Table 2 physical properties, in Table 3 chemical properties and methylene blue test results of the aggregates and related standards are given.

Table 1: Grading and fineness modulus of the aggregates

	Sieve (mm)								Fineness Modulus
	8	4	2	1	0,5	0,25	0,125	0,063	
	% Total Passing on Sieve								
F1-Çatalca	100	100	99	97	67	3	1	0,6	3,3
F2-Cendere	100	91	54	31	18	9	4	2,1	4,9

Table 2: Physical properties of the aggregates

	Physical Test Results		
	Loose Bulk Density (gr/cm^3)	Particle Density (SSD) (gr/cm^3)	Water Absorption (%)
	TS EN 1097-3	TS EN 1097-6	
F1-Çatalca	1,5	2,62	1,1
F2-Cendere	1,5	2,70	0,9

Table 3: Chemical properties and methylene blue test results of the aggregates

	Chemical Test Results			Methylene Blue (%)
	Water Soluble Chloride Content (%)	Water Soluble Alkali Content (eqv. Na_2O) (%)	Acid Soluble Sulphates (%)	
	TS EN 1744-1 @7.0	ASTM C 114-05 @17.2	TS EN 1744-1 @12	TS EN 933-9
F1-Çatalca	0,0002	0,0045	0,19	0,5
F2-Cendere	0,0001	0,0064	0,21	0,5

The content of organic impurities in the aggregates were tested according to the TS EN 1744-1. Test results show us that, the aggregates do not contain considerable amounts of organic impurities.

Petrographic properties of aggregates were determined according to ASTM C 295.

Visual examination of F1-ÇATALCA natural sand indicated that the particles were sub-rounded to rounded with medium sphericity and particle surfaces were mostly smooth textured. The individual grains of natural sand were glassy - transparent, dull white, red - brown - yellow, gray - black, green colors. Whole sample color was yellowish. On some sand grains clay lumps and friable particles were observed. Samples did not give reaction with HCl %10 acid test.

Visual examination of F2-CENDERE crushed sand indicated that the particles were angular to very angular with low sphericity. Particle surface was mostly rough textured and well packed. Laminations were observed on some grains. Crushed sand color was dark gray -gray (dry), dark gray, black (wet). Particle structure is fresh and dense, and can be classified as iron hydroxide containing sedimentary clastic rock, sandstone.

For determining the petrographic properties of the aggregates, thin sections were produced with fluorescent epoxy. These thin sections were analyzed under polarized microscope by yellow and blue filters.

2273 grains of F1-ÇATALCA natural sand were analyzed and in Table 4 petrographic analysis result is given.

Table 4: Petrographic analysis of F1-ÇATALCA natural sand

Constituents	(%)
Quartz	52
Potentially Deleterious Quartz	Trace
Chert	5
Metamorphic Rock	Trace
Plutonic Rock	3
Volcanic Rock	Trace
Sedimentary Rock	14
Carbonate Fragments	7
Feldspar	12
Mafic Minerals	6
Iron hydroxide containing grains	1

F1-ÇATALCA natural sand is also analyzed according to TI B 52. In Table 5 the analysis result is given.

Table 5: TI B 52 Analysis result of F1-ÇATALCA natural sand

Constituents	(%)
Dense - Chalcedon Chert	4
Porous - Chalcedon Chert	1,4
Opaline Chert	0
Total Alkali Reactive Material Content (Porous + Opaline Chert)	1,4
Total Number of Observation Points	2661

2391 grains of F2-CENDERE crushed sand were analyzed and in Table 6 petrographic analysis result is given. Organic matter stained components did not observed

Table 6: Petrographic analysis of F2-CENDERE crushed sand

Constituents	%
Sandstone	64
Siltstone	17
Clay Stone	1
Igneous Rock	2
Calcareous Sandstone	1
Carbonate Fragments	5
Quartz	2
Feldspar	1
Pyrite	~3
Iron hydroxide containing grains	4

2.2 Binders

For twelve different mortar mixes CEM I 42.5 R cement, ground granulated blast furnace slag and Class F Fly ash are used. In Table 7 Chemical Analysis of binders are given.

Table 7: Chemical analysis of binders (in %)

	Cement	Slag	Fly ash
	TS EN 197-1	TS EN 15167-1	TS EN 196-2
SiO_2	18,75	41,31	57,35
Al_2O_3	4,69	11,48	17,55
Fe_2O_3	3,29	1,02	12,10
CaO	63,28	36,00	2,91
MgO	1,45	6,00	-
Na_2O	0,37	0,50	0,07
K_2O	0,74	1,08	1,65
Na_2O Equivalent Total Alkali	0,80	1,21	1,16
SO_3	3,53	0,09	0,74
Loss of Ignition	3,69	0,00	0,72

3. MORTAR MIX DESIGNS

Twelve different mortar mixes were designed according to ASTM C 1260 and TI B 51. Mortar mixture designs are given in Table 8.

Table 8: Mortar Mixture Designs.

Mortar Mix No.	Binder (%)			Aggregate (%)	
	Cement	Slag	Fly Ash	F1-Çatalca	F2-Cendere
1	100	-	-	-	100
2	100	-	-	100	-
3	75	25	-	25	75
4	75	25	-	50	50
5	50	50	-	25	75
6	50	50	-	50	50
7	85	-	15	25	75
8	85	-	15	50	50
9	75	-	25	25	75
10	75	-	25	50	50
11	100	-	-	25	75
12	100	-	-	50	50

4. ASR TEST METHODS

ASR risks of F1-ÇATALCA and F2-CENDERE aggregates were investigated by short and long term tests. Thin sections were prepared from the mortars and microscopic observations were carried out. Experimental expansion results were compared with ASR activity on the thin sections.

4.1 ASTM C 1260

The ASTM C 1260 test procedure, short term test, was adopted to evaluate aggregates for their ASR potential. Short term ASR test method means that the tests put the materials in conditions that increase the rate of reaction compared to the rate at which it would occur in the field. ASTM C 1260 is aggressive because of the high temperature and the high concentration of hydroxide used in the test.

In this test, mortar bars were prepared with twelve different mix designs. The mortar bars were then removed from their molds after 24 hours and placed in water at room temperature. The temperature of the water was then raised to 80 °C in a cabin, and the mortar bars were stored in this condition for the next 24 hours. After the bars were removed from the water, they were measured for initial length and then submersed in a 1 normal (N) NaOH solution at 80 °C, where they were then stored for 14 days. Length change measurements were made periodically during this storage period.

In ASTM C1260, expansion criteria are specified. Expansions lower than %0.10 is considered innocuous, 0.10% -0.20% is considered potentially reactive and above %0.20 is considered reactive aggregate.

4.2 TI B 51

The TI B 51 test procedure (long term test) was adopted to evaluate fine aggregates for ASR potential. In this test, mortar bars were prepared with twelve different mix designs. The mortar bars were then removed from their molds after 24 hours and placed in water at room temperature. The temperature of the water was then raised to 50 °C in an oven, and the mortar bars were stored in this condition for the next 24 hours. After the bars were removed from the water, they were measured for initial length and then submersed in NaCl solution at 50 °C, where they were then stored for 20 weeks. Length change measurements were made periodically during this storage period.

It is assumed that, at the end of the test period, if the expansion of the mortar bar is higher than %0.05 the aggregate is considered reactive.

4.3 Microscopic observations

Thin sections were produced from the mortars. Fluorescent epoxy was used for producing thin sections. ASR sign investigated under polarized microscope with using yellow and blue filters.

5. TEST RESULTS AND EVALUATION

Physical and chemical properties of F1-ÇATALCA and F2-CENDERE aggregates are suitable as concrete aggregates.

In Figure 1 thin section photos, with crossed polar light, of F1-ÇATALCA natural sand and F2-CENDERE crushed sand are given.

In ASTM C 295 it is pointed that, alkali-silica reactive constituents found in aggregates include: opal, chalcedony, cristobalite, tridymite, highly strained quartz, microcrystalline quartz, volcanic glass, and synthetic siliceous glass. Aggregate materials containing these constituents include: glassy to cryptocrystalline intermediate to acidic volcanic rocks, some argillites, phyllites, graywacke, gneiss, schist, gneissic granite, vein quartz, quartzite, sandstone, and chert. According to ASTM C 295 both F1-ÇATALCA natural sand and F2-CENDERE crushed sand contain constituents that can cause ASR.

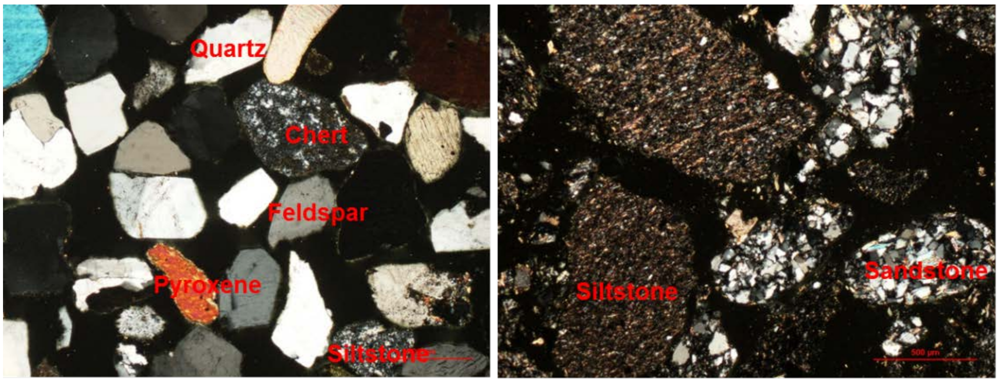


Figure 1: Thin section photos of F1-ÇATALCA natural sand and F2-CENDERE crushed sand. Thin section photos were taken with x 50 magnification.

According to the TI B 52 analysis result, if total alkali reactive material content is above 1%, the aggregate is considered as reactive. Total alkali reactive material (Porous + Opaline Chert) content of F1-ÇATALCA aggregate is 1.4%.

When the petrographic analysis are evaluated, the aggregates can be considered as potentially reactive.

ASR risks of F1-ÇATALCA and F2-CENDERE aggregates were also investigated by short and long term mortar bar tests. In Figure 2 short term test results are given.

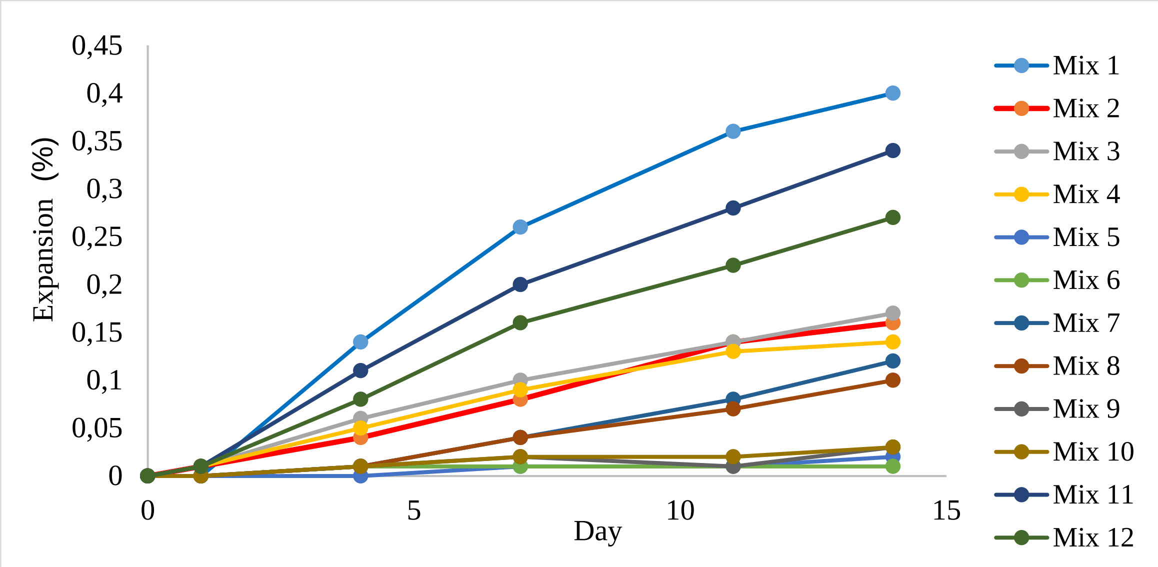


Figure 2: ASTM C 1260 Test results of 12 different mortar mixes

According to the ASTM C 1260, 3 of the 12 mortar mixes expansion were over the compliance criteria. All of those 3 mortar mixes do not contain mineral admixtures in in the cementitious phase. As petrographic investigation of aggregates, short term mortar bar test results also show both F1-ÇATALCA and F2-CENDERE aggregates are reactive or potentially reactive. Test results demonstrated that the usage of the mineral admixture, significantly reduced the length change expansion of the mortars.

Even at such a high temperature and high concentration of hydroxide testing environment, the usage of blast furnace slag significantly reduced the length change expansion of the mortars.

In Figure 3 long term mortar bar test results are given.

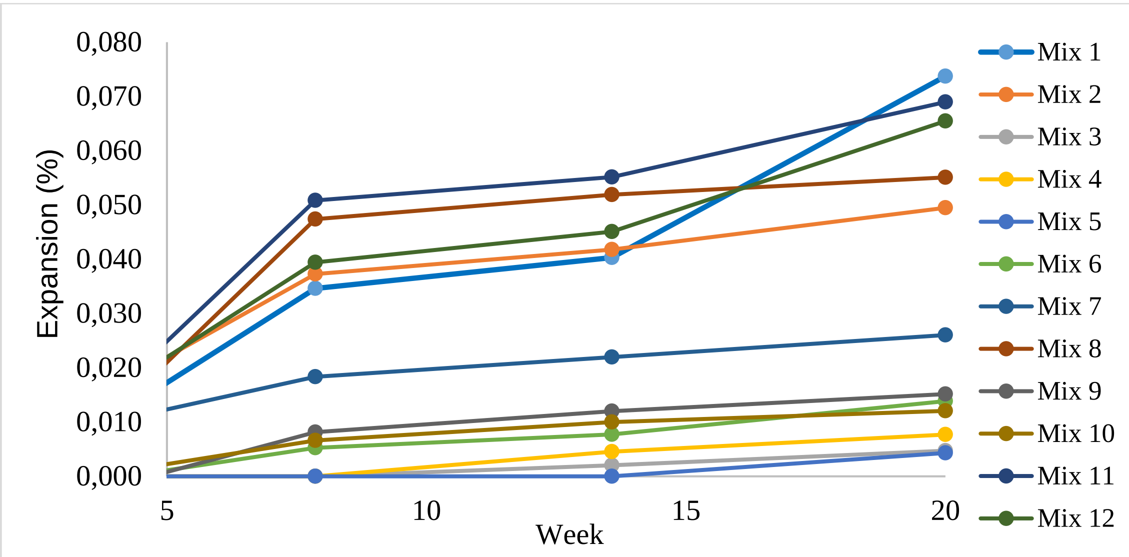


Figure 3: TI B 51 Test results of 12 different mortar mixes

According to the TI B 51, 4 of the 12 mortar mixes expansion were over the compliance criteria and Mix. 2 expansion was almost at the compliance criteria. 4 of those 5 mortar mixes do not contain mineral admixture in the cementitious phase. As petrographic investigation of aggregates, as short term mortar bar test results, long term mortar bar test results also show F1-ÇATALCA and F2-CENDERE aggregates are reactive or potentially reactive. Long term mortar bar test results clearly demonstrated that the usage of the mineral admixture, significantly reduced the length change expansion of the mortars.

In Figure 4 both short term and long term final expansion results of 12 different mortar mixes are given on the same graph. ASTM C1260 states that the aggregates can be considered innocuous if the expansion is below 0,1%, and reactive if the expansion is above 0,2%. This statement is further reinforced by the corresponding low TI-B 51 expansion values when ASTM C1260 expansion is below 0,1%, and high TI-B 51 expansion values when ASTM C1260 expansion is above 0,2%. For the expansion values in between 0,1% and 0,2%, ASTM C1260 requires further investigation. This is also reinforced by the TI-B 51 test, where high or low expansion values are measured for the corresponding samples.

Expansion values above 0,1% and below 0,2% (according to ASTM C1260) need further investigation, whether by long term testing and/or thin section analysis of the tested samples.

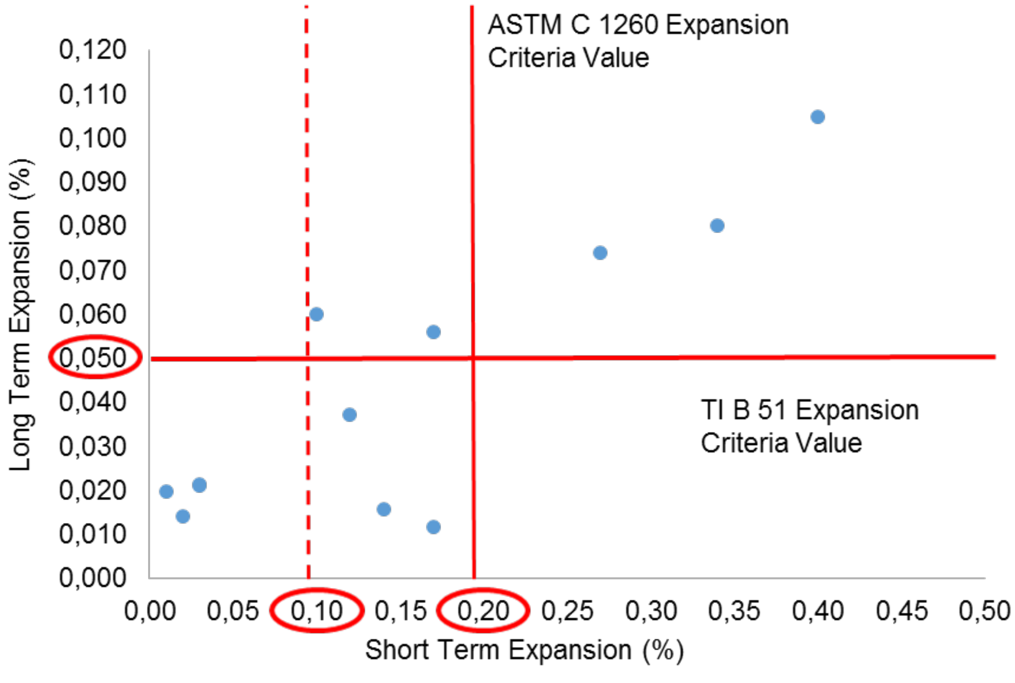


Figure 4: Short term and long term test results of 12 different mortar mixes

Thin sections were produced from the mortars for further investigation. Fluorescent epoxy was used in the production of thin sections. ASR signs were investigated under polarized microscope with yellow and blue filters.

According to the ASTM C 1260 and TIB 51 mortar bars thin section analysis, the higher the length change expansion, the more the number of ASR signs. According to the microscopic investigation, some aggregates were damaged, some aggregates were cracked towards the cement matrix. ASR gel was also observed inside some microcraks. In Figure 5 thin section photos of ASR sign are presented. Photos were taken respectively with crossed polar light (XPL), plane – polarized (PPL) light and with ultraviolet (UV) light. Thin section photos were taken with x 50 magnification.

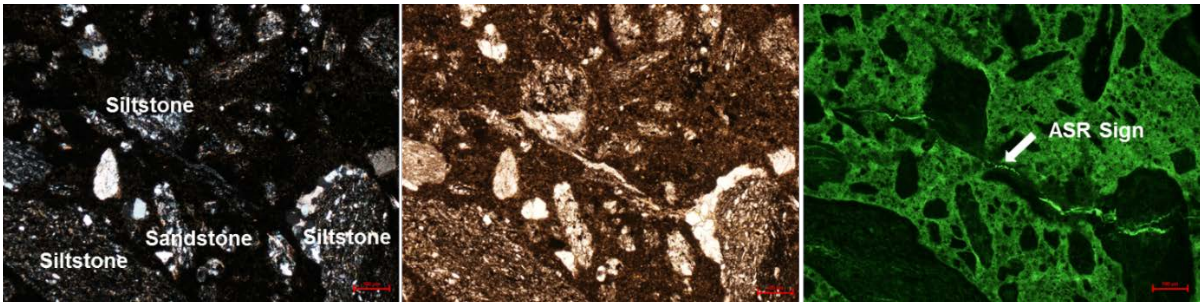


Figure 5: Thin section photos of ASR sign

6. CONCLUSION

The reactivity rate of aggregates with alkalis depend on the mineralogical properties and porosity of aggregates. Therefore, evaluation of aggregates with respect to ASR risk should contain not only physical, chemical, petrographic studies and short term mortar bar testing, but also long term ASR testing, with mortar or concrete bars, supported with microscopic observations.

It is also observed that mineral admixtures such as fly ash and slag are useful measures for ASR mitigation.

REFERENCES

[1] R.N. Swamy, “Alkali - Silica Reaction in Concrete”, 1990.
[2] Malhorta, V.M., “Petrography and Alkali – Aggrregate Reactivity, Course Manual”,Canada, 1991.
[3] H. Özkul, M. A. Taşdemir, M. Tokyay, M. Uyan, "Her Yönüyle Beton", Turkish Ready Mixed Concrete Association İstanbul, 2004.
[4] Wigum, B.J., Pedersen, L.T., Lindgard, J., “State of the art report: Key parameters influencing the alkali aggregate reaction”, Sintef Building and Infrastructure Part 1 &2, 2006.
[5] Keskin, G., “Evaluation of Cendere Valley Rocks as Concrete Aggregate”, Master's Thesis, Istanbul Technical University, Geological Engineering, Istanbul, 2011.

EFFECT OF BINDER TYPE AND WATER/BINDER RATIO ON CHLORIDE PENETRATION DEPTH OF CONCRETE EXPOSED TO ACTUAL MARINE ENVIRONMENT

Hilmi Aytaç (1), Hüseyin L. Sevin (1), Ahsanollah Beglarigale (2), Hüseyin Yiğiter (2) and Halit Yazıcı (2)

(1) Bursa Beton Ready-Mixed Concrete Company, Turkey

(2) Dokuz Eylül University, Civil Engineering Department, Turkey

Abstract

Reinforced concrete elements in marine environment are exposed to various physical and chemical influences. In general, it can be stated that the elements in the wetting-drying regions are exposed to highest rate of damage. Since there is a combined effect generated by chloride and sulphate ions in sea water, the concrete while on one hand bears the risk of destruction due to sulphate effect, on the other hand there is a risk for the reinforcement to be corroded by chloride (for reinforced concrete systems) ions as well.

There are discussions on which type of cement would be the best for use in marine environment. Although it seems that use of cement type with relatively high amount of C_3A would be more suitable due to its capability of binding chloride ions and therefore to reduce the risk of corrosion, it is a known fact that a concrete with resistance to sulphate effect should have as low C_3A content as possible.

In this study, the effects of cement type (CEM I 42.5R, CEM II 42.5R, CEM I-SR 5), water/binder ratio and use of fly ash on the chloride penetration depth of concrete were investigated on samples kept in actual marine environment. The results of the experiments revealed that the given parameters have remarkable effects on permeability of the concrete.

Key words: Sea water, cement type, corrosion, durability, C_3A, chloride penetration depth.

1. INTRODUCTION

The constructions, since the time they were built, are exposed to the effects of mechanic, physical and chemical factors generated by the external environment and within the course of time, they lose their original characteristics. Constructions built in marine environment or locations close to sea are exposed to the effects of one or a combination of these factors. The most significant problem that emerges in the reinforced concrete buildings, directly or indirectly affected by the marine environment, is reinforcement corrosion. This problem can be eliminated by physical and chemical protection of the reinforcement by a well-designed, impermeable and high quality concrete. Physical protection can be achieved by restraining the access of harmful substances to the reinforcement whereas chemical protection can be achieved by creating an environment with high pH. Despite these positive characteristics of the concrete, imperfections in design and application conduced the corrosion to become the primary contributing cause which determines the service life of reinforced concrete buildings of our time. Chloride ions penetrating into the concrete, in particular for the constructions recurrently exposed to wetting-drying effect, are retained in the concrete as a result of evaporation of water and chloride density increases as the number of cycles become more. In this case, more than the chloride ion concentration in sea water can be accumulated within the concrete [1-3]. The resulting reinforcement corrosion decreases the reinforcement cross-section and on the other hand corrosion products causing expansion and sulphate attacks have considerable destructive effects on the concrete.

There are criteria stipulated for direct use of sulphate-resistant cement in the technical specifications of many projects accomplished in the coastal lines of our country. Yet, it is not a proper measure use of sulphate-resistant cement by establishing the high amount of sulphate concentration contained in sea water. Although such type of cement is recommended against the lean sulphate effects, in terms of the corrosion of reinforcement, it does not seem to be a proper solution in the event of a combined chemical attack such as sea water effect. When the studies with respect to the subject matter is examined, no information concerning the direct use of sulphate-resistant cement in the constructions exposed to sea water is encountered.

It is evident that C_3A content of sulphate-resistant cement should be as low as possible for sulphate attack, notwithstanding the fact that use of cement with relatively high amount of C_3A content is favoured due to its capability to mitigate corrosion risk. At this point, it is so crucial to decide which the dominant effect would be. Within the frame of the foregoing, ready mixed concrete user brings up the use of sulphate-resistant cement for the constructions nearby marine environment and predicates all its assessments related with durability entirely on sulphate attack disregarding the reinforcement corrosion issue.

Considering all of these, in this study, the effect of water / binder ratio, concrete strength class, cement type and fly ash usage were investigated, by means of chloride penetration depth analysis performed in actual marine environment.

2. EXPERIMENTAL STUDY

2.1 Materials

Chemical and physical properties of cements types CEM I 42.5R, CEM II / A-M (P-L) 42.5R and CEM I 42.5 R-SR5 conforming to TS EN 197-1 standard are given in Table 1. C_3A content is derived through Bogue formula. C_3A content of CEM II cement was not calculated.

Table 1: Chemical and physical properties of cements

		CEM I	CEM II A-M/P-L	CEM I SR 5
SiO_2 (%)		19.07	23.17	19.66
Al_2O_3 (%)		5.61	6.16	3,98
Fe_2O_3 (%)		2.91	2.82	4,15
CaO (%)		62.28	56.57	64.82
MgO (%)		0.87	0.84	0.99
SO_3 (%)		2.75	3.08	2,45
Na_2O (%)		0.61	0.74	0,09
K_2O (%)		0.67	0.86	0,61
Loss on Ignition (%)		4.82	4.90	2.76
Additive (%)		-	18.07	-
Blaine (cm^2/g)		4780	5070	3790
Compressive Strength (MPa)	2 days	31.7	30.4	24.9
	7 days	51.3	46.0	44.2
	28 days	58.1	54.8	55.8
C_3A (%) (Bogue)		9.95	-	3.53

Chemical and physical properties of the fly ash obtained from Orhaneli/Bursa Thermal Power Plant are given in Table 2.

Table 2: Chemical and physical properties of fly ash

SiO_2 (%)	Al_2O_3 (%)	Fe_2O_3 (%)	SO_3 (%)	L.O.I (%)	Free CaO (%)	Spec. Grav.	Blaine Fineness (cm^2/g)	React. CaO (%)	45µ residue (%)	28 Day Act. (%)
51.47	25.08	10.89	0.88	0.73	0.10	2.21	3675	4.38	32.9	88.2

Crushed lime stones obtained from a single source were used. Modified polycarboxylate based superplasticizer admixture conforming to TS EN 934-2 was used in concrete mixes. Solid substance rate of plasticizer is 25,66%, its density is 1,090 and pH value is 5,90.

2.2 Mixture designs

Water / binder ratio (concrete class) is one of the main variables of during the experimental study. Designs are made for C25/30 class which is commonly used in ready mixed concrete sector and for C40/50 class which has comparatively high strength. Total binder dosage is selected as 350 kg/m^3 and water/binder ratio is selected as 0.56 for Class C25/30 concrete whereas binder dosage is 450 kg/m^3 and water/binder ratio is 0.43 for Class C40/50 concretes.

Another main variable in the study is the cement type. Sample productions were made with three different cement types being ordinary Portland cement (CEM I 42.5R), Portland composite cement (CEM II / A-M (P-L) 42,5R) and sulphate-resistant cement (CEM I 42.5 R-SR 5) in order to specify the effect of cement type on sea water resistance of concrete and reinforcement corrosion.

Concrete mixtures were also prepared by replacing 20% of cement with fly ash. The amount of water is identical for mixtures with or without fly ash for the same concrete class. Plasticisers in different amounts were used for the mixtures to have similar workability. In total, performances of 12 different mixtures were measured. Mixture proportions and characteristics of fresh concrete are given in Table 3.

Table 3: Concrete mixture designs and fresh state properties

<table>
<tr><th>Concrete Class</th><th>Mixture Code</th><th>Cement (kg/m^3)</th><th>F. Ash (kg/m^3)</th><th>Water (kg/m^3)</th><th>Plasticiser (%)</th><th>Plasticiser (kg/m^3)</th><th>0 / 5 (kg/m^3)</th><th>5 / 12 (kg/m^3)</th><th>12 / 22 (kg/m^3)</th><th>Water / Binder</th><th>Air Temp. (ºC)</th><th>Concrete Temp. (ºC)</th><th>Air content (%)</th><th>Slump (cm)</th></tr>
<tr><td rowspan="6">C25/30</td><td>C25CEMIFA</td><td rowspan="3">280</td><td rowspan="3">70</td><td rowspan="6">196</td><td>0.1</td><td>0.35</td><td>889</td><td>391</td><td>481</td><td rowspan="6">0.56</td><td>17</td><td>17</td><td>1.8</td><td>21</td></tr>
<tr><td>C25CEMIIFA</td><td>0.4</td><td>1.40</td><td>889</td><td>391</td><td>481</td><td>16</td><td>17</td><td>1.5</td><td>20</td></tr>
<tr><td>C25SRFA</td><td>0.1</td><td>0.35</td><td>889</td><td>391</td><td>481</td><td>12</td><td>12</td><td>2.1</td><td>21</td></tr>
<tr><td>C25CEMI</td><td rowspan="3">350</td><td rowspan="3">0</td><td>0.1</td><td>0.35</td><td>906</td><td>398</td><td>491</td><td>5</td><td>9</td><td>2.0</td><td>20</td></tr>
<tr><td>C25CEMII</td><td>0.4</td><td>1.40</td><td>906</td><td>398</td><td>491</td><td>2</td><td>9</td><td>2.1</td><td>20</td></tr>
<tr><td>C25SR</td><td>0.0</td><td>0.00</td><td>906</td><td>398</td><td>491</td><td>5</td><td>9</td><td>1.4</td><td>20</td></tr>
<tr><td rowspan="6">C40/50</td><td>C40CEMIFA</td><td rowspan="3">360</td><td rowspan="3">90</td><td rowspan="6">194</td><td>0.4</td><td>1.80</td><td>809</td><td>405</td><td>460</td><td rowspan="6">0.43</td><td>8</td><td>9</td><td>1.8</td><td>21</td></tr>
<tr><td>C40CEMIIFA</td><td>0.5</td><td>2.25</td><td>809</td><td>405</td><td>460</td><td>10</td><td>12</td><td>1.7</td><td>21</td></tr>
<tr><td>C40SRFA</td><td>0.2</td><td>0.90</td><td>809</td><td>405</td><td>460</td><td>13</td><td>16</td><td>1.8</td><td>21</td></tr>
<tr><td>C40CEMI</td><td rowspan="3">450</td><td rowspan="3">0</td><td>0.4</td><td>1.80</td><td>836</td><td>419</td><td>473</td><td>7</td><td>9</td><td>2.1</td><td>21</td></tr>
<tr><td>C40CEMII</td><td>0.6</td><td>2.70</td><td>836</td><td>419</td><td>473</td><td>12</td><td>12</td><td>2.3</td><td>21</td></tr>
<tr><td>C40SR</td><td>0.2</td><td>0.90</td><td>836</td><td>419</td><td>473</td><td>7</td><td>9</td><td>2.0</td><td>21</td></tr>
</table>

2.3 Research method

The research was conducted in laboratory and actual marine environment. The samples should be exposed to sea water in a safe and systematic manner. A wetting-drying mechanism was designed capable of carrying 600 samples with the dimensions of 75x75x285 mm. Wetting-drying cycles were performed automatically (Figure 1-2). Since wetting of the samples takes place in a short time and drying occurs in a long time, wetting-drying cycles were done as the samples were kept in the sea for 12 hours and in the air for 24 hours.

Wetting-drying platform consists of a main load-bearing system and associated with 4 baskets. Baskets where the cells containing the samples are placed, are suspended to the load bearing frame by means of pistons. When the 12-hours wetting period is completed, pistons hoist the baskets and thus the drying phase starts. Sample baskets kept await outside the water for 24 hours are immersed into the water when the drying phase is completed and then the wetting period starts again. The wetting-drying cycles go on in this manner and the number of cycles is monitored by means of a mechanic meter attached to the platform. The entire system is automatic and there is no need for manual treatment to the submersion-emersion process.

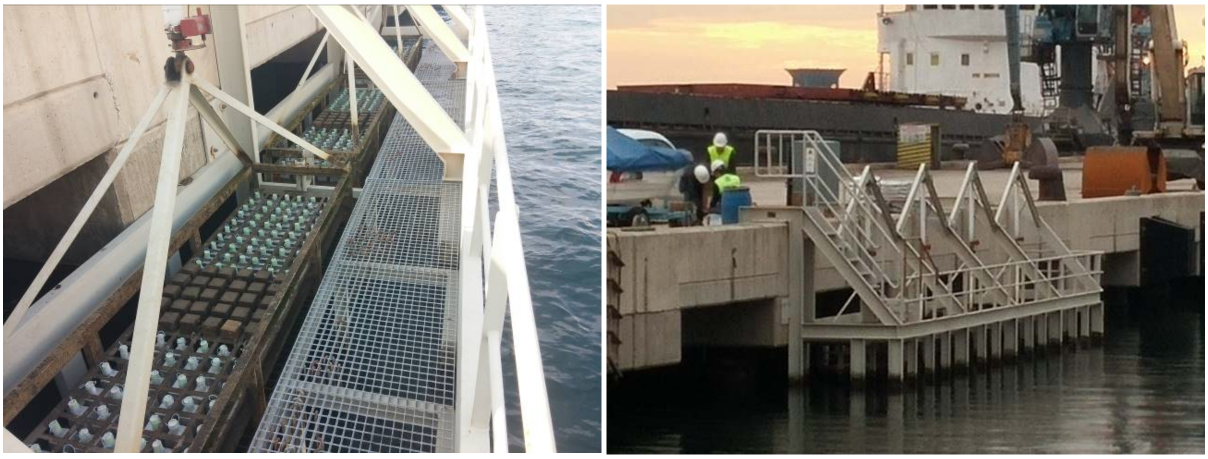


Figure 1-2: Wetting-drying mechanism and test samples installation

From all mixtures, 3 cubic samples each with an edge of 150 mm per each test age were taken to determine the compressive strength before the cycles and 20 prismatic samples each with size of 75*75*285 mm were taken to measure chloride penetration depth in different cycle periods. It is a known fact that strength developments would be different depending on cement type and use of fly ash. It was considered that the samples having pozzolanic material would be quite different strength level before the wetting-drying cycles. Therefore, it was aimed to reach approximately similar strength levels before sea water exposure. For this purpose, following the 28-days standard curing process, extended time of water curing was applied for some mixes. For example, when the mixture reach the desired strength level samples were taken out from the curing pool (keeping on air at room temperature) while other specimens were maintained in the water curing. After the strength level achieved a certain level, prismatic samples were taken to the port area and placed on the platform and cycles were started. Table 4 indicates the compressive strength values of the samples before being exposed to seawater.

Table 4: Compressive strength values of concretes prior to sea water exposure (MPa).

C25 CEMI	C25 CEMII	C25 SR	C25 CEMIFA	C25 CEMIIFA	C25 SRFA	C40 CEMI	C40 CEMII	C40 SR	C40 CEMIFA	C40 CEMIIFA	C40 SRFA
44.8	46.2	43.7	43.8	43.2	41.5	62.0	57.9	56.7	57.3	56.1	55.0

After a certain amount of cycles, prismatic beam samples were subjected to bending test and 0.1 N $AgNO_3$ solution was sprayed on the cross-section that revealed after bending. Colour changing zones were recorded as the chloride penetration depth [4].

3. RESULTS AND DISCUSSION

Chloride penetration depth values for Class C25/30 samples are shown in Figure 3. For each cycle, concretes without Fly Ash have deeper chloride penetration whereas concretes with Fly Ash content had lower chloride penetration depths. It is considered that chloride bonding effect of Fly Ash and reduced permeability as a result of pozzolanic reaction led to these results.

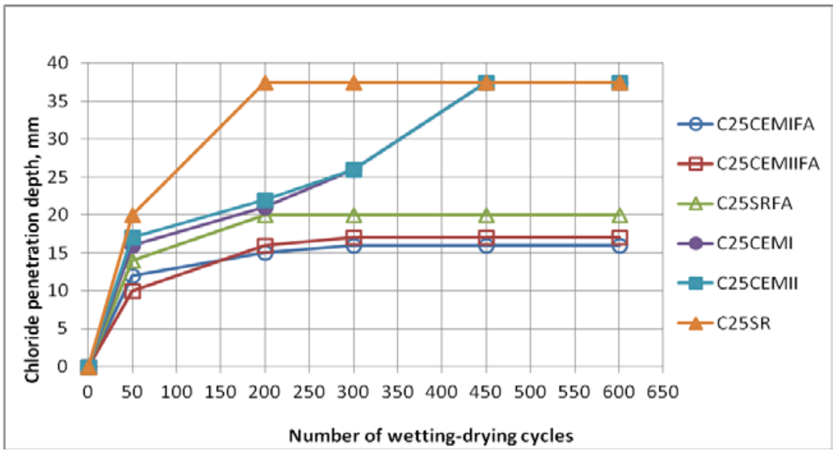


Figure 3: Chloride penetration depths for Class C25/30 concretes.

The highest penetration depths of mixtures with or without Fly Ash are determined in concretes with SR Type cement. Results showed the importance of binder type rather than the compressive strength.

After 50 cycles, CEMIIFA had the lowest chloride penetration depth whereas SR had the highest value. In such a short while, penetration depth of SR type cement is exactly twice the depth of CEMIIFA. The significance of binder type could be observed evidently in such short period of time.

After 200 cycles, C25CEMIFA and C25CEMIIFA had the lowest chloride penetration depths. These two mixtures have almost the same performance. SR type cement showed the highest penetration depth (37,5mm). Taking into consideration that this result was obtained from samples with 75*75 mm cross-section, it can be concluded that chloride had penetrated the entire sample cross-section. Here, the possibility of C25SR mixture to achieve 37,5 mm value at an earlier stage should not be disregarded. Furthermore, stabilization of chloride penetration depth after 200 cycles does not denote cease of chloride penetration and most probably chloride continues to penetrate. Such a feeling arises because chloride depth more than 37,5 mm could not be measured since sample cross-section sizes are 75*75 mm. It is considered that, insufficient C_3A content to get into reaction with chlorides would be the reason for penetration of chloride to the entire section in C25SR mixture in such a short period of time.

Penetration depths of C25CEMI and C25CEMII mixtures achieved the maximum value (37,5 mm) in 450 cycles and chloride penetrated the entire cross-section. It can be concluded that more C_3A content compared to C25SR provided somewhat extra time to this mixture. Most probably, chloride penetration process goes on as in C25SR mixture.

When C25CEMIFA and C25CEMIIFA mixtures are taken into account, a slight increase can be observed from 200 to 300 cycles. When it is considered that such slight increase is at a negligible level, it is seen that penetration depths of mixtures with Fly Ash remain stable during 20 months between 200 to 600 cycles. Among those mixtures, C25SRFA had the highest value with 20 mm.

While after 450 cycles chloride penetrates the entire cross-section independent from cement type in mixtures without Fly Ash, the level of chloride penetration depth in mixtures with Fly Ash is almost the half of maximum penetration depth.

C25CEMI and C25CEMII, C25CEMIFA and C25CEMIIFA mixtures had highly comparable performances during the wetting-drying cycles.

Figure 4 indicates chloride penetration depth values for Class C40/50 samples.

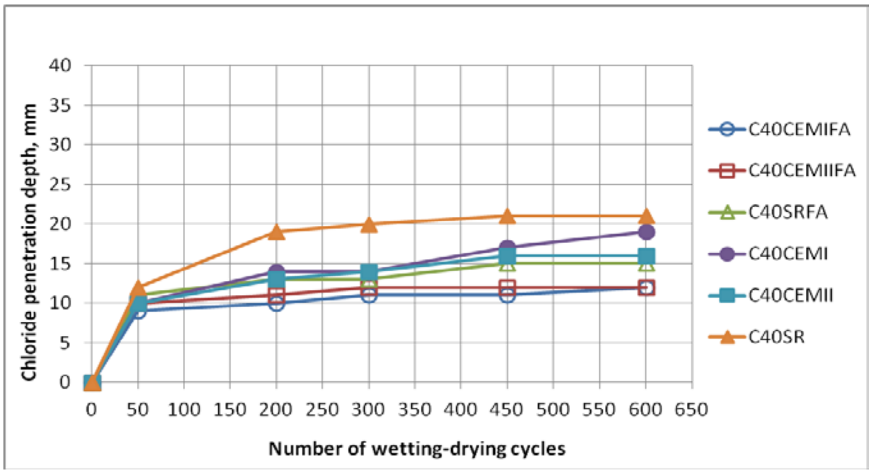


Figure 4: Chloride penetration depths for Class C40/50 concretes.

Not surprisingly, chloride penetration depths for Class C40/50 concrete were lower compared to Class C25/30 concrete. Although fly ash was not used in C40SR mixture, penetration values of almost the same level (~20 mm) with C25SRFA after 200 cycles is significant regarding the effect of compressive strength.

Rank of test results of class C40/50 mixtures according to chloride penetration depth value is notably similar to those of Class C25/30 concretes. Lower chloride penetration depth value is obtained in mixtures with mineral additives as in Class C25/30.

The highest chloride penetration value of mixtures with or without Fly Ash is derived from mixtures with CEMISR cement, as in Class C25/30.

C40CEMI and C40CEMII, C40CEMIFA and C40CEMIIFA mixtures showed similar performance to chloride penetration as in Class C25/30.

It is observed that chloride penetration depths of mixtures other than C40SR mix remain in a narrower range compared to Class C25/30. It can be said that compressive strength is more dominant factor compared to cementitious type. In other words, chloride penetration depths values are lower and in a restricted range compared to Class C25/30 due to the concretes having higher strength and therefore being more impermeable.

4. CONCLUSIONS

In this research the effect of concrete class, cement type and fly ash usage on the chloride penetration resistance was investigated under actual seawater exposure. Test results revealed the following conclusions.

Mixtures with fly ash are more resistant to chloride penetration compared to mixtures without fly ash. High chloride binding capacity of mineral additives and reduced permeability as a result of pozzolanic reaction are the most crucial reasons of taking lower readings for chloride penetration depth.

Concretes with highest resistance to chloride penetration are the mixtures with fly ash content where CEM I and CEM II cements are used. It is assumed that high amount of C_3A in these mixtures compared to CEM I SR Type cement allows more chloride bonding and thus the resistance is increased.

Concretes with lowest resistance to chloride penetration are mixtures with sulphate-resistant cement and concrete without fly ash content.

Accordingly, the following order can be given for chloride penetration resistance:

CEMIFA ≈ CEMIIFA > SRFA > CEMI ≈ CEMII > SR

The above results are independent from concrete class.

The effect of binder type from the point of chloride penetration depth become more important in concrete class C25/30. Accordingly, it can be asserted that the significance of binder type will diminish when concretes with higher quality (C40/50 and above) are used.

Highly accelerated chloride penetration is in question during the first phases of wetting-drying cycles. Therefore, a design with higher early strength should be made for the constructions exposed to sea water, in particular for the elements in wetting-drying zones.

Use of only sulphate-resistant cement without using mineral additives such as fly ash, blast furnace slag and silica fume would be insufficient for reinforcement protection.

Whatever the binder type, concrete cover should be as high as possible for constructions exposed to sea water. It can be recommended that use of at least 50 mm concrete cover, particularly reinforced concrete elements within the wetting-drying zone.

Increase of chloride penetration as the amount of C_3A decreases, as specified in the literature, was verified during this research. This is why sulphate-resistant cement concrete displayed the worst chloride penetration performance.

Sulphate-resistant cement, an effective solution for only sulphate effect, becomes no more a solution in case of a combined effect with chloride attack. In addition, even use of mineral additive together with this type of cement would not become a satisfactory solution. Based on all above, when an assessment based solely on binder type is conducted, it seems to be most effective choice to use combinations, with mineral additives, of CEM I and CEM II type cements which contain relatively higher amounts of C_3A.

ACKNOWLEDGEMENT

This study was sponsored by the Scientific and Technological Research Council of Turkey (TÜBİTAK, Project Code: 112M899). The authors gratefully acknowledge the support of TÜBİTAK.

REFERENCES

[1] Baradan, B., Yazıcı, H. and Ün, H., ‘Betonarme yapılarda kalıcılık’, Dokuz Eylül University Faculty of Engineering Publications, İzmir, (2002).

[2] ACI 222.3R-03, ‘Design and construction practices to mitigate corrosion of reinforcement in concrete structures’, Manual of Concrete Practice, American Concrete Institute.

[3] Andrade, C., Alonso, C. and Sarria, J. ‘Corrosion rate evolution in concrete structures exposed to the atmosphere’, Cement and Concrete Composites, 24, (2002), 55-64.

[4] Otsuki, N., Nagataki, S. and Nakashita, K., ‘Evaluation of the $AgNO_3$ solution spray method for measurement of chloride penetration into hardened cementitious matrix materials’, Construction and Building Materials, 7, Issue 4, (1993), 195-201.

EVALUATION OF RAPID CHLORIDE INGRESS IN CONCRETES INCORPORATING SILICA FUME

Şakir Erdoğdu (1) and Ufuk Kandil (1)

(1) Karadeniz Technical University, Turkey

Abstract

The two basic characteristics for concrete are strength and durability. These are in most cases mutually dependent with one another. From sustainability point of view, durability is the key criterion as it is the dominant characteristic controlling the service life. Durability becomes significant when concrete is in contact with waters contain chlorides and sulfates. Concrete may deteriorate as a result of direct interaction of sulfates with the hydration products of cement and rebar corrosion. The shortcut for preventing this scenario is to design and produce concrete as much dense as possible. To solve this problem, mineral additives are generally incorporated in concrete mix. Concretes containing mineral additives are considered to be less permeable compared to those without mineral additives since such materials are usually finely ground. Silica fume is generally used for this purpose as it is the finest among these. In the meantime, silica fume is frequently used in cases where high strength is aimed since its pozzolanic reactivity is relatively high.

In this study, concretes were produced with binders in which cement was replaced by silica fume at different ratios, and the rapid chloride permeability, the maximum depth of water penetration, the capillary coefficient and the compressive strength were measured to determine the effectiveness of silica fume replacement. 5% silica fume replacement resulted in a relatively high reduction in chloride ingress. This improvement can also be observed in the maximum penetration depth of water and the capillary coefficient. A considerable increase in compressive strength is also observed as the ratio of silica fume is increased. Overall, the results revealed that silica fume incorporation is favorable with respect to the strength gain and durability and hence sustainability aspects.

Keywords: Concrete, durability, silica fume, rapid chloride ingress, maximum depth of penetration of water, capillary coefficient

1. INTRODUCTION

Concrete is one of the most commonly used materials in construction sector as it is relatively low cost, readily available and can be produced and shaped as desired, and also as it sufficiently resistant to the aggressive environments compared to its alternatives.

Concrete is an alkaline material with high pH. Therefore, concrete mostly deteriorates in acidic environments due to the chemical reactions taking place. Permeability is the chief factor influencing the durability of concrete exposed to such conditions [1].

The factors controlling the permeability of concrete are the number and the size of the voids in concrete, their distribution and the conditions of the surrounding environment. Minimization of the number and the size of such voids would be possible as long as the principles of concrete technology are applied properly [2].

Mineral additives such as silica fume, fly ash and ground granulated blast furnace slag are the common materials that have been used in the production of high strength concrete by replacing Portland cement by them at certain ratios. A considerable energy can be saved and a reduction in CO_2 emission is possible by incorporating such materials as cementitious components of binders in the production of concrete [1,3]. The amount of CO_2 emission is approximately 1 kg per 1 kg of cement production [4]. The reduction provided in gas emission is more or less equivalent to the replacement ratio in case silica fume is used as the mineral constituent.

Silica fume is a by-product of the manufacture of silicon metal and ferro-silicon alloys. The process involves the reduction of high purity quartz (SiO_2) in electric arc furnaces at temperatures in excess of 2000°C. It is a very fine powder consisting mainly of spherical particles or microspheres of mean diameter about 0.15 microns, with a very high specific surface area of 15000–25000 m^2/kg. Each microsphere is on the average 100 times smaller than an average cement grain [5]. Silica fume reacts chemically with the calcium hydroxide produced by the hydration of the Portland cement to form calcium silicate hydrates (C-S-H). Silica fume is highly reactive due to the high proportion of non-crystalline SiO_2 and the large surface area [6].

Reinforced concrete is a worldwide construction material [7]. Chloride ingress in concrete is undesirable due to its electrochemical reaction with reinforcing steel in concrete and the eventual detrimental effect on the durability of reinforced concrete structures [8]. Permeation of sulfate and chloride ions into concrete takes place by diffusion and capillary suction [9]. Corrosion of rebar in concrete may result in structural damage to buildings, bridges, viaducts, marine structures, and many other similar structures. The money spent annually is approximately 5 billion Euros to compensate for the damage in reinforced concrete structures in Europe [10].

It is stated out in many studies that it is possible to reduce the permeability of concrete considerably and increase its strength by replacing a certain amount of Portland cement with silica fume [11-13]. For example, in a study [11], it is stated that concrete with a water to cement ratio of 0.50 and 10% silica fume substitution yielded a compressive strength 25% higher compared to the concrete without silica fume at the end of 28 days of standard curing. Based on the results of rapid chloride permeability test carried out on the same concrete, compared to concrete without silica fume, a reduction of approximately 80% in chloride permeability is measured.

2. PURPOSE OF THE STUDY

In the scope of this study, the effect of silica fume replacement of cement at different ratios on the chloride ingress, water permeation, capillarity, and compressive strength of concrete was investigated.

In the experimental program, concrete mixes containing 290 kg/m^3 and 340 kg/m^3 cementitious materials were produced. The reason for choosing these dosages is these binder contents was that these are the binder contents generally used in the production of concrete grades C20/25 and C25/30 commonly used in the local construction sector. Cement was replaced by silica fume at ratios of %5, %10, and %15 for each cement contents. Control mixes without silica fume for each cement content were produced for comparison. Slump was kept within 60 mm±10 mm for all mixes. This was realized by using a super plasticizer chemical admixture. All concrete specimens were kept under standard (20^{o}C±2^{o}C, in lime saturated water) curing condition until the testing age.

3. EXPERIMENTAL WORK

3.1 Materials used

The aggregate used was a mixture of calcareous crushed aggregate with a maximum size of 16 mm and crushed sand. The particle density at SSD state, water absorption and moisture content of the aggregates are given in Table 1. The gradation of the aggregate mix along with the reference curves is given in Figure 1.

Table 1: Particle density, water absorption, and moisture content of the aggregates.

Aggregate	Particle density, Mg/m^3	Water absorption, %	Moisture content, %
Crushed stone coarse aggregate	2.74	1.00	0.25
Crushed sand	2.64	2.60	1.60

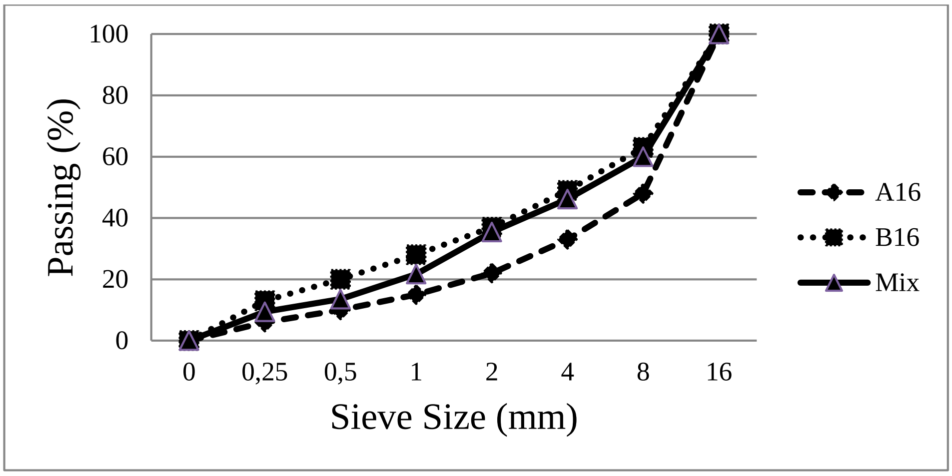


Figure 1: Gradation of the aggregate mix and the reference curves.

In the program, CEM I 42.5 R type cement manufactured by Aşkale Trabzon Cement Factory was used. The properties of the cement are given in Table 2.

Table 2: Chemical composition, physical and mechanical properties of cement.

Chemical Composition		Physical and Mechanical Properties		
Oxides	Content, %	Retained on 45 μm sieve, %		9.8
SiO_2	19.46	Retained on 90 μm sieve, %		1.0
Al_2O_3	5.11	Specific surface-(Blaine), m^2/kg		412.6
Fe_2O_3	3.31	Particle density, Mg/m^3		3.12
CaO	60.23	Setting times-(Vicat), min.	Initial	140
MgO	2.08		Final	200
SO_3	3.05	Water demand, %		29.2
Na_2O	0.27	Soundness, mm		1.0
K_2O	0.69	Compressive strength, MPa	2-day	28.0
Cl^-	0.02		7-day	40.4
Loss on ignition	3.00		28-day	51.5

The silica fume used is provided by İKSA Concrete and Construction Chemicals. A super plasticizer chemical admixture was also used in the experimental program.

3.2 Mix proportions

The mix proportions of the concretes prepared are given in Table 3. The water to binder ratio was kept constant at 0.50 for all mixes. Four 150 mm cube, two 100 mm cube, one 100 mm diameter and 200 mm height cylinder specimens were prepared for each batch. Two 150 mm cubes were used for compressive test while the other two were used for water ingress test. The two cube specimens of 100 mm were used for determining capillarity coefficient. The cylinder specimen was used for rapid chloride permeability test. All concrete specimens were tested at the end of 28 days of standard curing.

Table 3: Mix proportions of the concretes produced.

Production No	Silica fume replacement, %	Amount of constituent materials, kg/m^3				Slump, mm
		Cement	Water	Aggregate	Silica fume	
1	0	290	145	1994.5	0	60
2	5	275.5		1989.4	14.5	80
3	10	261		1984.2	29	45
4	15	246.5		1979.1	43.5	60
5	0	340	170	1883.9	0	65
6	5	323		1877.8	17	50
7	10	306		1871.8	34	55
8	15	289		1865.8	51	50

3.3 Rapid chloride permeability test

The rapid chloride permeability test was carried out according to ASTM C 1202. Initially, 25-mm slices were cut off from the two ends of the 100-mm cylinder specimens and then 50-mm sliced specimens were prepared from the remaining part of each specimen for the test. Prior to the testing, the concrete specimen was initially water saturated. The specimen was then placed in the test set up shown in Figure 2 so as to have one face of the specimen in contact with a 3% NaCl solution and the other face with a 0.3M NaOH solution. The cylindrical faces of the specimen not in conduct with the solution were isolated using a silicone agent to prevent water leakage. The charge that passed through the specimen within 6 hours under 60 V potential difference was measured in Coulombs (C).

Figure 2: Set up for rapid chloride permeability test.

Based on the charge passed, chloride ion permeability may be classified according to ASTM C 1202 as given in Table 4.

Table 4: Chloride ion permeability classification according to ASTM C 1202.

Electrical Charge Passing (Coulomb)	Chloride Ion Permeability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very low
<100	Negligible

3.4 Water permeation test

Water permeation test was performed according to TS EN 12390-8. The specimens were subjected to a water pressure of 5 bars for a period of 72 hours and then they were split perpendicular to the face of the specimen subjected to water ingress to determine water permeation. The apparatus used for the test is shown in Figure 3. The maximum depth of water ingress on the splitted face was determined by marking the water profile as indicated in Figure 4.

Figure 3: Set up for water ingress.

Figure 4: Marked depth of water ingress.

The evaluation regarding the maximum depth of water permeation measured on concrete specimens according to TS EN 12390-8 is given in Table 5.

Table 5: Criteria for water permeation in concrete.

Maximum Water Permeation, mm	Classification
> 50	Pervious Concrete
< 50	Impervious Concrete
< 30	Impervious Concrete to Aggressive Environments

3.5 Capillary test

Cube specimens of 100-mm were prepared for capillary testing. The test was carried out according to ASTM C1585. The specimens were initially kept in an oven at 70°C for two days and then were left in laboratory condition for cooling. Following the cooling process, the specimens were placed in the set up shown schematically in Figure 5 so as to have the bottom 2 mm ±1 mm of specimens submerged in water. The side faces of the specimens were sealed to prevent water ingress. The amount of water absorbed with time due to capillary suction was measured and recorded at time intervals of 0, 5, 10, 20, 30, 45, 60, 120 and 1440 minutes. The capillarity coefficients in m^2/s were then calculated using these measurements.

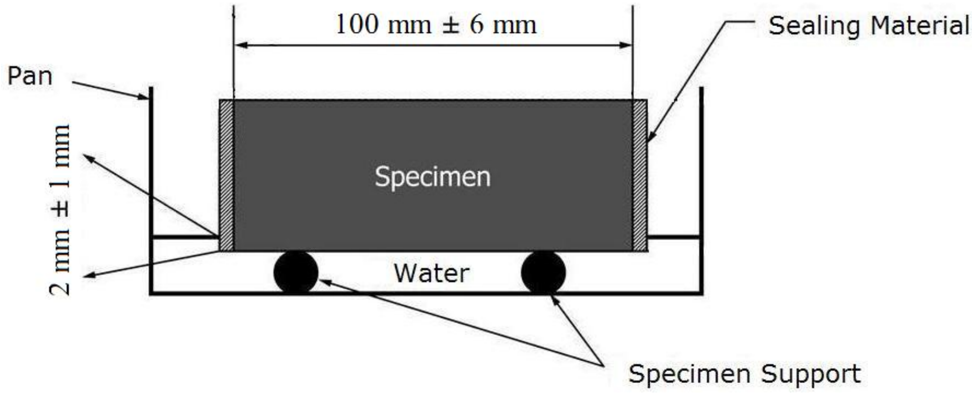


Figure 5: Set up for the capillary test.

3.6 Compression test

Compressive strengths were determined on 150 mm cube specimens. The test was carried out conforming to procedure described by TS EN 12390-3.

4. EXPERIMENTAL RESULTS

The rapid chloride ingress measurements obtained according to ASTM C 1202 are given in Figure 6. The associated graph is a clear picture of the variation of the rapid chloride permeability of concretes depending on the silica fume replacement at ratios of 0%, 5%, 10% and 15%. Evaluation is carried out for concrete specimens of two batches containing total cementitious materials (binder) of 290 kg/m^3 and 340 kg/m^3. A considerable decrease in the chloride permeability of concretes was observed as the replacement ratio increased.

In a study [11], a reduction of 81% in rapid chloride permeability has been reported for concretes with total cementing materials content of 372 kg/m^3 and a silica fume replacement ratio of 10%. In this study, 77% and 79% reductions were obtained for concretes with cementing materials of 290 kg/m^3 and 340 kg/m^3 binder contents, respectively.

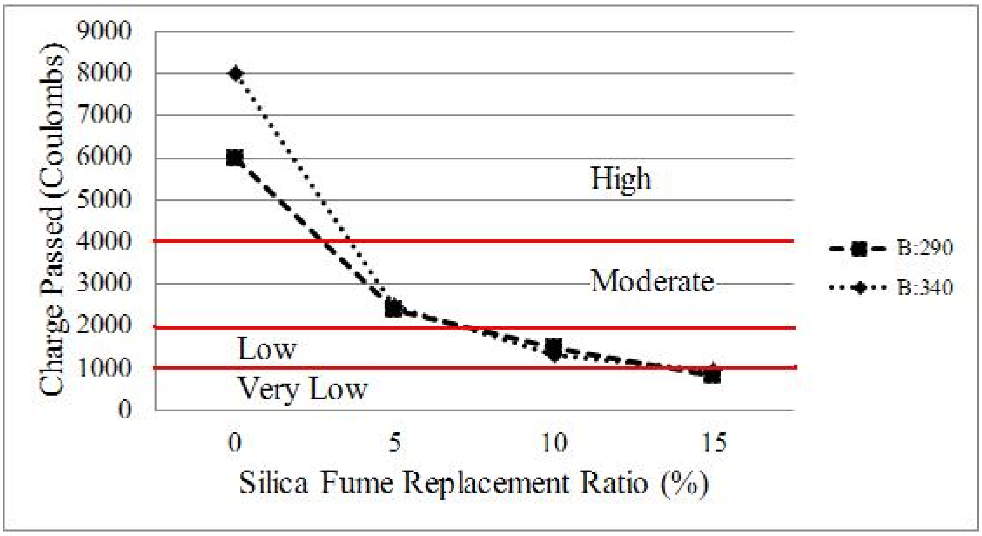


Figure 6: Rapid chloride permeability profiles for concretes.

Figure 7 illustrates the variation of the maximum depth of water ingress depending on the replacement ratio of silica fume for concretes produced with cementing materials contents of 290 kg/m^3 and 340 kg/m^3. According to the relevant standard TS EN 12390-8, concrete with a maximum depth of water ingress smaller than 50 mm is considered impervious. In the case of exposure to aggressive environment, this value is desired to be smaller than 30 mm.

As seen from Figure 7, the maximum depth of water for concretes containing cementing materials of 290 kg/m^3 exhibit a significant decrease as the silica fume replacement increases while the maximum depth of water ingress for concretes with cementing materials content of 340 kg/m^3 did not change with increasing silica fume replacement level. In other words, at high total binder contents, the silica fume replacement did not affect the water ingress. This finding is corroborated by the research [14] carried out on concrete of 0.5 water to binder ratio and a binder content of 330 kg/m^3.

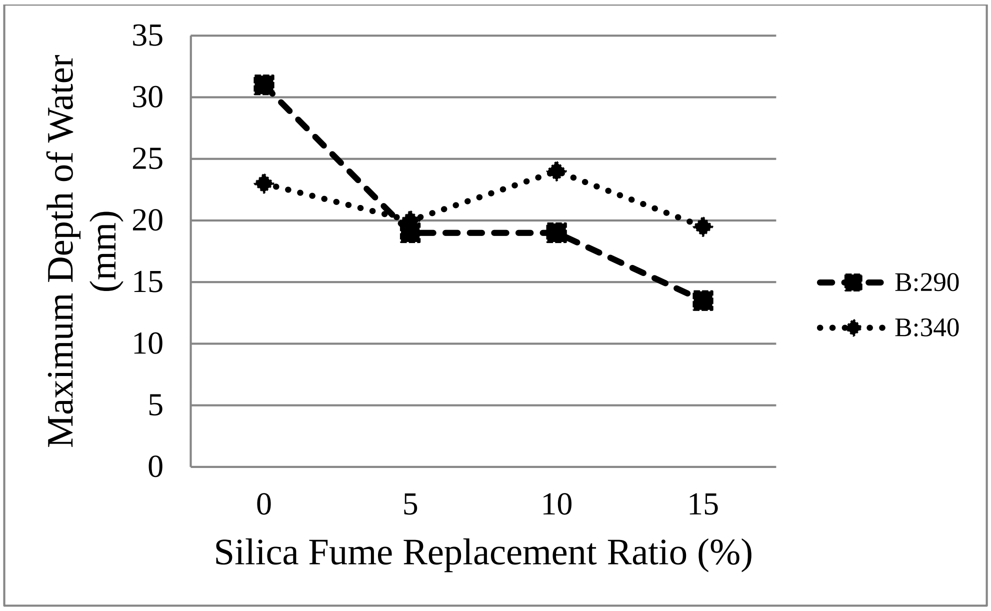


Figure 7: The maximum depths of water ingress measured for concretes.

The capillarity coefficients obtained on concretes are given in Figure 8. As can be seen from the graph, the capillary coefficients indicate a remarkable decrease as the silica fume replacement increases for both concretes containing 290 kg/m^3 and 340 kg/m^3 cementing materials.

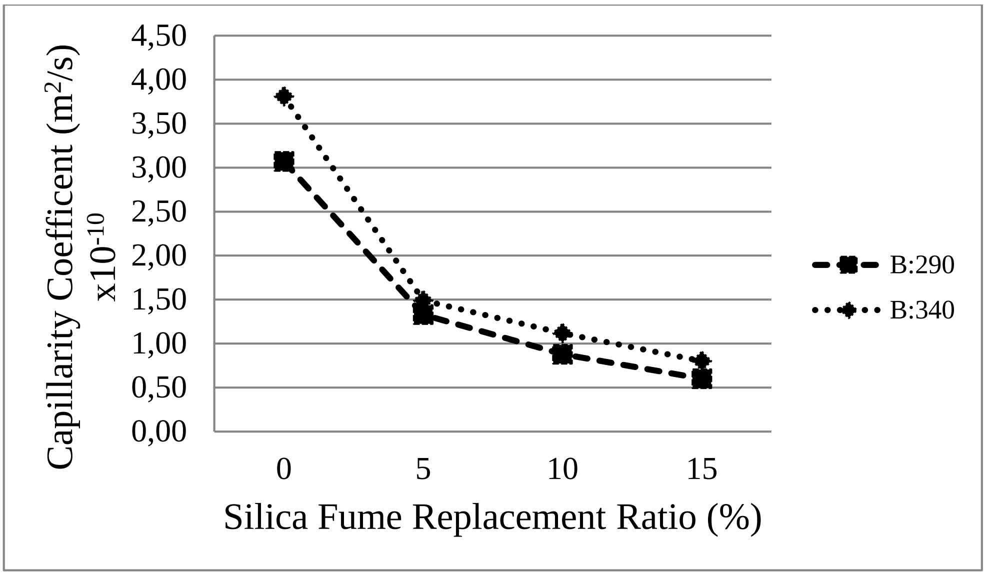


Figure 8: Capillarity coefficients measured on concretes.

The compressive strengths obtained on concretes are illustrated in Figure 9. As can be seen from the figure, a noticeable increase in the compressive strength of concretes is observed as the silica fume replacement increases. The tendency in the compressive strength increase is quite similar for both concretes containing 290 kg/m^3 and 340 kg/m^3 cementing materials. This result is corroborated by the results in reference [11].

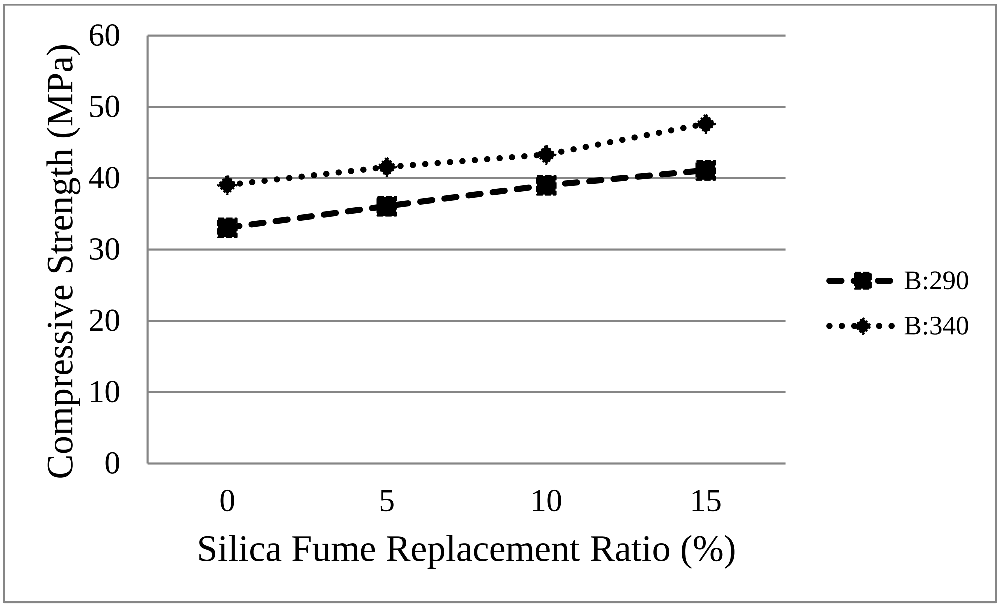


Figure 9: Compressive strengths versus silica fume replacement.

5. CONCLUSIONS

Based on the experimental study carried out, the following main findings may be outlined:

- The rapid chloride permeability values decreased remarkably for both concretes containing different binder contents as the silica fume replacement ratio increases. The reduction is much more noticeable for concrete with higher binder content.
- The reduction observed in the maximum water ingress depending on the increase in the silica fume replacement is more pronounced for concretes containing cementing materials of 290 kg/m^3. In view of impermeable concrete production, this indicates that silica fume is more effective in concretes of low binder contents.
- Depending on the increase in the silica fume replacement, the reduction observed in the capillary coefficient is significant for both concretes containing cementing materials of 290 kg/m^3 and 340 kg/m^3.
- The compressive strengths of concretes regardless of the cementing materials contents increase as the silica fume replacement ratio increases.
- Consequently, regardless of the cementing materials contents, the durability and the compressive strength properties of the concretes improved as the silica fume replacement ratio increased. It is obvious that silica fume replacement up to 5% to 10 % is favorable particularly for concretes exposed to environments containing sulfates and/or chlorides. This is also desirable so far as the sustainability is concerned.

REFERENCES

[1] Mehta, P.K. and Monteiro, P.J.M., "Concrete: Microstructure, Properties and Materials", 3rd ed., McGraw Hill, USA (2006).

[2] Baradan, B., Yazıcı, H. and Ün, H., "Beton ve Betonarme Yapılarda Kalıcılık (Durabilite)" 1st ed., THBB, İstanbul, Turkey (2010).

[3] Li, Z.J., "Advanced concrete technology", Hoboken: John Wiley & Sons (2011).

[4] Reiner, M., "Technology, Environment, Resource and Policy Assessment of Sustainable Concrete in Urban Infrastructure", Ph.D. Thesis, University of Colorado at Denver and Health Sciences Center, USA (2007).

[5] Yeğinobalı, A., "Silis Dumanı ve Çimento ile Betonda Kullanımı", 6th ed., TÇMB, Ankara, Turkey (2009).

[6] Khan, M.I. and Siddique, R., "Utilization of Silica Fume in Concrete: Review of Durability", *Properties, Resources Conservation & Recycling,* 57 (2011) 30-35.

[7] Sideris, K.K. and Savva, A.E. (2005) "Durability of Mixtures Containing Calcium Nitrite Based Corrosion Inhibitor" *Cement and Concrete Comp.*, 27 (2005) 277–287.

[8] Vu, K.A.T. and Stewart, M.G., "Structural Reliability of Concrete Bridges Including Improved Chloride-Induced Corrosion Models", *Structural Safety*, (2000) 22 (4) 313–333.

[9] Martys, N.S. and Ferraris, C.F., "Capillary Transport in Mortars and Concrete", *Cement and Concrete Research,* (1997) 27 (5) 747-760.

[10] Klinghoffer, O., Frølund, T. and Poulsen, E., "Rebar Corrosion Rate Measurements for Service Life Estimates" *Paper presented at the ACI Fall Convention Committee 365 ''Practical Application of Service Life Models''*, Toronto (2000).

[11] Ramezanianpour, A.A., "Effect of Curing on the Compressive Strength, Resistance to Chloride Ion Penetration and Porosity of Concretes Incorporating Slag, Fly Ash or Silica Fume", *Cement and Concrete Composites,* 17 (1995) 125-133.

[12] Gesoğlu, M., Güneyisi, E. and Özbay, E., “Properties of Self-Compacting Concretes Made with Binary, Ternay and Quaternary Cementitious Blends of Fly Ash, Blast Furnace Slag and Silica Fume”, *Construction and Building Materials*, 23 (2009) 1847-1854.
[13] Bagheri, A.E., Zanganeh, H. and Moalemi, M.M., “Mechanical and Durability Properties of Ternary Concretes Containing Silica Fume and Low Reactivity Blast Furnace Slag”, *Cement and Concrete Composites*, 34 (2012) 663-670.
[14] Uluöz, N., Özbebek, H., Arslan, M. and Açık, H., “Farklı Çimento Tipleri ve Mineral Katkılar ile Üretilen Betonların Mekanik ve Geçirimlilik Özellikleri”, *Beton 2011 Hazır Beton Kongresi, Bildiriler Kitabı,* İstanbul, (2011) 376-387.

ASSESSMENT OF CONCRETE DURABILITY OF NEW PANAMA CANAL LOCKS THROUGH THE USE OF RESISTIVITY

C. Andrade (1), N. Rebolledo (1), F. Tavares (1), R. Pérez (2) and M. Baz (2)

(1) IETcc-CSIC-Spain;

(2) GUPC: Grupo Unidos por el Canal (Sacyr)

Abstract

Panama Canal connects the Pacific Ocean and the Caribbean Sea, being a crucial shortening in the world navigation and international trade. Now 100 years old, the canal is insufficient for the new large boats. New parallel locks are being built by a consortium "*Grupo Unidos por el Canal*", GUPC, whose engineering division is headed by the firm Sacyr, S.A. The concrete on the new structure is reinforced as an anti-seismic precaution. In its specifications, the Panama Canal Authority, ACP, requires a 100-year service life for the so called marine concrete, defined to mean conformity with the 1000-coulomb electrical charge set out in ASTM 1202 and application of a reliable method for calculating service life.

In present paper some of the designed concrete mixes and their performance are described. As an alternative to the ASTM 1202 test, resistivity measurements and natural chloride diffusion tests were proposed. Additionally the service life calculation was made through a program with a numerical model. The relationship between electrical charge and resistivity values is discussed, along with the variation in these parameters over time, the chloride diffusion values and the "age factor" proved to have an even more critical effect on predictions than the diffusion coefficient. The use of the resistivity is a very practical manner to control concrete production due to its non-destructive character.

Keywords: Concrete, chlorides, resistivity, diffusion.

1. INTRODUCTION

Due to its strategic location Panama Canal has constituted a route of cardinal importance for world trade. It has around 80 Km in length going from one Ocean to the other: Atlantic and Pacific. As the Lake Gatun used as part of the navigation path inside the country is 26 m above sea water level, the canal was made by building three locks on each Ocean side to reach the lake level as indicated in figure 1. The locks have two parallel chambers in order to have the possibility of the simultaneous passing of two ships as figure 1 depicts. The maximum size of the ships is 294.1 m in boat sleeve, 32.3 m in boat length and until 12 m in ship draft (ships called Panamax).

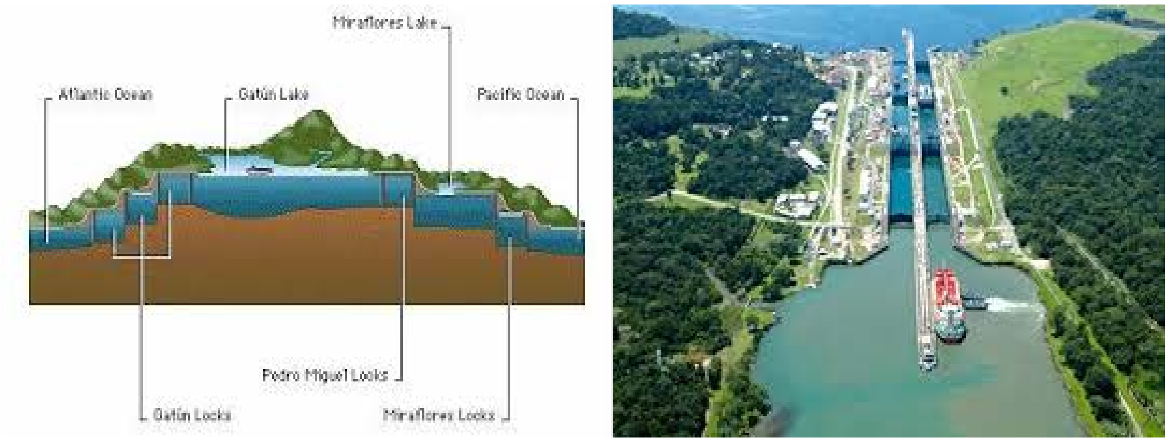


Figure 1: Profile of the hight of the inland Gatun lake an the need to build locks (right part) for the navigation.

With a view to an enlargement to accommodate deeper vessels, the Panama Canal Authority (Spanish initials, ACP) organised an international competition to build two new sets of locks. The works were awarded to the "*Grupo Unidos por el Canal*" (Spanish initials, GUPC), whose engineering division is headed by a Spanish firm, Sacyr S.A. The solution selected (figure 2) consisted for construction also contains 3 locks for each Ocean side entrance. With the construction of new chambers for the water recycling 60% of the water used will be saved. This recycling decreases the loss of fresh waters of Lake Gatun into Oceans. The end of the construction was commissioned for 2014 to celebrate the centenary of the old canal, but it was not achieved, being beginning of 2016 the foreseen end.

Figure 2: Proposed new locks with side water saving basins

The Panama Canal Authority’s, ACP, specifications require a 100-year service life for the concrete in all members, defined to mean conformity with the 1000-coulomb for electrical charge set out in ASTM 1202 [1] and application of a reliable method for calculating service life. They also establish a series of requisites to minimise heat of hydration-induced cracking in the concrete and cover depths of around 10 cm.

The works were begun in 2011, with a laboratory in charge of verifying compliance with the 1000-coulomb electrical charge and calculating the 100-year service life. After conducting initial trials with different mixes, in May 2011 the GUPC consortium contacted some of the authors of this communication to study the possibility of using alternative approaches to prove compliance with the specifications. The IETcc suggested: a) different prime materials and batching to manufacture a series of alternative concrete mixes; b) the testing of natural chloride diffusion (ponding tests) and continuous monitoring by measuring electrical resistivity over time; and c) the use of LIFEPRED, a numerical model [2]developed at the institute for calculating service life based on Fick’s law [3-7].

To predict chloride ingress is controversial in spite that several models were proposed in the last years [8]. Present paper describes the research conducted for some of the mixes designed in which the electrical charge and resistivity were obtained, as well as the diffusion coefficients by the ponding tests. The service life values yielded by the LIFEPRED numerical model and the resistivity model are also given. The relationship between electrical charge (in coulombs) and resistivity values is discussed [9-11], along with the variations in these parameters over time, the chloride diffusion values obtained in parallel and the "age factor” [12], whose effect proved to have an even more critical effect on predictions than the diffusion coefficient. The results indicate that continuous resistivity monitoring is a cost-effective way to ensure compliance with owners’ durability requirements.

2. EXPERIMENTAL

The methodology applied is illustrated here with only four mixes of each side of the country: A for Atlantic and P for Pacific (A3 and P3, A11 and P11, A50 and P50 and A56 and P56), representative of the over 50 studied. Their compositions are listed in Table 1.

Table 1: Composition of four concrete mixes studied

Plant		**Atlantic**				**Pacific**			
GUPC Code		**SMC-A3**	**SMC-A11**	**SMC-A50**	**SMC-A56**	**SMC-P3**	**SMC-P11**	**SMC-P50**	**SMC-P56**
Mix		**5796**	**1372**	**5777**	**6850**	**823**	**5067**	**5434**	**5481**
		375 (23PN 0SF)	**375 (15PN 5SF)**	**330 (12.5PN 5.8SF)**	**332 (13.0PN 0.0SF)**	**375 (23PN 0SF)**	**375 (15PN 5SF)**	**330 (12.5PN 5.8SF)**	**332 (13.0PN 0.0SF)**
Cement	Type	Panama CEM II	Panama CEM II	Panama CEM II	Panama CEM II	CEMEX CEM II	CEMEX CEM II	CEMEX CEM II	CEMEX CEM II
	kg/m^3	288.75	300	264	289	288.75	300	264	289
Pozzolan	kg/m^3	86.38	56.3	47	43	86.87	56.3	47	43
Silica fume	kg/m^3	-	18.8	19	0	-	18.8	19	0
w/c ratio	-	0.30	0.29	0.34	0.34	0.35	0.3	0.34	0.34
Superfine Sand	kg/m^3	4.5	4.5	49	49	4.31	4.5	49	44
Fine sand (0 – 4.75 mm)	kg/m^3	567.5 +206.25 =773.75	573+211 = 784	686	716	522.5+217.5 = 740.0	535+220 = 755	686	703
aggregate (4.75 – 19 mm)	kg/m^3	1296.25	1394	1324	1272	1321.25	1312	1354	1342
28-day strength	MPa	50.1 49.5	57.8 58.1	52.1 51.5	42.6 41.3	53 53.8	61.4 61.0	38.6 40.9	45.3 44.5

Cement type CEM II (ASTM) was used, with additions such as natural pozzolan and silica fume. A "superfine" fraction of sand with pozzolanic properties was also employed, although for batching purposes it was regarded as an aggregate rather than a mineral addition. The w/c ratio was on the order of 0.3. The 15x30-cm cylindrical specimens studied were prepared at a real plant in Panama, where they were cured in a humidity chamber for 28 days and later sealed in water-tight packaging for continued curing and shipped to Madrid, where they arrived 38 days after casting.

Upon receipt of the materials, three specimens of each composition were tested by means of the chloride penetration (ASTM C1543) [13] and for voids volume by ASTM C 642. *Porosity* by mercury intrusion porosimetry (MIP) was measured on a Micromeretics PORESIZER mercury intrusion porosimeter. *Concrete resistivity* was determined by Wenner four-point test, described in Spanish standard UNE 83988-2, was chosen (figure 3) [14]. The non-steady state diffusion coefficient and the chloride concentration on the concrete surface were calculated by applying Fick's second law of diffusion (Equation 1).

$$C_x = C_S \cdot \left(1 - erf\left[\frac{x}{2\sqrt{D_{ns} \cdot t}}\right]\right) \qquad (1)$$

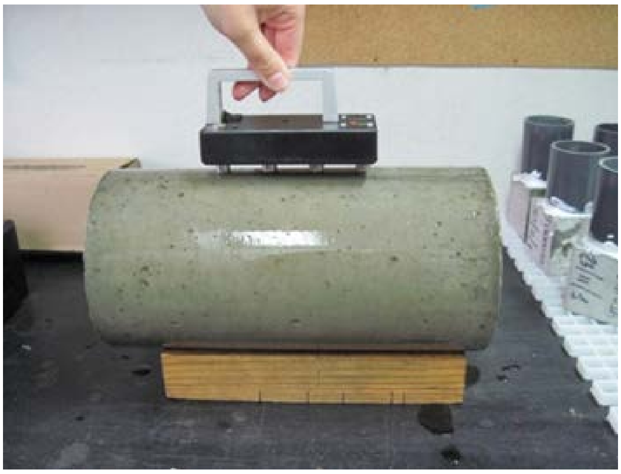


Figure 3: Resistivity measured with a four-point resistivimeter.

The salinity of the chambers is around 30 g/l in the wing walls in contact to the oceans, the between 15 to 20 g/l in the lower cambers, less than 5 g/l in the Moderate chamber and less than 1 g/l in the upper chamber.

The LIFEPRED model in one-, two- and three-dimensional versions were developed, although the 3-D version is not discussed hereunder. Unlike other models which assume a semi-infinite medium, LIFEPRED assumes a finite medium due to the presence of reinforcing steel. This characteristic induces substantial variations in the results if the cover is small because in that case the chloride accumulates on the bar surface. The program can also handle data on variations in temperature and environmental humidity. Either a characteristic or several "age" factors can be entered and the ceiling concentration can be visualised separately. The user interface, with very few data entry screens, is highly intuitive.

The operating sequence is as follows: a) generation of geometry and bar position; 2) determination of geometry mesh size; 3) data entry: material characteristics; 4) data entry: surface concentration, age factor and temperature; 5) processing; and 6) delivery of results in graphic form (Figure 2).

2.1 Resistivity-based model

The model developed [10, 12] to calculate the service life from resistivity, based on the 28-day value in water-saturated conditions was applied. The model was able to calculate the corrosion propagation period. For the present purposes, only the time to reinforcement depassivation, i.e., the first term in Equation 2, was used:

$$t_l = t_i + t_p = \frac{x^2 \cdot \rho_{ef} \cdot \left(\frac{t_a}{t_0}\right)^q \cdot r_{Cl,CO_2}}{F_{Cl,CO_2}} + \frac{P_{corr} \cdot \xi \cdot \rho_m \cdot \left(\frac{t_a}{t_0}\right)^q}{k_{corr}} \tag{2}$$

Where x= cover depth, ρ_{ef}= effective or nominal resistivity; ρ_m= mean resistivity in the

Specific climate; $r_{Cl,CO2}$ = chloride-cement paste reaction factor; $F_{Cl,CO2}$ = environmental factor that depends on the chloride content in the environment; q= age factor governed by progressive cement hydration; P_{cor}= depth of corrosion in the reinforcement bar; ξ = environmental factor that depends on climate and k_{cor}= constant that relates resistivity to corrosion rate.

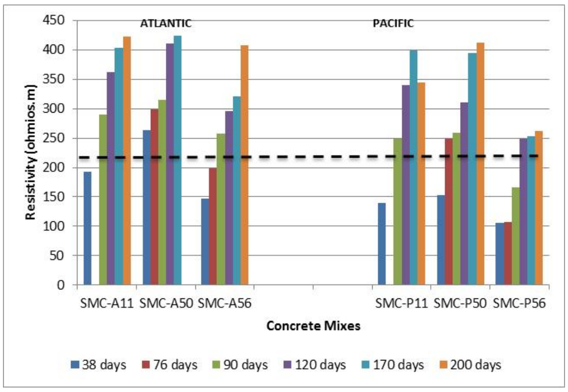
In addition to the resistivity value in water-saturated 28-day concrete, service life calculations with this model call for: 1) periodic resistivity readings to find the age factor, q; 2) calculation of the chloride reaction factor from the multi-regime test; and 3) identification of the environmental factor to be applied.

3. RESULTS

The resistivity values over time are shown in Figure 4 left, while Table 3 gives the age factors obtained from 38- and 120-day data. The 38- and 120-day diffusion coefficients are shown in Figure 3 right for all the mixes except A3 and P3, for which tests were conducted on the 14-month materials only.

Table 2: Age factor values found from variations in resistivity over time

38/120-DAY RESISTIVITY AGE FACTOR					
SMC-A11	SMC-P11	**SMC-A50**	**SMC-P50**	**SMC-A56**	**SMC-P56**
0.492	0.584	0.349	0.609	0.629	0.718



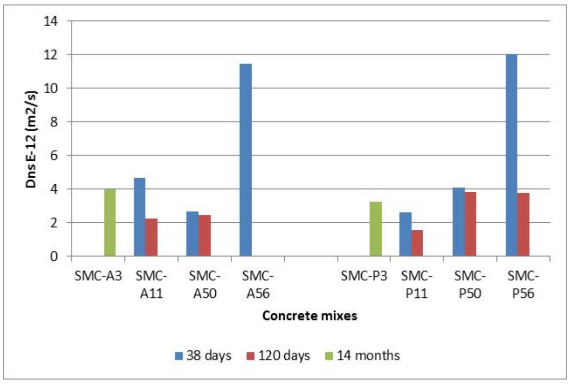


Figure 4: LEFT- Resistivity values up to 200 days and RIGHT- Diffusion coefficients at 38- and 120-day for all mixes except A3 and P3 (tested after 14 months only).

4. DISCUSSION

As noted earlier, further to the specifications, the only requirements to be met by the concrete were a 1000-coulomb at an undetermined age, and proof of a 100-year service life by application of a computer model. The two requirements were independent of one another and had to be confirmed simultaneously, subject to the condition that a 100-year service life could not be inferred from compliance with a maximum electrical charge of 1000 coulombs.

A different approach was submitted to prove these requirements 1) Based on the equivalence between electrical charge in coulombs determined as per ASTM standard 1202 and resistivity, the proposal entailed replacing continuous mix monitoring for the former with resistivity measurement which, as a non-destructive method, can be used to monitor one and the same specimen over time, 2) Natural diffusion measured in ponded specimens, not initially envisaged, was proposed as the source of actual chloride penetration to be entered in a model based on Fick's law of diffusion and 3) Lastly, in light of the relationship between resistivity and apparent diffusion coefficients, the resistivity age factor was confirmed to be equivalent to actual penetration measurements at several ages for service life calculation purposes, and to yield very similar values, in general. Hence, although a different methodology than specified in the ACP specifications for concrete was applied; the comparisons made showed that resistivity is a parameter apt for monitoring concrete durability in situ.

The relationship between the electrical charge in coulombs defined in the ASTM 1202 test and resistivity is graphed in Figure 5 for one of the samples: ρ (Ωm) = 200000/ Q.. The regression coefficient was generally high for the equation fitted to the data.

The relationship obtained for the apparent diffusion coefficient, D_{ns}, shown on the right in Figure 5, derives from Einstein's equation that relates diffusivity to conductivity [9]. On the other hand, the values found with LIFEPRED program for the mixes studied. Three cover depths were analysed: 100, 125 and 150 mm. The cover depth required to comply with the 1000-coulomb requirement (equivalent to 200 Ωm) could be found for each chamber or salinity level on the grounds of these calculations.

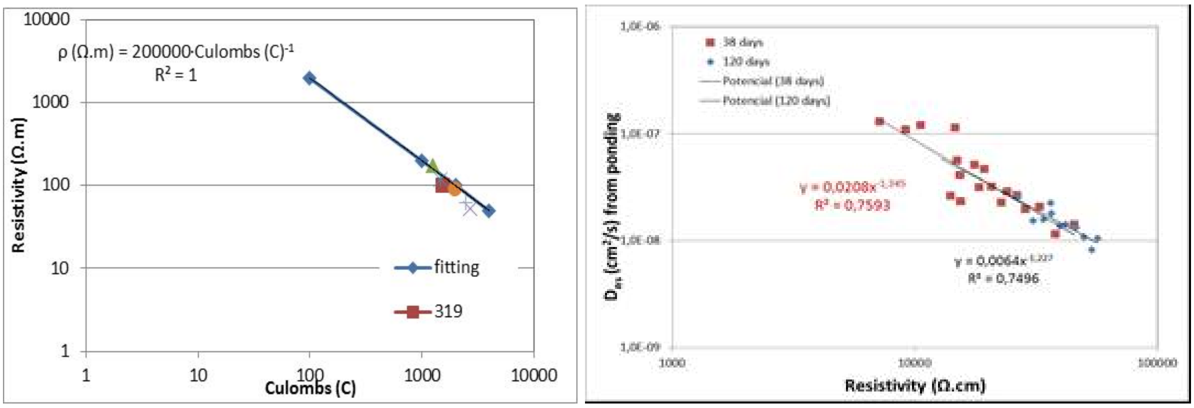


Figure 5: Left- relationship between electrical charge in coulombs and resistivity; and right- relationship between resistivity and the 38- and 120-day apparent diffusion coefficients.

Service life was also computed with the resistivity model (Equation 2) using only the initiation period, for the limit state defined by the ACP was the absence of corrosion after 100 years. As the hypothetical cases in Figure 6 shows, very similar results were found with the two models, which alternately and indistinctly delivered the more conservative prediction. The explanation is to be found in the predictive component of the resistivity model, which is based on the square root of time, therefore respecting the quadratic relationship between distance travelled and time.

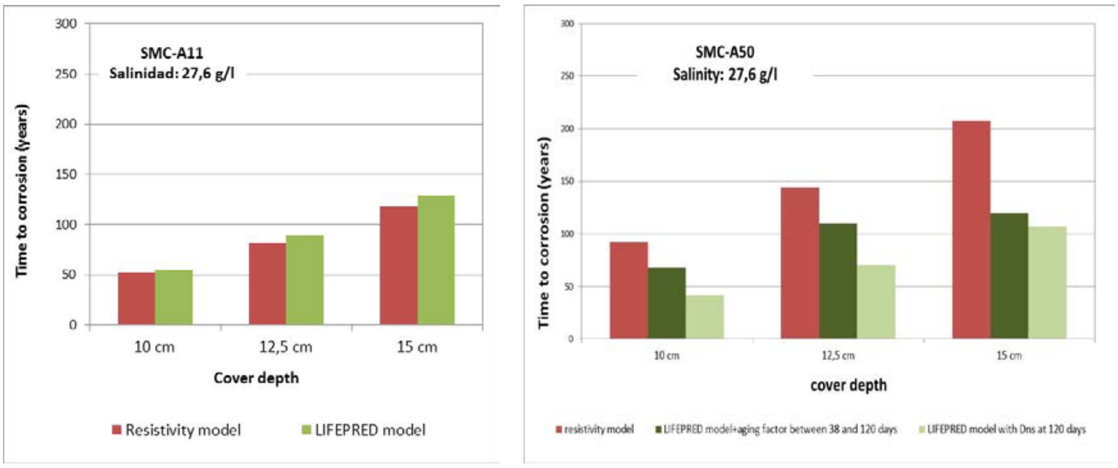


Figure 6: Left- comparison of the LIFEPRED and resistivity models for mix A11; right-comparison for mix A50 of resistivity model to LIFEPRED using the aging factor between 38/120-days and LIFEPRED using the D_{ns} value obtained from ponding at 120-days.

5. CONCLUSIONS

The conclusions that can be drawn up are of different nature. On the proposal to substitute the method of ASTM 1202 by the resistivity with a non-destructive character it can be deduced that:

- There is a constant equivalence between coulombs with resistivity. 200 Ωm = 1000 Q.
- Resistivity serves for:
  – the monitoring of the quality control of the concrete production
  – The monitoring of the aging factor
  – The verification on site of the fulfilment of the 200Ωm
  – The prediction of service life.
- Also there is an equivalence between resistivity and D_{ap} obtained by natural diffusion

With respect to the mixes tested:

- The concrete mixes adopted with the corresponding cover depth have shown > 100 years of life in the predictions

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance received from GUPC and CICP, a private company, in conducting the study, as well as ACP’s willingness to accept new approaches to service life calculations. Funding provided by the Spanish Ministry of Education and Science (now Ministry of the Economy and Competitiveness) for the Consolider SEDUREC project of “Safety and Durability of Construction Structures” under which some of the basic work to develop LIFEPRED was performed.

REFERENCES

[1] ASTM 1202 (2010). Standard Test Method for Electrical Indication of Concrete’s Ability to resist Chloride Ion Penetration. Annual Book of ASTM Standards.
[2] C. Andrade and F. Tavares LIFEPRED. Service Life prediction program. 2012.

[3] Tuutti, K., “Corrosion of steel in concrete”, Swedish Cement and Concrete Institute (CBI) nº 4-82. Stockholm (1982).
[4] Baroghel-Bouny V., Wang X. and Thiéry M. Prediction of chloride binding isotherms by analytical model or numerical inverse analysis-Proceedings of the 2nd Symposium on Service Life Design for Infrastructures. (DBMC Service life). Delft University of Technology. October (2010).
[5] Saetta A., Scotta R., Vitaliani R., - Analysis of chloride diffusion into partially saturated concrete24- ACI Materials Journal 90 (1993) 441–451.
[6] Sagüés A.A., - Modeling the Effects of Corrosion on the Lifetime of Extended Reinforced Concrete Structures -Corrosion -October (2003) 854-866.
[7] Mangat, P.S., Molloy, B.T-Predicting of long term chloride concentration in concrete. Materials and Structures, 27, (1994) 338-346.
[8] Gulikers, J. (2004), Critical evaluation of service life models applied on an existing marine concrete structure. -NORECON Seminar 2004: Repair and Maintenance of Concrete Structures, Copenhagen April 19-20, (2004).
[9] Andrade, C. (1993). -Calculation of chloride diffusion-coefficients in concrete from ionic migration measurements. -Cement and Concrete Research 23(3): 724-742.
[10] C. Andrade. 2004, Calculation of initiation and propagation periods of service-life of reinforcements by using the electrical resistivity. International Symposium on Advances in Concrete through Science and Eng., RILEM Symposium, March 22-24, Evanston (Illinois, USA).
[11] Andrade C., Whiting D. A comparison of chloride ion diffusion coefficients derived from concentration gradients and non-steady state accelerated ionic migration- Materials and structures, RILEM, vol 29, October 1996, pp476-484.
[12] C. Andrade, M. Castellote, R. D’Andrea - Measurement of ageing effect of chloride diffusion coefficients in cementitious matrices- Journal of Nuclear Materials, 412 (2011) 209-216.
[13] ASTM C1543- 10a Standard Test Method for Determining the Penetration of Chloride Ion into Concrete by Ponding
[14] Pr-UNE 83988 - 1 Test methods for concrete durability. Determination of the electrical resistivity. Four points or Wenner method.

A HOLISTIC PERSPECTIVE ON THE ROLE OF CONCRETE ADMIXTURES FOR SUSTAINABLE CONCRETE CONSTRUCTION

Ara A. Jeknavorian

Ph.D., Jeknavorian Consulting Services, Chelmsford, MA USA

Abstract

Chemical admixtures have long been known for the beneficial role they play in improving the engineering properties of concrete and mortar mixtures. Looking back over the past decades, the use of air entraining agents and superplasticizers can be considered among the major influences that have allowed extending the use of concrete for many diverse applications. Moreover, ACI 212.3R reports that chemical admixtures provide numerous benefits to concrete mixtures such as increasing compressive and flexural strengths at all ages and increasing durability through reduction in permeability. A holistic review of the numerous functions provided by chemical admixtures is discussed to help identify future admixture capabilities required for the design, placeability, and increased service life of sustainable concrete construction. Performance attributes of the next generation of chemical admixtures will include allowing the acceptable use of a wider range of aggregates and supplementary cementitious materials, and enabling more predictable plastic and hardened concrete properties.

Keywords: Chemical admixtures; superplasticizers; ASTM; polycarboxylates; manufactured sand; and clay

1. INTRODUCTION

Long before sustainability became a focus for the concrete construction industry, the use of chemical admixtures were a contributing factor for both economizing concrete mixtures as well as reducing their carbon footprint [Sabnis 2012]. The capability of water reducing admixtures to either increase concrete workability without a change in water or paste content, or reduce water content without a significant change in workability has been and continues to be the most commonly used capability of chemical admixture technology. Moreover, over the past decades, the application of chemical admixture technology has dramatically expanded to improve concrete rheology and durability as well as enabling the use of marginal quality concrete materials [Jeknavorian 1997].

The broad range impact of chemical admixtures relative to the production and performance of concrete mixtures is represented in Figure 1. From the perspective of sustainability of concrete structures, chemical admixtures play a critical role in improving durability by helping mitigate numerous factors that can contribute to the deterioration of in-place concrete due to physical and chemical distresses. For concrete producers, chemical admixtures enable a wide range of approaches and material options to achieve diverse design performance targets. Contractors particularly benefit from improved placement, setting, and finishability operations provided by chemical admixtures. Overall, chemical admixtures have become an essential component in concrete mixtures, helping to expand the concrete market by enhancing the economy, sustainability, and performance of concrete.

The rate of new admixture development and commercialization has outpaced the ability of various organizations to create standards that provide guidance and performance requirements [Nmai 2012]. For example, corrosion inhibiting admixtures were used in concrete construction projects long before the ASTM C1582 was an officially approved standard. To address this time gap between commercial use of new admixtures and the development of new standards, a new type "S", special performance, classification was added to the ASTM C494 Standard for Chemical Admixtures. The type S performance criteria provides users of such admixtures guidance on expected performance parameters such as set time, strength, length change, and freeze-thaw durability, until which time a standard can be developed specific to the performance of the new admixture. New chemical admixtures such as lithium salts for ASR control, viscosity-modifying admixtures (VMA), shrinkage reducing admixtures, and admixtures which provide primarily slump retention with minimal water reduction would typically be initially tested as Type S until a standard specific to the admixture's special performance is developed.

At the 2011 NRMCA Sustainability Conference [Jeknavorian 2011], this author discussed the following emerging admixture technologies that can lessen the impact of concrete construction on the environment: polycarboxylate (PC) - based superplasticizers, special-type slump retaining PCs, the concept of clay mitigating agents, admixtures for pervious concrete, nano-admixtures, and admixtures dispensed on Ready-mix truck for real time slump control [Jeknavorian 2011].

In this paper, the discussion on new, innovative commercial or near commercial admixtures is continued to include: (a) a review of the three-way benefit of normal/high range water reducing admixtures; (b) synergistic interaction of polycarboxylate-based superplasticizer with calcium salt-based accelerators for increased rate of cement hydration and early strength; (c) chemical admixtures to beneficiate sands; and (d) additives to mitigate undesirable adsorption capacity for carbon residue in fly ash.

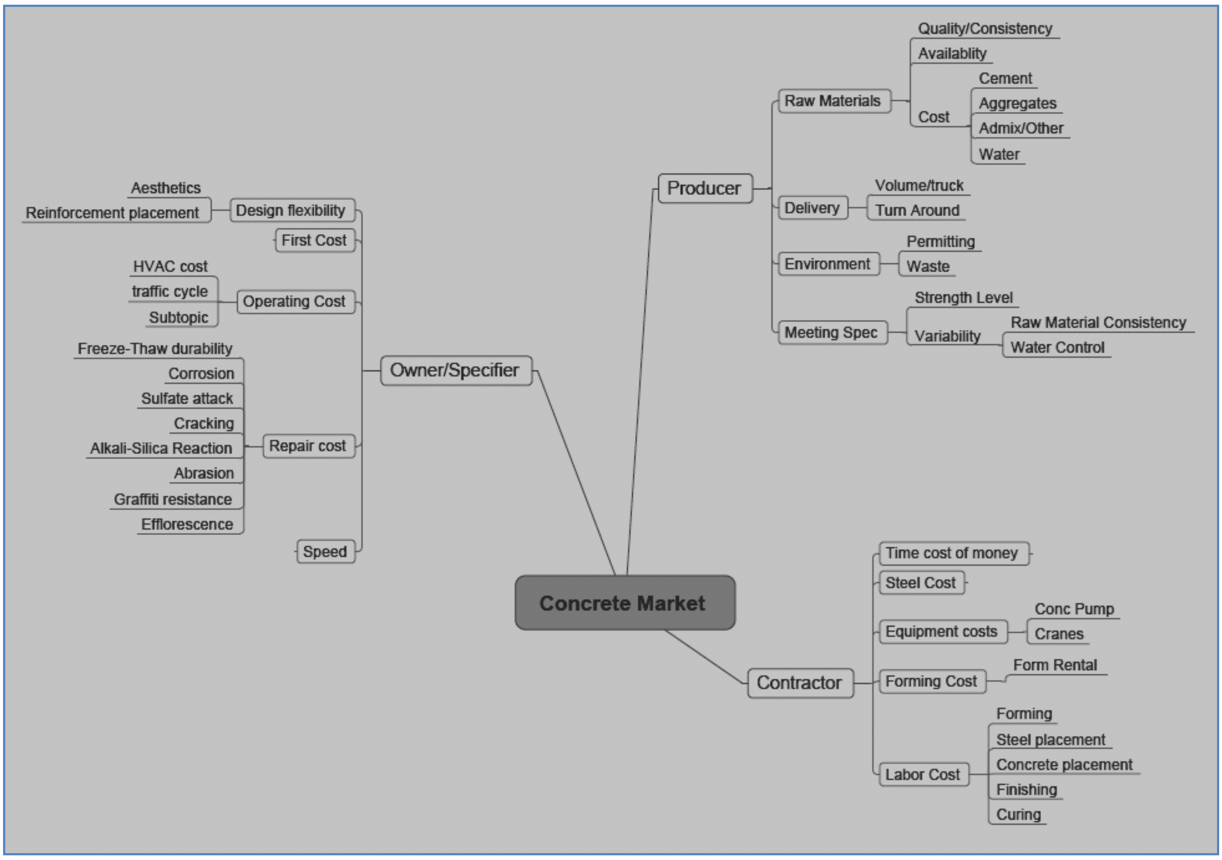


Figure 1: Impact of chemical Admixtures on Concrete Construction

2. IMPACT OF SUPERPLASTICIZERS ON CONCRETE SUSTAINABILITY

The benefits of high range water reducing agents (also known as superplasticizers) to improve the mixing, placing, compaction, finishing, strength, and durability of concrete mixtures has become well established. To affect these procedures and properties, superplasticizers are used to either reduce large amounts of mixing water while maintaining an acceptable level of workability, or transform very low slump concrete into a flowing or self-leveling mixture. Reducing the water/cement ratio (W/C) can produce a densified paste structure and improve paste-to-aggregate bonding, which in turn, increases the strength development of concrete and enhances resistance to chemical attack [ACI 212.3R-10]. By far, the ability to lower the mix water content for essentially all concrete mixtures without significantly altering the workability of the concrete is the most common application of chemical admixture technology. The mix designs and performance data reported in Table 1 illustrate the three common applications of superplasticizers. These applications are made possible by the cement de-flocculating mechanism imparted by water reducing admixtures. The flocculation of cement particles upon mixing with water is essentially common to all Portland cements based on the nature of surface charges imparted during the clinker grinding process. Once the hydrating cement particles deflocculate, the water released from the floc can be reduced to either achieve increased strength and reduced permeability, thus allowing for cement replacement with supplementary cementitious materials or even reducing the cement content. For the increased workability application, achieving higher slumps without a superplasticizer would require proportionally higher cement and water contents.

Table 1: Impact of Superplasticizers on the Workability and Compressive Strength of Concrete Mixtures. Quantities expressed in SI units.

	Reference	**High Strength**	**Flowing Concrete**	**Cement Reduced** Mix
Cement	356	356	356	267
Sand	712	742	772	845
Stone	1127	1216	1068	1187
Water	178	133	178	133
HRWR, l/m^3	-	3.5	3.5	2.6
W/C	0.50	0.38	0.50	0.50
Slump, mm	115	125	240	125
Compressive Strength, MPA				
1-day	9.7	19.2	11.9	10.5
7-day	28.3	39.4	31.2	29.5
28-day	35.3	46.8	38.3	36.8

3. SYNERGISTIC INTERACTION BETWEEN CHEMICAL ADMIXTURES: OPPORTUNIUTY FOR LOW CEMENT CONTENT CONCRETE

An example of a relatively unexpected performance demonstrated by polycarboxylates involves an interesting synergy with calcium-based set accelerators. The following data, summarized in Table 2, was reported by a prestress concrete producer, who had been using a combination of a traditional ASTM C494 type A water reducer and a Type G NSFC/Lignosulfonate-based superplasticizer in a 390 kg/m^3 concrete mix for pre-stress piles. This remarkable strength difference, obtained by merely changing the superplasticizer type from an NSFC to polycarboxylate, has become an accepted benefit for those producing concrete with these additives, and was the focus of major study [Jeknavorian 2000]. Interestingly, the strength difference does not seem be associated with increased heat of hydration, but rather is related to a denser microstructure produced by the polycarboxylate-based admixture. The increased compressive strength from this admixture combination can enable higher cement substitutions with SCMs with minimal delay in time of set and early strength gain. In Table 3, a series of laboratory mixes was prepared to demonstrate how this admixture synergy could nearly restore both time of set and early strength for a concrete mixture with 40% cement replacement with fly ash relative to a reference mix without fly ash. The last mix in Table 3 with both a PC-based HRWR and calcium nitrite corrosion inhibitor has a time of set just one hour longer than the reference mix (mix 1), and a one day compressive strength 85% of the mix 1.

4. CHEMICAL ADMIXTURES TO BENEFICIATE IMPACT OF MANUFACTURED SANDS

Aggregate shape, texture, and grading have been known to have a significant effect on the rheological performance of fresh concrete. Moreover, while the optimization of aggregate selection can provide both technical and economical benefits, the availability of materials and construction operations can often dictate the use and proportioning of certain aggregate sources, such as manufactured sands, which can adversely impact the rheology of cementitious mixtures.

Table 2: Synergistic Interaction of PC HRWR and Calcium-based Accelerators on Compressive Strength PC. Mix Design: Cement T-IIM - 390 kg/m3 (658 pcy); W/C - 0.32

	NSFC/WR	PC
PC, ml/100 kg	-	455
NSFC, ml/100 kg	1300	-
WR, ml/100 kg	130	-
CANI, l/m3	26.6	26.6
VR AEA, ml/100 kg	78	39
Slump. mm	75	115
Air, %	5.4	5.5
Initial set, hr:min	3:50	2:30
1-day strength, MPA	32.4	43.1

CANI – 32% solution of calcium nitrite
VR – Vinsol Resin-based air entraining agent
WR – ASTM C494 Type Water Reducing Agent

The use of certain chemical admixtures, such as viscosity modifying admixtures, have been found to often avoid the need to increase cement and water contents in order to overcome the loss of workability that can accompany aggregate sources which feature flat, elongated, angular, and rough particles [Jeknavorian 2006]. Figure 2 illustrates the effect of VMAs on reducing pump pressures that can occur with increased use of manufactured sands. The reduced pump pressures result from the lubricating effect of the paste which is maintained between the sand particles through the viscosifying effect of the VMA.

Table 3: Effect of Cement-Fly Ash-Admixture Combinations on Concrete Performance 420 kg/m^3 Total Cementitious

	Fly Ash (Class F)	Water	Admixture	Slump	Air	Initial Set	Final Set	Comp. Strength 1-Day	7-Day	28-Day
	% replace	w/c	%solids/cm	mm	%	(hr:min)	(hr:min)	mpa	mpa	mpa
Baseline	0	0.50		140	1.5	4:22	6:33	7.0	19.6	27.5
+ fly ash	40	0.50		215	0.9	9:20	13:01	3.1	11.7	16.9
+6% water cut	40	0.46		145	0.9	8:27	11:59	3.4	13.8	19.4
+18% water cut	40	0.38	0.13%PC-500	145	3.2	7:48	10:59	5.5	22.1	28.2
+CANI	40	0.38	0.13%PC-500 2.0% Ca Nitrite	165	3.6	5:20	8:15	6.0	24.3	30.1

5. CHEMICAL TREATMENT OF MANUFACTURED SANDS FOR CLAY MITIGATION

The presence of fine clays can be particularly harmful to concrete mixes because of their ability to absorb large amounts of mix water. This can lead to reduced workability, or reductions in strength due to extra water required to increase workability and increased drying shrinkage cracking. A new chemical solution (a clay mitigating agent or CMA) has recently been developed which attaches itself to clay particles and minimizes excess absorption of water [Kyriazis 2011]. The performance of the CMA can be inferred from the reduced MBV values illustrated in Figure 3.

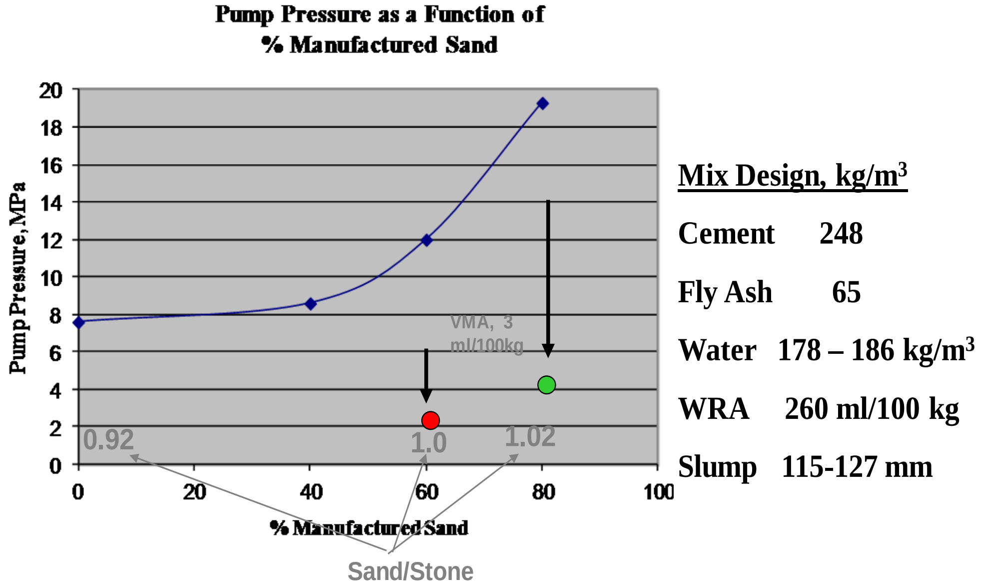


Figure 2: Effect of VMAs and Manufactured Sands on Pump Pressure

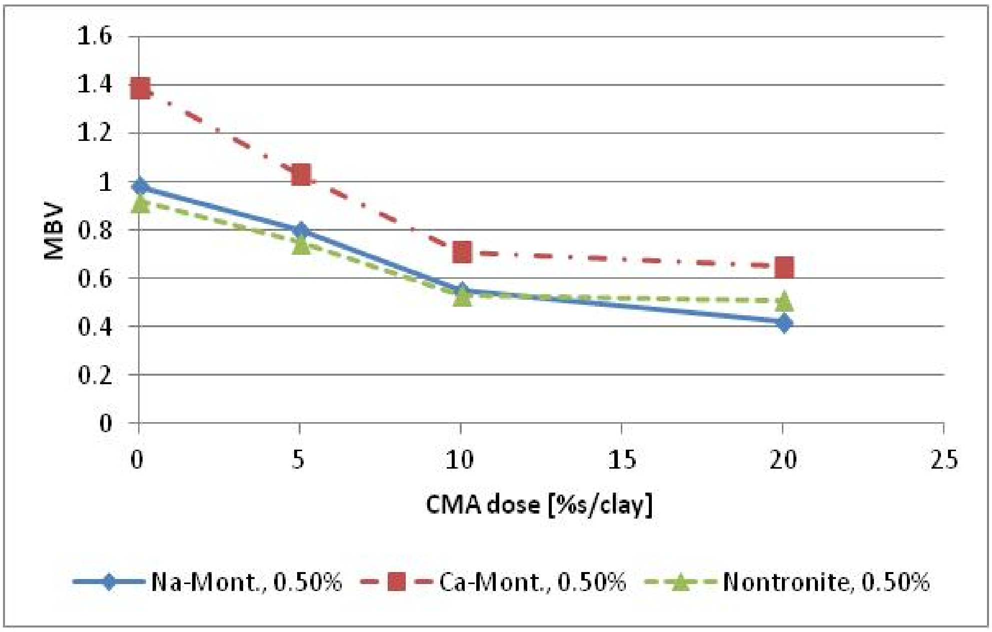


Figure 3: Effect of a CMA on the MBV

The benefits of CMA for concrete are shown in Figures 4 and 5, where laboratory concrete tests were performed comparing untreated versus treated sands with the clay mitigation chemical. Concrete mixes were identical except for the fine aggregate and water content. Water was adjusted to obtain similar slumps [90 mm +/- 5 mm]. As more washed sands are replaced with unwashed sands, the water demand of the untreated sand increases, leading to subsequent decreases in strength. Using the CMA, the w/c is maintained similar to the reference mix, which only contains 20% unwashed sands. Up to 80% replacement with untreated sand was possible without any detrimental effects.

6. CHEMICAL TREATMENT OF FLY ASH FOR CONSISTENT AIR ENTRAINMENT

With the expected increased use coal ash for the concrete construction industry, the impact of fly ash quality on concrete performance will continue to be an important factor in controlling the rate of cement replacement in concrete mixtures. The impact of the variability in the residual carbon content on air entrainment is one of the most significant parameters of concern. Chemical treatments, comprised of competitive agents capable of being preferentially adsorbed onto the fly ash carbon content of the fly ash, has been the focus of recent research activity

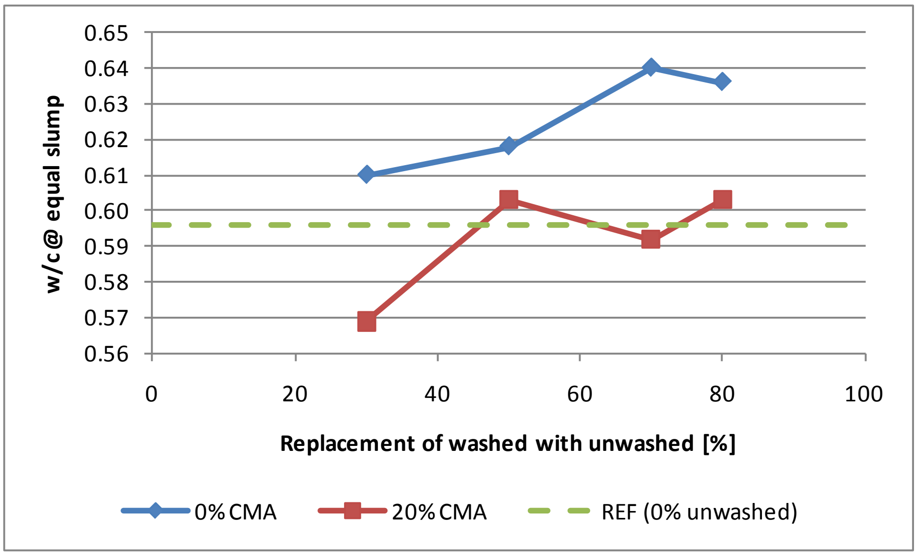


Figure 4: Effect of chemical treatment on w/c with increased replacement of washed with unwashed sands.

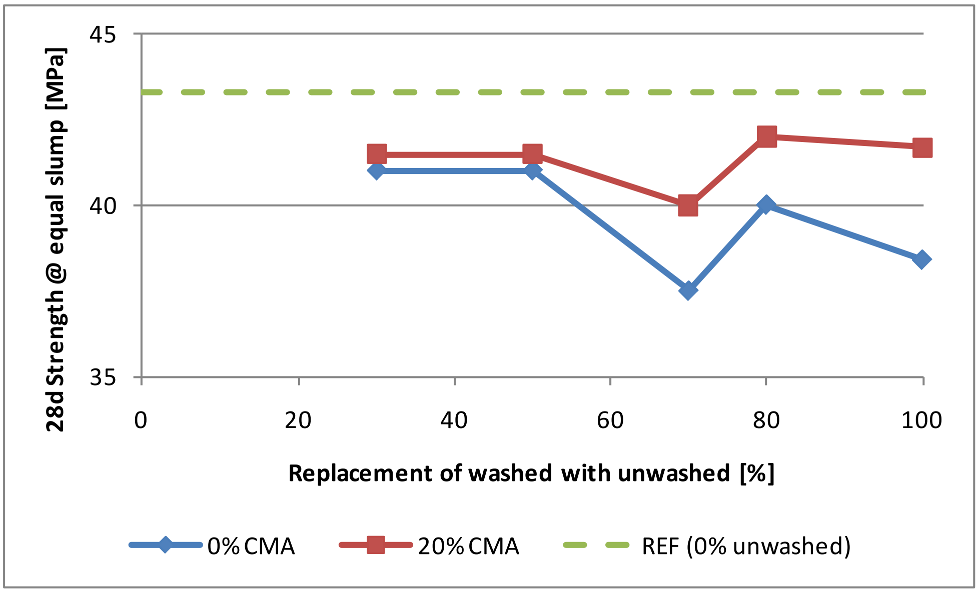


Figure 5: Effect of chemical treatment on strength with increased replacement of washed with unwashed sands

[Howard, Jolecouer,]. The requirements of such agents are as follows: (a) neutral impact on air entrainment; and (b) more rapid irreversible adsorption versus air entraining agents and other materials used in chemical admixture formulations. Furthermore, a simple procedure needs to be available that allows rapid determination of the amount of treatment chemical required to sufficiently neutralize the adsorption capability of the fly ash carbon content. Also, a preferred mode of addition needs to be established which assures optimum batch-to-batch consistency in air content at time of concrete discharge. Some promising technologies may soon be commercialized, which are expected to significantly facilitate increased fly ash use in concrete construction.

7. CONCLUSION

Considering all the trends associated with the concrete construction industry, the use of chemical admixtures is expected to play an increasingly important role in terms of mixture design, rheology control to facilitate placing, and long term durability. Many new technologies such as polycarboxylate-based superplasticizers have dramatically improved both production efficiency and quality of concrete, thus gaining rapid acceptance by concrete producers. Many exciting challenges and opportunities lie ahead for the continuous evolution of concrete as a versatile and durable construction material. The increased development and rapid commercialization of innovative chemical admixtures is expected to play a key role in the future of sustainable concrete.

REFERENCES

[1] Sabnis, G., *Green Building with Concrete, Sustainable Design and Construction,* CRC Press, 2012.

[2] Jeknavorian, A., Roberts, L.R., Jardine, L., Koyata, H., and Darwin, D., *Condensed Polyacrylic Acid-Aminated Polyether Polymers as Superplasticizers for Concrete,* Fifth CANMET/ACI Conference on Superplasticizers and Other Chemical Admixtures in Concrete, Venice 1997.

[3] Jeknavorian, A.; Berke, N.; Connolly, J.; and Prescott, J., *Synergistic Interaction of Condensed Polyacrylic Acid-Aminated Polyether Superplasticizer with Calcium Salts,* SP-195: The Sixth Canmet/ACI Conference on Superplasticizers and Other Chemical Admixtures in Concrete, SP 195, 2000, 585-600.

[4] Nmai, Charles, *C494 Standard on Chemical Admixtures*, SP-288, Tenth International Conference on Superplasticizers and Other Chemical Admixtures, 2012,

[5] Jeknavorian, A. *Chemical Admixtures of the Future: Opportunities and Challenges for Sustainable Concrete Production, Placement, and Service Life,* NRMCA International Concrete Sustainability Conference, Boston, 2011.

[6] ACI 212.3R-10 Report on Chemical Admixtures for Concrete

[7] Jeknavorian, A., Hazrati, K., Bentur, A., Koyata, H., McGuire, D., and Sandberg, P., *Use of chemical admixtures to modify the rheological behavior of cementitious systems containing manufactured aggregates,* American Concrete Institute, SP-239, Eighth CANMET/ACI International Conference on Superplasticizers and Other Chemical Admixtures in Concrete, 2006, 15-29.

[8] Kyriazis, A., Perry, B., Koeheler, E., and Bablouzian, L., US20130263761 A1 *Clay-Bearing Manufactured Sands for Hydratable Cementitious Compositions* 2011.

[9] Howard, C., Kabis, S., and Farrington, S., *Advances in Chemical Beneficiation of Fly Ashes Containing Natural Carbon or Powdered Activated Carbon,* 2013 World Coal Ash (WOCA) Conference, Lexington, KY, 2013.

[10] Jolicoeur, C., To, TC, Benoit, E., Hill, R., and Zhang, Z. , *Fly ash-carbon effects on concrete air entrainment: fundamental studies on their origin and chemical mitigation,* 9th ACI international conference on superplasticizers and other chemical admixtures in concrete, Seville, Spain: ACI, SP-262. 2009. ,

FROST RESISTANCE AND WATER IMPERMEABILITY AS MAIN FACTORS OF CONCRETE DURABILITY IN RUSSIAN EXPOSURE

S.A. Podmazova

PhD, Chief Researcher, Institute Betona NIIZHB

Abstract

Research on the dependence on strength of the water tightness and frost resistance of concrete has been carried out in the Scientific and Research Institute on Concrete – NIIZHB, Moscow, Russia. The results of research have practical application when designing the concrete compositions for different types of structures. The production requirements for concrete where strength is specified have been formulated.

On the basis of research results, tables showing the main characteristics of concrete which for it is necessary to specify concrete properties depending on service conditions of structures have been suggested.

Keywords: Concrete strength, water tightness, frost resistance, durability

1. INTRODUCTION

In the Russian Federation since the 1990s the ratio between production of precast and monolithic concrete has changed towards increase of production of concrete for monolithic construction - from ready mix concrete suppliers.

The structure of production of RMC has also lately undergone changes: the concrete compressive strength class increased from C20 to C40, C60 and even sometimes to C100. High strength concrete demands closer attention to concrete composition and production technology of concrete mixes as well technology of concrete work at the construction site.

At the stage of structural design the designer specifies the necessary requirements to concrete including concrete strength class and, depending on building service conditions – concrete classes (grades) of water tightness and frost resistance. However, requirements specified in the project for these characteristics are often not coordinated: concrete strength, water tightness and frost resistance. To meet all design requirements, on a concrete mixing plant the mix compositions are specified then corrected by trial batches. Sometimes, the designer can specify a strength class, for example C15, and a water tightness grade of W8. The B15 strength class concrete will not provide specified grades of water tightness of W2 and higher. This means that for such concrete, to obtain all design characteristics it is necessary to achieve some more significant concrete characteristics, so that other specified requirements will also be met.

Research in recent years has been carried out to establish and confirm relationships "water tightness - strength" and "frost resistance - strength".

The results of this research has wide application and have started to be used when developing production norms, and subsequently these technological parameters have been presented in tables in the standard *«Concrete for bridge structures. Specifications»*. For the first time, in the normative document design requirements for the production of concrete are specified, both for strength, and for water tightness and frost resistance.

According to the requirements of Russian Norms and Regulations 2.03.11-85 *«Protection of construction structures against corrosion»*, the protection of concrete structures against corrosion in different environments first depends on implementation of requirements for water tightness and frost resistance.

To provide these indicators of quality, it is necessary when developing a project for a building or a structure to specify increased requirements for strength, in order to achieve the required values of water tightness and frost resistance. Thus, tables 1 and 2 show how it is necessary to specify design strength of concrete to also provide the required parameters of water tightness and frost resistance. Using data in these tables, recommendations are made for limiting values of the technological parameters which will provide necessary protection, depending on the aggressive action resulting from environmental exposure conditions.

If at design stage of structures and buildings, it is necessary to specify classes of durability, and grades on frost resistance and water tightness, the data specified in table 3 and table 4 could be used.

All these data should be entered into the project and in the works project; the concrete manufacturer, having received the order for concrete of the set strength, when selecting the concrete compositions, can determine the technological parameters of the mix. As a result, the necessary strength of a building or a construction will be provided.

European standard EN206 *«Concrete. Specification, performance, production and conformity"* recommends limiting value for composition and properties of concrete at which required performance of concrete in this or that environment of service can be provided, including freeze/thaw resistance, (table F1). However, in this European standard, classification of concrete by water tightness and the frost resistance are absent. Therefore, requirements of this table cannot be transferred directly to Russian construction norms and regulations. Modified table F1 (table 3) was developed, which could be used in a Russian version of EN 206 if it is accepted as a national standard in the future.

Table 1: Specified parameters to ensure water proofing resistance of concrete

Technological indicators	Waterproofing grade, W , atm					
	4	6	8	10	12	14
Concrete class, in	20	25	30	35	40	45
Water-cement ratio, W / C	≤ 0,6	≤ 0,55	≤ 0,45	≤ 0,4	≤ 0,38	≤ 0,35
Chemical additives or organo-mineral	Water reducing / plasticizing				water reducing / plasticizing; or increase the density of the concrete (reducing permeability)	

Table 2: Specified parameters to ensure frost resistance of concrete

Parameters to ensure frost resistance of concrete technological factors	frost resistance grade (freeze/thaw cycles)			
	$F_1$75 - 100*	$F_1$200 - 300*	$F_1$400 - 600*	$F_1$700 - 1000*
	-	$F_2$100**	$F_2$200**	$F_2$300-500**
Concrete class, B (C)	≥20	≥25	≥27,5	≥30
Cement consumption, kg / m^3	>300	>320	>340	>360
water-cement ratio	<0,6	<0,55	<0,5	<0,45
Admixtures according to GOST 24211	Water reducing / plasticizing	Air-entraining and water reducing / plasticizing		
Air entrainment,%	-	3 - 7		
- * Frost resistance of all types of concrete, except concrete road and airfield pavements, F_1 **- **** Морозостойкость бетонов дорожных и аэродромных покрытий, F_2.				

*Note: Frost over F1-frost+fresh water, F2-frost+saltwater for all types of concrete, air-entrainment required.

Cement type is determined by the grade of frost resistance. When frost resistance grade F1200 (F2100) cement type should meet the requirements of GOST 10178, GOST 22266 GOST 31108, or the content of tricalcium aluminate (C3A) should be not more than 8% and the amount of mineral additives (slag) not more than 15%.

Table 3: Exposure Classes

	Exposure classes																	
	No risk of corrosion or attack	Carbonation-induced corrosion				Chloride-induced corrosion						Freeze/thaw attack				Aggressive chemical environments		
						Sea water			Chloride other than from sea water									
	X 0	XC 1	XC 2	XC 3	XC 4	XS 1	XS 2	XS 3	XD 1	XD 2	XD 3	XF 1	XF 2	XF 3	XF 4	XA 1	XA 2	XA 3
Max w/c	-	0,65	0,60	0,55	0,50	0,50	0,45	0,45	0,55	0,55	0,45	0,55	0,55	0,50	0,45	0,55	0,50	0,45
W, and frost, F, resistance	-	**W2**	**W4**	**W6**	**W6-W8**	**W6-W8**	**W8**	**W8**	**W6**	**W6-W8**	**W8**	**$F_1$200**	**$F_1$300**	**$F_1$400-$F_1$600**	**$F_1$700-$F_1$1000**	**W6**	**W6-W8**	**W8**
Min.strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C 35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Min. cement content, kg/m3	-	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360

Table 3(cont.): Exposure Classes

Min. air content, %	-	-	-	-	-	-	-	-	-	-	-	-	4,0[a]	4,0[a]	4,0[a]	-	-	-
Other requirements	-	-	-	-	-	-	-	-	-	-	-	Aggregate in accordance with EN 12620 with sufficient freeze/thaw resistance				-	Sulfate-resisting cement[b]	

Notes to table 3:

a Where the concrete is not air entrained, the performance of concrete should be tested according to an appropriate test method in comparison with a concrete for which freeze/thaw resistance for the relevant exposure class is proven.
b Where sulfate in the environment leads to exposure classes XA2 and XA3, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate or high sulfate- resisting cement should be used in exposure class XA2 (and in exposure class XA1 when applicable) and high sulfate-resisting cement should be used in exposure class XA3.
c Where the A-value concept is applied the maximum *w/c* ratio and the minimum cement content are modified in accordance with 5.2.5.2.
Waterproofing grade, W, is determined by the method of GOST 12730.5-84 "Concrete. Methods for determination of water pressure resistance "; frost resistance grade, F, is determined in accordance with GOST 10060-2012 "Concrete. Methods for determination of frost resistance. "

REFERENCES

[1] Guide to selection of structures of normal concrete. - M: Stroyizdat, 1979.
[2] Recommendations about selection of compositions of normal and fine-grained concrete (to GOST 27006-86), 1990.
[3] Standard 40619399-001-2010 Concrete for bridge structures. Specifications.
[4] Set of rules of the joint venture 28.13330.2012 Protection of construction structures against corrosion. The Construction Norms and Regulations SNiP 2.03.11-81

AEROGEL-INCORPORATED CONCRETE: MECHANICAL ANALYSIS AND DURABILITY PERFORMANCE

Savaş Erdem (1) and Ezgi Gürbüz (1)

(1) Istanbul University, School of Civil Engineering, Istanbul, Turkey

Abstract

There has been an exponential increase in the use of nanotechnology-based additives in concrete. One nanotechnology-based additive that deserves the attention of researchers is granular aerogel with extremely low density. In this study, the influence of different percentages (% 0,25 , % 0,5 , % 0,75 by volume) granuler aerogels on the mechanical, dynamic and durability properties of structural concrete has been studied through measurement of compressive strength, fracture energy under impact loading and water permeability. The experimental results show that the aerogel usage up to a certain percentage not only reduces the unit weight of the concrete but also provides significant improvements on the strength of concrete and the permeability. However, aerogel usage has caused brittle fracture of concrete comparing to control concrete when being subjected to short-duration dynamic loading. Finally, findings from this study will provide the dissemination of aerogel containing smart concrete technology.

1. INTRODUCTION

Aerogels are foams with transparent, highly porous, open cell, low density. Their microstructure, comprised of connected host particles and nanoscale pores, as well as elemental composition can be tailored by solution chemistry through a process known as sol-gel method. As a result of this special microstructure, these lightweight materials exhibit many interesting and unusual features [1].The density of the aerogel consisting of air spaces constituting 94-98 % of total volume, changes between 3–100 kg/m^3 depending on the porosity. It has properties of low thermal conductivity (0.003–0.02 W/mK) and a good level of fire and sound resistance [2].

The weight of structure can be reduced by decreasing the self-weight of reinforced concrete by means of reducing unit weight of normal concrete having a high coefficient of thermal conductivity because of high unit weight. Also an economical design can be made by reducing cross section of carrying elements. Also, by reducing the unit weight of concrete, the coefficients of heat conductivity and thermal expansion decrease and fire resistance increase. However, because of the amount of space in concrete increases, strength of the concrete decreases, corrosion resistance reduces and sensitivity to moisture increases [3].

It is possible to obtain heat insulated light weight concrete by using aerogel with concrete together because of that aerogel is almost entirely composed of air. Also, although aerogel is the lightest material of the world, it has carrying capability of weight up to 1600 times of its density [4]. This property may allow light weight concrete to increase its low resistance caused by spaces in it. Because of the aerogel can be synthesized as both superhydrophobic and superhydrophilic, it may have a positive effect on the impermeability of the concrete against water and harmful liquids.

In this study, the effects of hydrophilic aerogel as an additive on the compressive strength, impact resistance and water permeability of concrete have been experimentally investigated.

2. EXPERIMENTAL STUDIES

The physical properties of the granular aerogel used is given Table 1. The results indicate that the aerogel is a nano-porous material with over 94 % of the volume being air voids leading to very low density.

Table 1: Technical properties of the aerogel used

Hydrophilic aerogel	
Surface area	790-840 m^2/gr
Diameter	8-10 nm
Porosity	> %94
Apparent porosity	90-100 kg/m^3
Surface chemical groups	-OH

The commercially available hydrophilic aerogel granules are used in this investigation, as schematically shown in Figure 1.

Figure 1: Hydrophilic aerogel used

Ordinary Portland cement CEM 1 42.5 R was used to produce all the concrete mixes. A total of four different types of concrete mixes were cast and tested, namely plain concrete (KB), concrete containing %0,25 granular aerogel (AB % 0.25), concrete with %0,5 granular aerogel (AB % 0.50) concrete including %0,75 granular aerogel (AB % 0.75) by volume of concrete. All the tests were carried out at an age of 28 days. Table 2 shows the mix proportions of the concrete studied for saturated and surface dry conditions of the aggregates.

Table 2: Concrete mix proportions for $1m^3$

Concrete ID	Cement (kg/m^3)	Water (kg/m^3)	Agregate (kg/m^3)	Aerogel (kg/m^3)
KB	360	205	1830	0
AB (% 0.25)	360	203	1830	0,1
AB (% 0.50)	360	199	1830	0,2
AB (% 0.75)	360	195	1830	0,3

After the mixing procedure (Figure1), the fresh concrete was filled into steel moulds in two layers and then consolidated by using a vibration table to release possible entrained air voids and covered with plastic sheet to prevent excessive surface moisture loss. Thereafter, the specimens were left in their moulds for 24 h, and finally cured at 20 ± 2 °C in a water tank until the day of testing.

2.1 Compressive strength experiment

The compressive strength of the mixtures were measured on 150 mm cubes in accordance with the relevant Turkish-European Standards. The loading has been applied taking care not to bump the sample, with maximum %10 deviation ratio from selected rate until it reaches the biggest load at a 0,2-1 MPa (N/mm2) constant loading rate. The following equation is used to calculate the compressive strength:

$$fc=F/Ac \quad (1)$$

Fc, F and Ac symbols shown in equation represent concrete compressive strength (MPA), maximum load reached during fracture (N), cross sectional area of the sample (mm^2) on which pressure load has been applied, respectively.

2.2 Impact (Charpy) experiment

The impact fracture parameters of the mixtures were determined by testing small prism beams having a size of 40x40x160 mm. The pendulum type devices shown in Figure 2, are used in impact experiment. The pendulum with G weight and l length has been rised to a h1 height where it will have potential energy determined before and after placing the sample properly, the dial indicator has been brought to start position. The pendulum has been released properly thus breaking the sample, the hammer broken the sample by pulse impact has rised h2 height consisting b pitch angle with vertical axis of sample. The difference between the potential energy when the pendulum came into contact with sample and the potential energy left in pendulum after sample broken, gives the energy required to break the sample, in other words impact resistance. In this case the fracture energy is expressed by the following equation:

$$K.E. = Gxh1 - Gxh2 = Gx(h1-h2) = Gxlx(\cos b - \cos a) \quad (2)$$

Figure 2: Charpy testing apparatus

2.3 Water permeability experiment

The permeability of the concrete samples with or without aerogel in 150x150x150 mm sizes has been carried out by water device (Figure 3) with compressor and pressurized water as specified in the related Turkish-European Standard.

Figure 3: Water permeability testing apparatus

The 500 kPa water pressure has been applied for 72 hours to the samples placed in device. After the pressure having been applied for specific time the experiment has been ended and the sample removed from the device. The sample has been split up into two equal part in perpendicular position to the place where the pressurized water has been applied and the water pressure in greatest depth from the experiment area on which pressure applied has been measured and recorded rounding the nearest millimeter.

3. RESULTS AND DISCUSSION

3.1 Experiment results of the compressive strength

The compressive strength of the reference concrete and the concretes containing aerogel for 28 days has been given in Table 3 and displayed graphically in Figure 4.

Table 3. 28-day compressive strength test results

	28-day Compressive Strength (MPa)	
Concrete ID	Mean Compressive Strength (MPa)	Concrete Type/KB
KB	28.5 (1,55*)	1
AB (% 0.25)	35.3 (1,58*)	1.24
AB (% 0.50)	23.4 (2,37*)	0.82
AB (% 0.75)	19.2 (3,84*)	0.67

* Denotes standard deviations

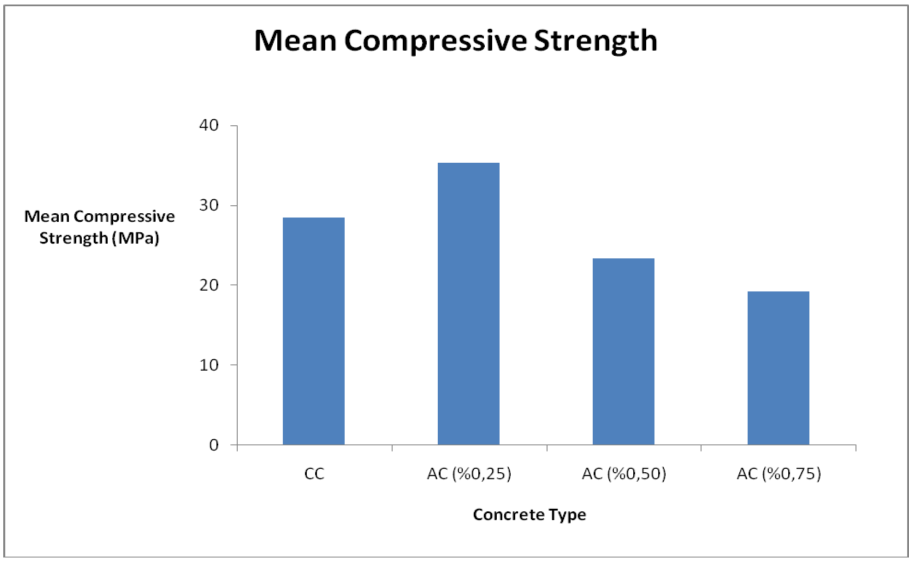


Figure 4. Graphical display of the compressive strength results

According to the experiment results, the compressive strength of the sample has increased by 1.24 ratio at the samples containing 0.25% aerogel but decreased by 0.82 ratio at the samples containing 0.50% aerogel and 0.67 ratio containing 0.75% aerogel. As almost all of the concrete has formed from air spaces, increasement of aerogel above a certain rate makes the concrete less resistant to compressive load.

It is known that the increase of the voids (reducing homogeneity) in concrete not only reduces the concrete weight but also decreases the strength of it [5]. Nevertheless, it has been seen that it is possible to increase the strength of concrete while reducing its weight by the addition of ideal amount of aerogels.

3.2 Charpy (Impact) experiment results

The results of impact resistance experiment have been given in Table 4 and illustrated graphically in Figure 5. As seen in the table and chart, beneficial effect on the concretes with aerogel comparing to control concrete decreases in respect of impact resistance if the ratio of water /cement kept constant and the control concrete samples for 28 days accepted reference.

Table 4. Fracture energy and impact strength of 28-days concrete

Concrete ID	KB	AB (% 0.25)	AB (% 0.50)	AB (% 0.75)
Fracture Energy (J)	149.1 (11,8*)	143.9 (18,54*)	119.4 (9,41*)	39.2 (7.44*)
Impact Strength (J/m²)	93187.5 (7375*)	89916.7 (11589.72*)	74604.2 (5882.09*)	24500 (3756.62*)

* Denotes standard deviations

The impact effect which is an external force applied in a shorter time than one- third of the natural vibration period of material creates an external work on concrete. The concrete also needs to balance this energy with an internal work and the concrete changes its shape clearly to create an internal work. However, it can be clearly seen that these reactions cause the concrete to break suddenly as it makes more brittle (less fracture energy dissipation under dynamic loading) due to space increasement in it.

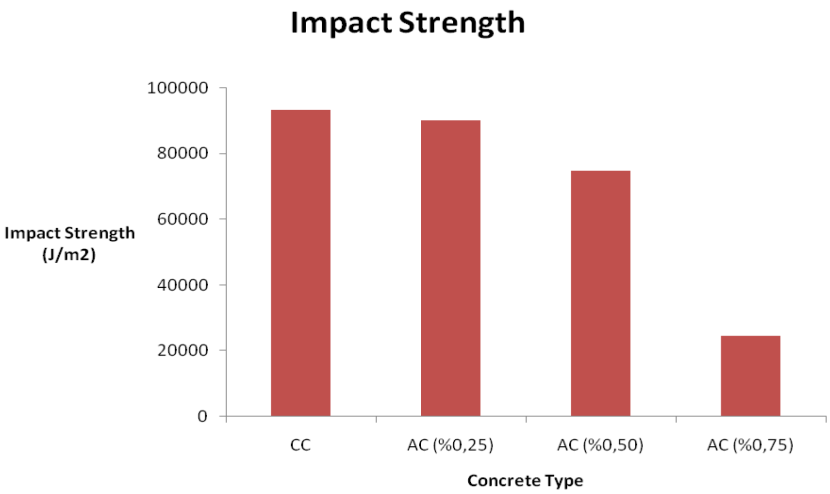


Figure 5. Graphical display of the results of the impact test

3.3 Experiment results of water resistance

The permeability at the end of 28 days cure of the concrete samples with or without aerogel (the largest water treatment depths) has been indicated in Table 5 and graphically in Figure 6 rounding the nearest value in mm.

Table 5: Water permeability values of 28-days concrete

Concrete ID	Water Permeability (mm)
KB	15,5 (0,58*)
AB (% 0.25)	13 (1*)
AB (% 0.50)	16 (1*)
AB (% 0.75)	20,5 (0,58*)

* Denotes standard deviations

The water permeability results of the concretes left in cure for 28 days have been determined by measuring the distance of water progress in the concretes broken by applying tensile stress. The concrete containing 0,25 % aerogel has yielded less water permeability and more positive results comparing the control concrete. However the water progress in sample has increased proportionally with the increasement of aerogel in concrete. Due to its hydrophilic structure aerogel has caused the concrete to be porous since it absorbed an amount of water needed for chemical reaction (hydration) of the concrete, when it has contacted with water.

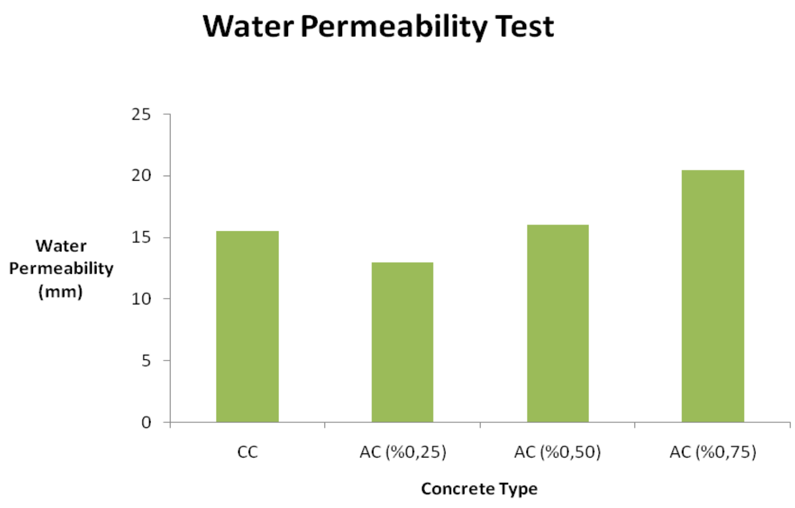


Figure 6: Graphical display of the test results of the water permeability

5. THE RESULTS

The following conclusions was made from the experimental research on the effects of the hydrophilic granular aerogel on the mechanical and durability performance of concrete.

1) The aerogel usage up to a certain percentage not only reduces the unit weight of the concrete but also provides significant improvements on the strength of concrete.

2) Aerogel usage has caused brittle fracture of concrete comparing to control concrete when being subjected to short term severe dynamic loading (stroke)

3) The satisfactory results have been obtained for the optimum value of aerogel regarding permeability as parallel to the results of pressure strength.

The existing work can guide scholars that are undertaking theoretical and experimental research on this subject and this can result in a major contribution to the construction and other sectors.

REFERENCES

[1] Fricke, J., Tillotson, U, T., ‘Aerogels: Production, Characterization, and Applications’, *Thin Solid Films* **297** (1997) 212–223.

[2] Gao, T., Jelle, B.P., Gustavsen, A., & Jacobsen, S., ‘Aerogel-incorporated concrete: An experimental study’, *Construction and Building Materials* **52** (2014) 130–136.

[3] Taşdemir, C., ‘Hafif Betonların Isı Yalıtım ve Taşıyıcılık Özelikleri’, *Türkiye Mühendislik Haberleri* **427** (2003/5) 57-61.

[4] Rao, A.V., Kulkarni, M.M., Amalnerkar, D.P., Seth, T., ‘Surface Chemical Modification of Silica Aerogels Using Various Alkyl-Alkoxy/ChloroSilanes’, *Applied Surface Science* **206** (1-4) (2003) 262-270.

[5] Neville, A., 'Concrete: Neville's insights and issues', *Thomas Telfords Publishing,* London, 2006.

COST-BENEFIT ANALYSIS OF CONCRETE MIX DESIGN ALTERNATIVES FOR IMPERMEABILITY

Göktuğ Aktaş (1), Yılmaz Akkaya (2), Can Bilal (3), Muhittin Tarhan (4) and Aslı Özbora Tarhan (5)

(1,4) Akçansa Çim. San. Tic. A.Ş., Turkey

(2,3) İstanbul Teknik Üniversitesi, Turkey

(5) Hyundai E&C

Abstract

Permeability is one of the main features for the design of concrete structures based on durability. Specifications, which consider the service life of structures, emphasize the permeability properties of hardened concrete as well as criteria for strength. These specifications may prescribe concrete constituent properties, including the type and content of admixtures and additives, and/or impose performance based limitations. Some of the suggested methods include using of waterproofing admixtures or mineral admixtures, limiting the water/cement ratio and/or controlling performance of concrete mixtures by testing permeability properties.

In this study effect of the methods, mentioned above, are investigated, and the optimum cost-benefit solution is evaluated. Water proofing powder admixtures, ground granulated blast furnace slag and various water/cement ratios are tested for different strength classes of concrete. Fresh and hardened properties of these mixes are compared, including workability and compressive strength. Permeability tests, including water absorption rate, initial surface absorption, penetration of water under pressure and chloride migration tests are performed on concrete mixtures. Performances of concretes are compared with respect to cost-benefit analysis.

1. INTRODUCTION

Necessary precautions should be taken into consideration for enhanced durability while designing a concrete structure for service life. Nowadays practice of mixing admixtures that forms crystals when contacted with water is getting popular. Other precautions that are advised are replacement of cement with mineral additives and limiting water to cement ratio. The objective of this study is to examine the impermeability of different concrete mixture design approaches and to determine a cost vs. benefit analysis.

The durability of concrete is related to the characteristics of its pore structure. Degradation mechanisms of concrete generally depend on the way potential aggressive substance can penetrate into the concrete, possibly causing damage. The concrete permeability is depending on the porosity and the connectivity of the pores. The more open the concrete pore structure, the more vulnerable the material is to degradation mechanisms caused by penetrating substances [1].

Crystalline admixtures are hydrophilic and the active ingredients react with water and cement particles in the concrete to form calcium silicate hydrates and/or pore-blocking precipitates in the existing microcracks and capillaries. The mechanism is analogous to the formation of calcium silicate hydrates and the resulting crystalline deposits become integrally bound with the hydrated cement paste [2]. These crystals generate pore blocking deposits that are found to improve the concrete’s ability to resist water penetration under pressure. These admixtures have been found to remain chemically active within the concrete and will seal additional gaps up to a certain size as they occur.

In the last few decades, industrial by-products such as blast-furnace slag have been increasingly used as mineral additives in cement and concrete. The use of these mineral additives can achieve not only economic and ecological benefits but technical benefits as well. It is generally accepted that, with proper selection of admixture, mixture proportioning and curing technique, mineral additives can greatly improve the durability of concrete [3].

The capillary pores decrease with the decreasing water/binder ratio. As result of the decreasing capillary porosity, the strength increases and the permeability decreases.

2. EXPERIMENTAL STUDY

2.1 Materials

CEM I 42,5R cement and GGBS are used in concrete production. Chemical and physical properties of binders are presented in Table 1. A commercially available waterproofing admixture is also used in the study. The FT-IR chart of the crystal admixture in Figure 1.

Table 1: Properties of Cement and GGBS

Cement		GGBS	
LOI(%)	2,89	LOI(%)	0
MgO(%)	1,25	MgO (%)	5,58
SO_3(%)	3,45	SO_3(%)	0,12
Cl^-(%)	0,0419	CaO (%)	35,57
Insoluble Residue(%)	0,5	Al_2O_3(%)	12,53
2. Day Compressive Strength(MPa)	26,9	(CaO+MgO)/ SiO_2	1
28. Day Compressive Strength(MPa)	51,2	45 micron residue (%)	50,8
Setting Time(Start./Finish.)	111/165	7. Day Activity (%)	50,8
Soundness(mm)	1	28. Day Activity(%)	77,2
Blaine (cm^2/g)	3680	Blaine (cm^2/g)	5331
Density (ton/m^3)	3,14	Density (ton/m^3)	2,92

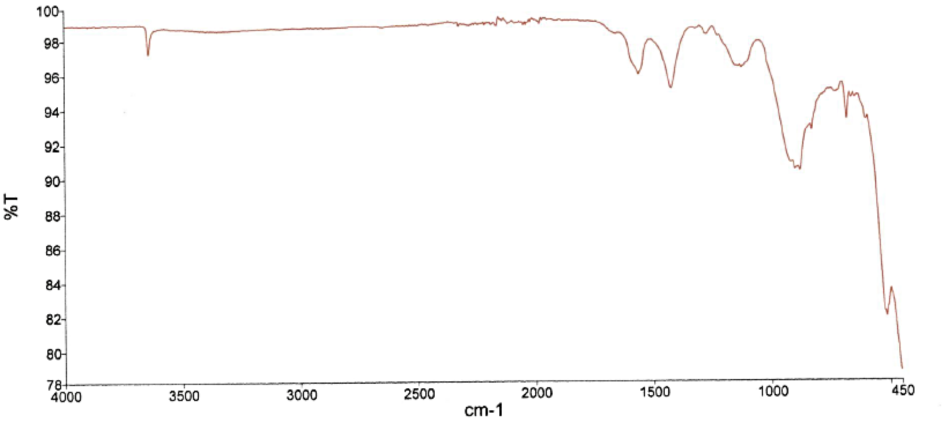


Figure 1: XRF analysis of waterproofing admixture

Natural silica sand and crushed limestone are used as fine and coarse aggregate (2 types), respectively. Sieve analysis of aggregates is presented in Table 2. Physical and chemical properties of aggregates are given in Table 3.

Table 2: Sieve analysis of aggregates

% PASSING													
Material Type	31,5 mm	22,4 mm	19 mm	16 mm	11,2 mm	8 mm	5,6 mm	4 mm	2 mm	1 mm	0,5 mm	0,25 mm	0,125 mm
Crushed Sand	100	100	100	100	100	100	100	94	46	25	18	14	10
Coarse 1	100	100	100	100	100	80	29	5	1	1	1	1	1
Coarse 2	100	100	99	82	9	1	1	1	1	0	0	0	0
Silica Sand	100	100	100	100	100	100	100	100	100	100	98	18	1

Table 3: Physical and Chemical properties of aggregates

Material Type	$CaCO_3$ (%)	Sand Eq. (%)	Meth. Blue (ml/g)	<0,063 mm (%)	SSD Weight ton/m3	Water Absorption (%)	Flakiness (%)	Los Angeles Abrasion (%)
Crushed Sand	95	59	1	8,0	2,7	0,9	-	-
Coarse 1	93	-	-	-	2,71	0,6	12	19
Coarse 2	90	-	-	-	2,71	0,6	9	19
Silica Sand	-	86	1	0,9	2,64	1,6	-	-

2.2 Concrete mixture designs and fresh concrete test results

In order to produce 3 different compressive strength classes, different binder compositions and amounts have been used. Slump, flow and unit weight measurements were made according to EN 12350-2, EN 12350-8 and EN 12390-7, respectively. S4 workability class was kept constant in all mixture designs. Self-compacting mixture designs were SF6 workability class. Water to binder ratio varied up to 0,03 points due to workability gained by addition of slag into mixture designs. All fresh concrete mixtures were non-segregating and pumpable. Mixture designs and fresh concrete test results are presented in Table 4.

Table 4: Mixture designs and fresh concrete test results

Mixture Design	W/B	CEMENT (kg)	SLAG (kg)	PLASTICIZER (%of binder)	CRYSTAL ADDITIVE (% of binder)	SLUMP/FLOW (mm)	UNIT WEIGHT (kg/m3)
NC20	0,72	250	-	0,9	-	190	2395
X-NC20	0,72	250	-	0,9	2,5	190	2384
NC30	0,59	300	-	1,5	-	190	2394
X-NC30	0,59	300	-	1,5	2,5	190	2399
NC30-SLAG	0,56	200	100	1,5	-	190	2409
X-NC30-SLAG	0,56	200	100	1,5	2,5	190	2391
SCC-40	0,48	340	-	1,6	-	600	2431
SCC40-SLAG	0,45	220	120	1,6	-	600	2428

2.3 Hardened concrete properties

Concrete samples were air cured for 24 hours and then cured in water at 20±2 °C for 27 days. 3 samples were used in all tests and results are presented in Table 5.

Capillary absorption tests were performed according to ASTM C1585, with 100mm diameter samples. Samples have been oven dried and their capillary absorption was measured for 7 days. For under-saturated in-situ environmental conditions, concrete absorption can be related to its pore structure.

Water penetration under pressure test was performed according to EN 12390-8. 150mm*150mm cubes were subjected to 500±50 KPa water pressure for 3 days. At the end of the exposure, samples were splitted into two halves and water penetration depth was measured. This test may provide information regarding the permeability of concrete when the structural member is subjected to water pressure.

Chloride migration tests were performed according to NTBuild 492 with 100mm diameter, 50mm thick cylinders. This test is aimed to determine the tendency of concrete to transport chloride ions. The diffusion coefficient, evaluated from this test, can be used in service life calculations of structures.

Initial surface absorption test (ISAT) was performed according to BS1881-5 with 100mm diameter and 50mm thick samples. Surface water absorption of samples were measured for one hour.

Compressive strength was determined according to EN 12390-3 with 150x150x150mm cubes. Unit weights were determined according to EN 12390-7.

Table 5: Permeability test results

Mixture Design	Chloride Migration Coeff. (10^{-12} m^2/s)	Capillary Water Abs. (m/s)		Average Water Penetration Depth (mm)	Maximum Water Penetration Depth (mm)	ISAT [ml/m2/s]		
		Initial	Secondary			10 MIN	30 MIN	60 MIN
NC20	24,6	48	17	19	27	0,034	0,019	0,013
X-NC20	23,9	43	13	18	20	0,044	0,023	0,015
NC30	17,5	32	10	14	17	0,041	0,022	0,017
X-NC30	12,3	32	9	8	16	0,047	0,021	0,013
X-NC30-SLAG	3,8	28	9	7	15	0,030	0,015	0,010
SCC-40	3,3	26	6	10	22	0,035	0,018	0,011
NC30-SLAG	11,9	27	9	6	11	0,027	0,015	0,009
SCC40-SLAG	2,4	18	7	3	3	0,022	0,013	0,010

3. COST BENEFIT ANALYSIS

Main constraints in construction industry are usually quality, time and economy. In this study, value of possible solutions was evaluated based on their cost and effectiveness, under given quality. Major cost impacts on mixture designs are caused by admixture, cement, slag and crystal admixtures, therefore those main items were considered and their prices were converted into price of a ton of cement. Price table of raw materials are presented in Table 6.

Table 6: Price table of raw materials

Material	**Price**	**Unit**
Cement	1	Per ton
Slag	0,9	Per ton
Normal Plasticizer	0,00375	Per kg
Super Plasticizer	0,0113	Per kg
Crystal Admixture	0,025	Per kg

In order to analyze results of 4 different permeability tests, a permeability rating is established. Selected test results are ranked based on these limits, as presented in Table 7. Rankings of the mixtures are presented in Table 8. Then, the ranking numbers are averaged to calculate a Harmonized Permeability Category (HPC).

Table 7: Permeability ranking limits of the test results

Ranking	Category	Chloride diffusion	Secondary capillary water absorption	Av. water penetration under pressure	ISAT 60 min.
1	High	20-25	12-18	11-20	0,016-0,020
2	Medium	11-20	8-11	4-10	0,012-0,015
3	Low	<10	<8	<4	<0,012

Table 8**:** Rankings of mixture designs

Mixture Design	Chloride Migration Coeff. (10 -12 m^2/s)	Secondary Capillary Water Absorption (m/s)	Average Water penetration (mm)	ISAT 60 MIN (ml/ m^2/s)
NC20	1	1	1	2
X-NC20	1	1	1	2
NC30	2	2	1	1
X-NC30	2	2	2	2
X-NC30-SLAG	3	2	2	3
SCC-40	3	3	2	3
NC30-SLAG	2	2	2	3
SCC40-SLAG	3	3	3	3

As a result, HPC, cost of mixture designs and value are tabulated in Table 9, below.

Table 9: HPC and cost of mixture designs

Mixture Design	HPC	Mixture Design Cost (tons of cement)	Value (cost/score)
NC20	1,25	0,26	30
X-NC20	1,25	0,41	43
NC30	1,50	0,32	123
X-NC30	2,00	0,50	104
X-NC30-SLAG	2,50	0,49	135
SCC-40	2,75	0,40	180
NC30-SLAG	2,50	0,31	236
SCC40-SLAG	3,00	0,39	252

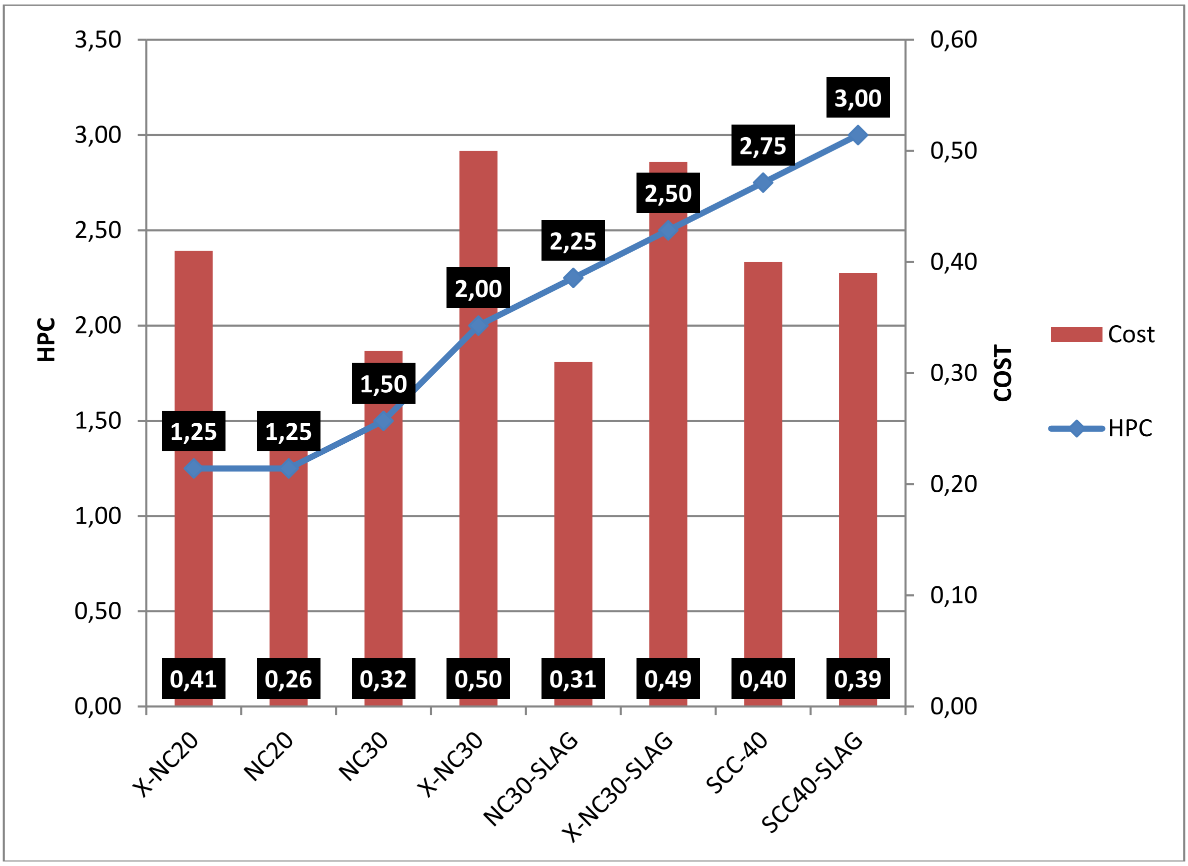


Figure 2: Cost and HPC for tested mix designs

Value for unit cost of mix designs are also investigated for finding out most effective solution. Value generated per unit cost has been calculated by cost divided by permeability. In addition to that service lives of a structure exposed to atmospheric chlorides and 50 mm concrete cover with tested mix designs have been calculated with LIFE365 software. LIFE 365 is a software designed to estimate the service life and life-cycle costs of alternative concrete mixture design proportions and life-cycle costs of alternative concrete mixture designs proportions and corrosion protection systems. All those tabulated in Figure 3 below could be used for determination of best value for given price.

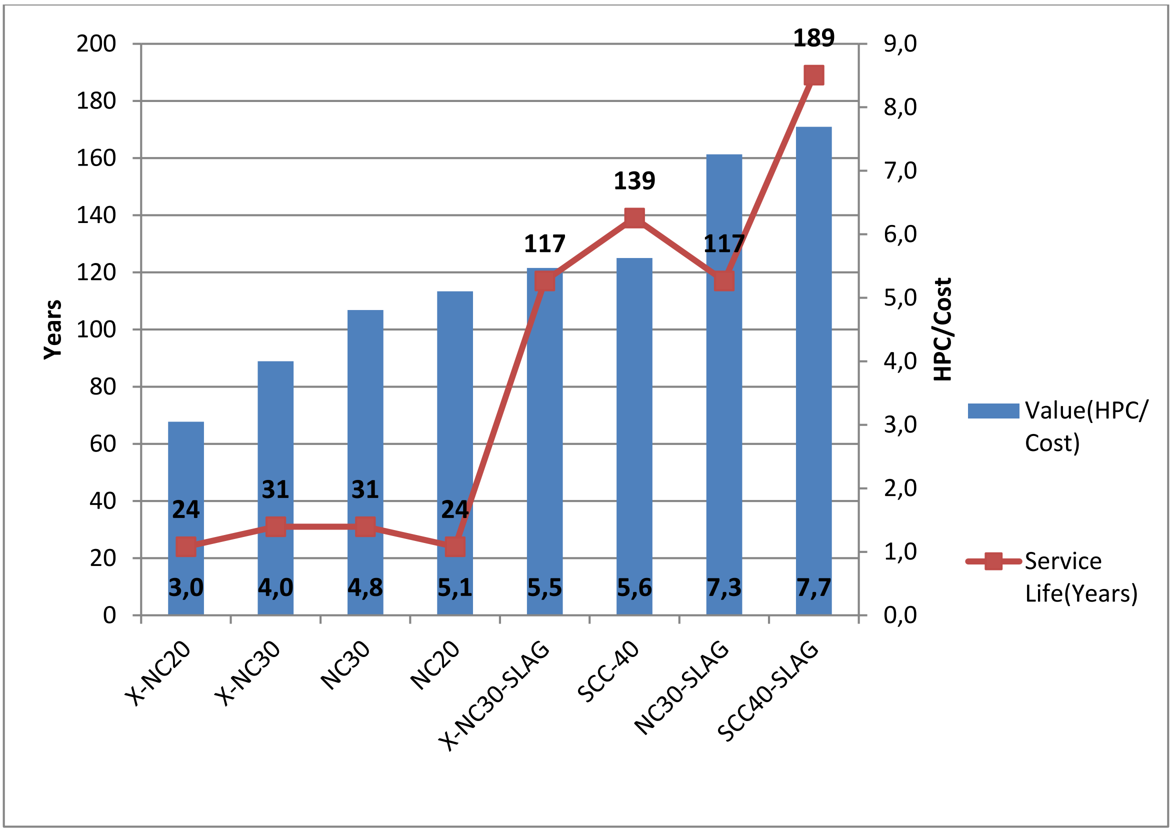


Figure 3: Service life and value for tested mix designs

4. CONCLUSION

- The correlation between different permeability tests varies considerably. Relevant environmental exposure conditions should be evaluated and criteria should be established based on durability requirements.
- Addition of crystal admixture is more effective when w/c ratio is low.
- Addition of slag found to be very effective for decreasing Cl permeability, for low w/c ratio.
- Chloride migration coefficient, capillary absorption and average penetration of water under pressure decreases with a strong trend when w/c ratio is decreased. However, the trends are not very strong for max. penetration of water under pressure and ISAT tests. This indicates that the porosity system in the concrete is far more complex to quantify by standard test methods. Pore system should be investigated by quantifying parameters such as connectivity, tortuosity, bottle-necks in the pore system. However, after extensive pretesting of permeability and the pore system in concrete, it may be possible to use standard testing for quality control of daily production.

- Based on cost-benefit analysis of mix designs lowering w/c ratio and GGBS addition to mix design is found to be a better overall solution than crystal admixture addition.

REFERENCES

[1] De Schutter, G., Audenaery, K., “Evaluation of Water Absorption of Concrete as a Measure for Resistance against Carbonation and Chloride Migration”, Materials and Structures, No. 37, pp.591-596, 2004.

[2] Report on Chemical Admixtures for Concrete, ACI 212.3R-10 (Reported by ACI Committee 212)

[3] Wu, Z., Naik, T.R., “Properties of Concrete Produced From Multicomponent Blended Cements”, *Cement and Concrete Research*, No. 32, pp. 1937–1942, 2002.

COMBINED EFFECTS OF TEMPERATURE AND WAITING TIME ON THE RHEOLOGY OF ECC

Müzeyyen Balçıkanlı (1), Erdoğan Özbay (2), Mete Arslan (3) and Mohamed Lachemi (4)

(1,2) Civil Eng Department, Mustafa Kemal University, Iskenderun, Hatay, Turkey

(3) Adana Cement Industry Inc., Adana, Turkey

(4) Civil Eng Department, Ryerson University, Toronto, Ontario, Canada

Abstract

Engineered Cementitious Composites (ECC) is a newly developed fiber-reinforced cementitious material with substantial benefits in terms of high ductility and improved durability. However, so far rheological properties of ECC have not received significant attention in literature. This study evaluates the combined effects of extreme weather conditions and waiting time (WT) on rheological properties of ECC. In order to find out the combined effects of five temperatures (5 °C to 50 °C) and four WTs (0 min to 20 min) on the rheological properties, twenty trials were performed on ECC mixtures with a Fly ash/Portland cement ratio of 1.2. Flexural and compressive strength test results are also presented. Test results indicated that among the investigated temperatures, 5 °C and 50 °C have significantly different effect on the yield stress and plastic viscosity of ECC. However, an increase in WT up to 20 minutes did not significantly alter the plastic viscosity and yield stress of the fresh material, nor did it affect the flexural and compressive strengths of the hardened material.

Keywords: Engineered cementitious composites; rheology; temperature; waiting time

1. INTRODUCTION

Engineered Cementitious Composites (ECCs) are a unique class of the new generation high-performance fiber-reinforced cementitious composites featuring high ductility and medium fiber content. Tensile strain capacities of 3% to 5% have been reported in ECC materials using polyethylene and polyvinyl alcohol (PVA) fibers with fiber contents below 2% by volume [1-3]. Unlike ordinary concrete materials, ECC strain hardens after first cracking like a ductile metal, and exhibits approximately 300 to 500 times more tensile strain capacity than normal and fiber-reinforced concrete. Even at large imposed deformations of several percent, crack widths of ECC remain small, at less than 100 μm [4]. So far, very few studies have been performed on the rheological properties of ECC. In this paper, the effects of temperature and WT after mixing on plastic viscosity, yield stress, compressive and flexural strength development of an ECC mix were investigated.

2. EXPERIMENTAL STUDIES

A standard ECC mixture was prepared with water to cementitious material (W/CM) ratio of 0.27 and a fly ash-Portland cement ratio (FA/PC) of 1.2 by mass [4]. A standard mortar mixer was used throughout this investigation. A temperature- and relative-humidity-controlled walk-in environmental chamber was used for the preparation and testing of the ECC mixtures. A rheometer was used throughout this investigation to measure the rheological properties of fresh ECC. The rheometer is fully automated and the entire experimental process was controlled by computer software [5,6]. The chamber was set to the required temperature level 24 hours before the experiment. Twenty trials were performed in total, at the five temperature levels (5 °C, 23 °C, 30 °C, 40 °C and 50 °C) and four WTs (0, 5, 10, and 20 minutes). From each trial, three 50-mm cubic specimens were prepared for the compressive strength test, and three 355 mm × 50mm × 76 mm prism specimens were prepared for the four-point bending test at 28 days.

3. RESULTS AND DISCUSSIONS

3.1 Plastic viscosity and yield stress

Figure 1 represents the yield stress and plastic viscosity behavior of the ECC depending on WT and temperature. As seen in Figure 1(a), an increase in WT generally increased the yield stress of the ECC. However, increases in the yield stresses within the range of 23 °C, 30 °C, and 40 °C were very low. However, at extreme temperatures (5 °C and 50 °C), the increases were very high. At zero WT after mixing (0WT), the lowest yield stress was observed at 40 °C. The yield stress values at 23 °C, 30 °C and 40 °C were very close to one another at all WTs. At 50 °C and up to 10 minutes WT, yield stress values were around 180 Pa, but this value jumped to 350 Pa at 20 minutes WT, resulting in the highest yield stress. Figure 1(b) depicts the variation of plastic viscosity with WT and temperature. The plastic viscosity values ranged from 0.55 Pa.s to 14.83 Pa.s. Increasing the temperature from 5 °C to 30 °C decreased the plastic viscosity gradually for all WTs, at temperatures beyond 30 °C, plastic viscosity followed an increasing trend between the 30 °C and 50 °C. The lowest plastic viscosity value was observed at zero minutes WT at 30 °C and the highest at 20 minutes WT at 50 °C. As seen in Figure 1(b), an increase in WT generally increased the plastic viscosity values of the ECC.

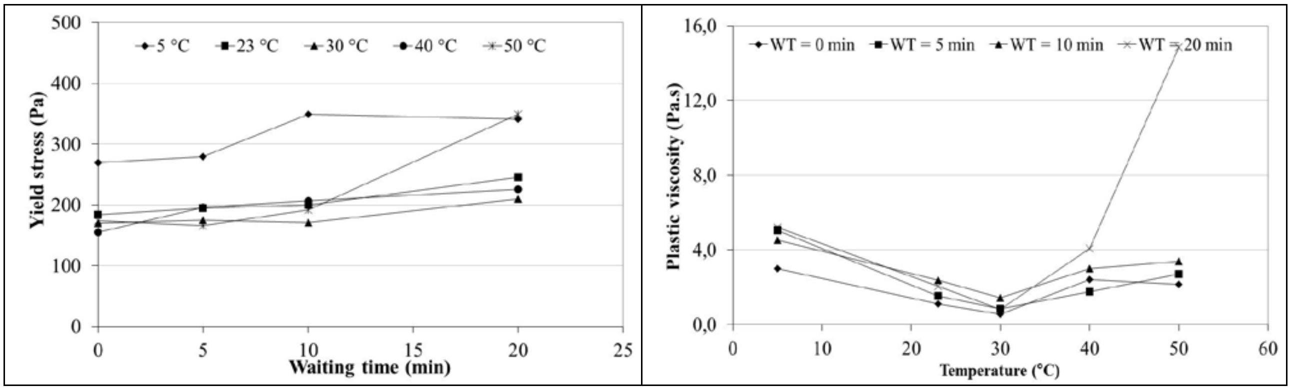


Figure 1: Variation of rheological parameters (a) Yield stress, (b) Plastic viscosity

3.2 Compressive and flexural strengths

The compressive strength of ECC varied from 55.0 MPa to 69.8 MPa. The flexural stress-mid-span deflection curves for different WTs and temperatures are shown in Figure 2. A slight reduction in the compressive strength of ECC at the two extreme temperatures (5 °C and 50 °C) was observed, while increasing WT from 0 to 20 minutes after mixing did not significantly alter compressive strength values. In the bending test, the ultimate flexural strengths varied from 9.42 MPa to 13.04 MPa.

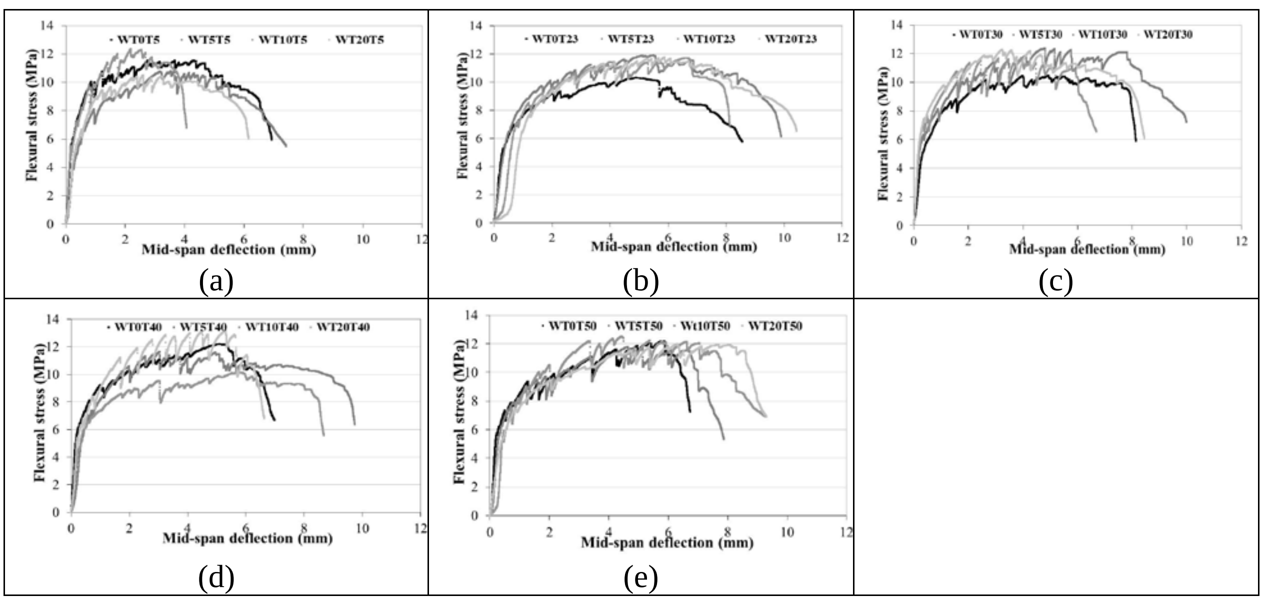


Figure 2: Flexural stress mid span deflection graphs (a) 5°C (b) 23°C (c) 30°C (d) 40°C (e) 50 °C

The total deflection, which reflects the materials' ductility, varied from 2.84 mm to 5.07 mm at the age of 28 days. The lowest ultimate deflection values were observed at 5 °C, and increasing WT gradually resulted in decreased ultimate deflection values at that temperature. However, for the rest of the temperatures studied ultimate deflections did not show any meaningful ascending or descending trend with WT.

4. CONCLUSIONS

Based on this study, the following conclusions can be drawn:

- Shear thinning behavior was observed in the fresh ECC mix at the temperatures and WTs studied, and an increase in temperature resulted in a slight decrease in the degree of shear thinning behavior.
- Variation of viscosity with temperature was more accentuated for the increase from 5 °C compared to those at 23 °C than those for from 23 °C to 30 °C and 40 °C. A slight increase was observed in yield stress with the increase in WT, but for 20 minutes WT at 50 °C, yield stress increased drastically.
- Apparently concave plastic viscosity versus temperature curves with minima around 30 °C were observed at all WTs studied. Increasing the temperature from 5 °C to 30 °C decreased the plastic viscosity.
- It was observed that ECC specimens at 5 °C and 50 °C had slightly lower compressive strength than those at 23 °C. However, flexural strength values did not show any evident trend with temperature and WT.

REFERENCES

[1] Wang, S., and Li, V.C., 'Engineered Cementitious Composites with High-Volume Fly Ash', *ACI Materials Journal*, **104** (3) (2007) 233-241.

[2] Li, V. C., 'Engineered Cementitious Composites—Tailored Composites Through Micromechanical Modeling,' *Fiber Reinforced Concrete: Present and the Future*, N. Banthia, A. A. Bentur, and A. Mufti, eds., Canadian Society for Civil Engineering, Montreal, Quebec, Canada, (1998) 64-97.

[3] Li, V. C., Wu, C., Wang, S. Ogawa, A. and Saito, T., 'Interface Tailoring for Strain-Hardening Polyvinyl Alcohol-Engineered Cementitious Composite (PVA-ECC),' *ACI Materials Journal*, **99** (5) (2002) 463-472.

[4] Yang, E.H., Sahmaran, M., Yang, Y., and Li, V.C., 'Rheological Control in Production of Engineered Cementitious Composites', *ACI Materials Journal*, **106** (4) (2009) 357-366.

[5] Nehdi, M., and Al-Martini, S., 'Coupled Effects of High Temperature, Prolonged Mixing Time, and Chemical Admixtures on Rheology of Fresh Concrete', *ACI Materials Journal*, **106** (3) (2009) 231-240.

[6] ConTec Operating Manual, The BML Viscometer, Viscometer **4** (2000) 20-70.

ENHANCING COMMUNITY RESILIENCY WITH READY MIXED CONCRETE

Tien Peng

National Ready Mixed Concrete Association, USA

Abstract

Over the past few decades, there was an exponential increase in human and material losses from disaster events. Natural disasters are physically, socially, and psychologically devastating to a community. It can be extremely difficult to rebuild and restore the lives of residents after the destructive event. Where do organizations and governments begin to help its constituents? Resilient infrastructure policies move the community from reactive approaches to a proactive stance where stakeholders actively engage in reducing many of the broad societal and economic burdens that disasters can cause. Investing in resiliency, from strengthening building codes to adopting voluntary standards can be surprisingly cost-effective, greatly reducing the impact of natural hazards. Policies affecting building practices can also be instrumental in increasing economic investment in making the socio-economic dimension of our society resilient and climate proof. Building with robust materials such as concrete is complementary outcome of resilience strategies.

1. INTRODUCTION

The past two decades saw record-setting loss of life and property in virtually every part of the world, including Europe. Between 1998 and 2009, natural hazards and technological accidents caused nearly 100,000 fatalities and affected more than 11 million people. The heat wave of 2003 had the highest human losses over western and southern Europe, with more than 70,000 fatalities. The Izmit, Turkey earthquake of 1999, took more than 17,000 lives. The largest disasters due to natural hazards caused a loss of about €150 billion in the 32 European Economic Area (EEA) member countries (EEA, 2010).

While there is credible concern from climate-induced sea-level change with global warming projected to intensify the hydrological cycle and increase the occurrence and frequency of flood events in large parts of Europe (IPCC, 2013), estimates of changes in flood frequency and magnitude remain highly uncertain. Moreover, there has been no clear long-term trend in major storms in Europe (Matulla, 2008) or wildfire events (EPI, 2015). Therefore, most of the increased disaster losses cannot necessarily be attributed to an increased occurrence of hazards. The increase in losses can be explained to a large extent by population migration and accumulation of economic assets in hazard-prone areas.

Over the past 50 years, the population living in European coastal municipalities has almost tripled to reach 196 million inhabitants and the total value of economic assets located within 500 meters from the coastline has multiplied to an estimated €500-1000 billion in 2000 (Doody, 2004). That’s 43% of the inhabitants of the 22 EU Member States lived in with 50 km of the sea. This high concentration of people in historically hazardous regions – stormy northwest and seismic prone southeast coasts- has produced many economic benefits, but the combined effects of booming population growth and commercial development have strained the infrastructure that provided these economic benefits.

At the same time, wealth and the value of their possessions have increased substantially. Almost 40 % of the EU's GDP is generated in these maritime regions, and a staggering 75 % of the volume of the EU's foreign trade is conducted by sea. These changes in concentration of population and economic values are significant contributors to the increased property losses from natural hazards.

2. COMMUNITY DISASTER RESILIENCE

Disasters result not as much from the destructive agent itself but from the way in which communities are (or are not) prepared. Disasters happen when the natural systems are encroached upon by human development. The extent of disruption caused by a disaster is greatly influenced by the degree to which society chooses be fortified for the event. It is apparent that there needs to be significant shift in how we address natural disasters, moving away from the traditional focus on response and recovery toward emphasis on resiliency, that is, preventive actions to reduce the effects of a natural hazard.

Resilience can be understood as the capacity to anticipate and minimize potential destructive forces through adaptation or resistance. Basically addressing changes in the environment requires actions to mitigate their negative effects. If we identify resiliency, not solely as a state of preparedness for disaster, but as a desired characteristic of a sustainable society, one that maintains economic, environmental and social capital, we can begin to plan for resilience. Resilience can be expressed in terms of functionality and the time to recover following a disruptive event (Figure 1).

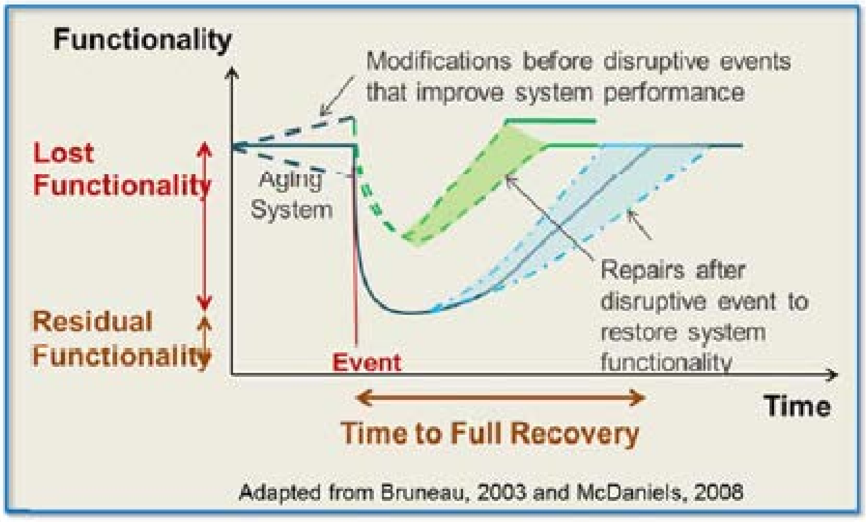


Figure 1: Functional Resilience (NIST, 2014)

Our built environment has enabled societies to thrive and grow and become increasingly interconnected and interdependent from the local to global levels. And because of society's concern for the environment, we have placed a great deal of emphasis on recycling rates and carbon footprints. It is ironic that we are surprisingly willing to invest considerable amounts of upfront capital for a BREEAM (Building Research Establishment Environmental Assessment Methodology) or LEED (Leadership in Energy and Environmental Design) certification to achieve a mere 20-30% energy efficiency, yet be completely satisfied if the structure meets only the code minimum requirements for seismic or wind load.

Balancing long term development plans with the ability to adapt to the needs of a rapidly evolving society is vital to the ultimate success of a building life plan. Planners should consider the building's potential for future use and re-use. In addition, a sustainable building should be designed to sustain suffer minimal damage due to natural disasters such as hurricanes, tornadoes, earthquakes, flooding and fire. Otherwise, the environmental, economic and societal burden of our built environment could be overwhelming. A building that requires frequent repair and maintenance or complete replacement after disasters would result in unnecessary cost, from both private and public sources, and environmental burdens including the energy, waste and emissions due to disposal, repair and replacement.

3. ECONOMIC BENEFITS OF HAZARD MITIGATION

Building a strong, resilient community is a sound strategy to prepare for climate, geophysical and extreme weather shocks. But it also holds the promise of meaningful employment and strong local economies. In the aftermath of a disaster businesses can suffer costly damage, be cut off from supplies, lose sales, and experience disrupted operations. In some cases, they may even be forced to close permanently. When businesses and industries fail or falter, the communities they serve can feel serious impacts, ranging from a lack of access to goods and services to the loss of income and jobs.

Natural hazard mitigation is a resilience strategy that saves lives and money. In 2005, the US Multihazard Mitigation Council (NIBS, 2005) of the National Institute of Building Sciences conducted an independent study funded by the Federal Emergency Management

Agency (FEMA) to study assess the effectiveness of disaster mitigation. The report quantified the future savings every dollar spent on mitigation saved four dollars in avoided future losses. The mitigation grant programs funded by FEMA cost the federal government €3.2 billion ($3.5 billion) from 1993 to 2003 but yielded a societal benefit of $14 billion. The benefits of mitigation were defined as the potential losses to society that were avoided as a result of investment in mitigation. Those benefits include:

- Reduction in property damage
- Reduction in business disruption
- Reduction in non-market damage (environmental damage to wetlands, parks, wildlife and historic structures)
- Reduction in deaths, injuries and homelessness
- Reduction in cost of emergency response (ambulance and fire service)

4. BUILD WITH ROBUST MATERIALS

There are many pathways to a resilient community. Adoption of the Eurocodes is a great start. Another key step towards resilience is to build with robust building materials. Some of the qualities of robust building materials include versatility, strength, wind and water resistance, seismic resistance, fire resistance, energy efficiency and durability. Concrete building systems are especially suited to provide resistance to natural hazards. Concrete has the necessary hardness and mass to resist the hail of storm events and flying debris of tornadoes. Concrete is fire resistant and non-flammable, which means it can contain fires and will not contribute to the spreading of fire. Reinforced concrete framing systems can be designed to resist the most severe earthquakes without collapse. Concrete doesn’t rot or rust even if it is subject to flooding.

Case Study 1 (High Wind and Storm Surge): There are many examples of structures built with heavy building materials, such as concrete, surviving major disasters. When Hurricane Katrina, the most economically devastating disaster in US history, slammed into the coastal counties of Mississippi with sustained winds of 125 mph and a storm surge that reached 28 feet, the only house to survive along the beachfront of Pass Christian, MS was the Sundberg home Scott and Caroline Sundberg were 85% complete building their dream home along the Mississippi coast when the Hurricane hit. All other homes on the beachfront were completely destroyed. They built their home using insulating concrete forms (ICFs) for the walls and cast-in-place concrete frame construction for the lower level, floors and roof precisely for this reason to survive the devastating effects of a hurricane.

Case Study 2 (Wildfires): As it stands, wildfires ravage about 2,000 square miles of land in Europe each year with warning of a 200 percent increase by 2090 (Khabarov, 2014). A 1993 wildfire in Laguna Beach, CA, consumed 17,000 acres and destroyed 366 homes in a single day. The home of To Bui and Doris Bender Los Angeles Times named the “miracle house” (Underwood 1995) shows the lone survivor which remained protected by an envelope of non-combustible, cementitious stucco wall cladding and concrete roof tiles. Detailing such as stucco cladding on walls, eaves and trim, as well as Class A concrete tile roof, prevented combustion of the exterior amidst the firestorm that swept through the community.

Case Study 3 (Tornado): Tornadoes are not a uniquely US phenomenon. Approximately 300 touch down in Europe a year (Prociv, 2013). The EF-4 tornado that roared through Tuscaloosa, AL, on April 27, 2011, leveled block after block in the Forest Lake neighborhood. The only thing left standing was a closet at the Blakeney residence on 16th Street East. The closet was built as a safe room using 8-inch reinforced concrete masonry to withstand high winds and flying debris caused by tornadoes (Jones 2011). Small windowless rooms such as a walk in closet are ideal locations for a safe room in a home. Concrete proves to be the most widely used method for constructing safe rooms**.**

Case Study 4 (Storm Surge/Flooding): Hurricane Sandy was a wakeup call for New York City. Sea level rise, increased coastal flooding, more frequent and intense rainstorms, and increased annual precipitation all threaten the functioning of her critical infrastructure. NYC is staking steps in developing strategies to prepare for these impacts with "green infrastructure" solutions like green roofs, plantings, rain barrels and porous pavement to naturally control and treat stormwater pollution by absorbing polluted runoff instead of diverting the flow straight into their waterways (NYC 2010). And they have the added benefit of making the city a more pleasant place to live overall by increasing greenspace, cooling overall temperatures, and boosting local jobs.

5. CONCLUSION

Resilience planning offers communities an opportunity to play a major role in determining the essential services and infrastructure needs that underpin their economic vitality, improve the health and safety of its citizens, and support sustainability.

Disaster mitigation by building with robust material works and is cost effective. Spending time and money up front to reduce the likelihood of loss during a natural disaster can bring significant benefits to building owners and communities including lower insurance costs, higher property values, security to building owners, maintaining a consistent tax base, and minimizing the cost of disaster response and recovery.

In the end, no community can ever be completely safe from all hazards. Generally, it would be uneconomical to design buildings to survive a direct blow from a tornado with 200 mph wind speeds or magnitude 9.0 earthquake. But resilience promotes greater emphasis on what communities can do for themselves before a disaster hits, and how to strengthen their local capacities, rather than be dependent on our ineffectual governmental agencies and aging centralized infrastructure. Disasters are inevitable, but their adverse consequences need not be.

REFERENCES

[1] European Environment Agency (EEA) Technical Report 2010, "Mapping the impacts of natural hazards and technological accidents in Europe: An overview of the last decade", No 13/2010

[2] Intergovernmental Panel on Climate Change (IPCC), 2013, Climate Change 2013 The Physical Science Basis : Working Group I Contribution to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change http://www.ipcc.ch/report/ar5/wg1/

[3] Matulla, C, et. al., "European Storminess: Late nineteenth Century to Present", Climate Dynamics August 2008, Volume 31, Issue 2-3

[4] Earth Policy Institute (EPI), http://www.earth-policy.org/images/uploads/graphs_tables/fire.htm, Accessed March 2015

[5] Doody, P, Ferreira, M., et. al., "Living with coastal erosion in Europe", Luxembourg: Office for Official Publications of the European communities, European Communities, 2004

[6] National Institute of Standards and Technology (NIST), Disaster Resilient Framework, http://www.nist.gov/el/building_materials/resilience/4th-disaster-resilience-workshop.cfm, Accessed March 2015
[7] National Institute of Building Sciences (NIBS). 2005. Natural Hazard Mitigation Saves: An Independent Study to Assess the Future Savings from Mitigation Activities. Washington, DC.
[8] Khabarov, N., Krasovskii, A., et. al., “Forest Fires and Adaptation Options in Europe”, Springer Berlin Heidelberg, 2014 http://link.springer.com/article/10.1007/s10113-014-0621-0/fulltext.html
[9] Underwood, John. (1995) “Fire Resistant Details”. Fine Homebuilding (FH). pp. 90-93 June 1, 1995. http://www.finehomebuilding.com/how-to/articles/fire-resistant-details.aspx
[10] Prociv , K, “From Domestic to International: Tornadoes Around the World” 2013, Accessed 2015 http://www.ustornadoes.com/2013/07/25/from-domestic-to-international-tornadoes-around-the-world/
[11] Jones, Adam. 2011. “Safe rooms, shelters to be in demand after deadly tornado”. Tuscaloosa News. http://www.tuscaloosanews.com/article/20110516/NEWS/110519770?tc=ar. Accessed November 13, 2012.
[12] New York City (NYC) Stormwater Design Manual, Chapter 5, August 2010 http://www.dec.ny.gov/docs/water_pdf/swdm2010chptr5.pdf, Accessed March 2015

MODEL EUROPEAN ENVIRONMENTAL PRODUCT DECLARATIONS FOR CONCRETE ADMIXTURES

Trevor Grounds

General Secretary- European Federation of Concrete Admixtures Associations (EFCA)

Abstract

Concrete should rightly be regarded as an engineered construction material that has a major role to play in providing the sustainable built environment demanded for future generations.

Admixtures are an essential constituent in helping modern concrete achieve and optimise its unique combination of properties.

Environmental Product Declarations (EPD) are a key element in the evaluation of the sustainability of construction and in future will be important in providing reliable data for building information modelling (BIM).

EFCA- the European Federation of Concrete Admixtures Associations, representing 11 national admixture associations - first published Generic EPDs for six categories of admixtures in 2006.

- water reducing/plasticising and high range water reducing/superplasticising
- Hardening accelerating
- Set accelerating
- Set retarding
- Air entraining
- Water resisting

In 2012, EN 15804 "Sustainability of construction works — Environmental product declarations — Core rules for the product category of construction products" was published. Since then EFCA and its members have worked to provide Model European EPD for the same categories of admixtures to ensure that the information is updated to be compliant with the rules in EN 15804. This paper provides the background and details of these newly updated admixture EPD.

Keywords: Concrete admixtures, environmental product declarations, concrete sustainability, EFCA(European Federation of Concrete Admixtures Associations.)

1. INTRODUCTION

Concrete should rightly be regarded as an engineered construction material that has a major role to play in providing the sustainable built environment demanded for future generations.

Its unique combination of strength and durability, thermal mass and flexibility of design enable it to make a valuable contribution at each stage of the life cycle of a project. Locally and responsibly sourced, it is fully reusable or recyclable and uses recycled and by-product materials - an ideal material for the resource efficient, circular economy that is the aim of Europe today. Admixtures are an essential constituent in helping modern concrete achieve and optimise this combination of properties by:

- Improving strength and resistance to damage from harsh environments
- Ensuring consistency during delivery and placing over a wide range of workability even for very high flow concrete
- Improving quality of finish and reducing service life repair
- Reducing embodied energy and carbon
- Improving site practice
- Delivering cost benefits to the concrete producer and user

Environmental Product Declarations (EPD) are a key element in the evaluation of the sustainability of construction and in future will be important in providing reliable data for building information modelling (BIM).

EFCA has been in existence since 1984 and represents 11 European national associations of concrete admixture producers. Among its aims and objectives are:

- To act as the voice of the European concrete admixtures industry when approaching authorities, institutions, organisations or any other competent body on an international level.
- To provide the common message of the industry to make known its position and views to the European Commission, European Parliament, CEN and other Groups dealing with issues such as European Legislation, European Harmonisation of Standards and certification.
- To present the views and interests of manufacturers, contractors and consultants in Technical, Health, Safety and Environmental matters.
- To work for the improvement of the technical standards of the industry, for example through technical knowledge, innovation and environmental protection.
- To advance and encourage the use of admixtures by means of lectures, publications and other activities including presentations at conferences.
- To work closely with other European organizations in the field of concrete, constituents and supply, in order to promote the use of concrete as the premier construction material of choice.

In keeping with these aims in 2006, working with the National Association member companies across Europe, EFCA took a lead by publishing generic EPD for six admixture types. Since then CEN Technical Committee TC 350 has been working on a suite of standards aimed at harmonising the way that sustainability of construction projects is measured. EN 15804 “Sustainability of construction works — Environmental product declarations — Core rules for the product category of construction products” [1] was published in 2012. This effectively rendered the original EFCA EPDs out of date through not being in full compliance with the EN 15804 rules.

During 2014, EFCA and its members have worked to rectify this and in 2015 will publish six new Model EPDs in compliance with EN 15804 and externally verified by the respected German Institute for the Building and Environment (IBU), a programme holder within the ECOPlatform (wwwbau-umwelt.com).

2. THE EFCA 2015 MODEL EPDS

Model EPDs have been published for six categories of admixtures that are covered by EN 934 [2]:

- Water reducing/plasticising and high range water reducing/superplasticising
- Hardening accelerating
- Set accelerating (for concrete and shotcrete)
- Set retarding
- Air entraining
- Water resisting

During 2014, materials and production data from across the European plants which are part of the EFCA federation were reviewed and life cycle analysis, which is the basis of an EPD, carried out by environmental consultants Thinkstep (www.thinkstep.com). Using the IBU product category rules in compliance with EN 15804 these were converted into a structured EPDs that meet the requirements for declaration and verification stipulated in EN ISO 14025 " Environmental labels and declarations - Type III environmental declarations - Principles and procedures." [3]

The term "Model" EPD is used to show that the results of the Life Cycle Assessment provided in each declaration have been selected from the product within each category that showed the highest environmental impact (i.e. a worst-case scenario). This method was chosen so that anyone wanting to use the EPD data for an analysis of concrete and mortar products could use the model data in the knowledge that they are adopting a conservative, safe value that represents this admixture type across the EFCA members.

In order to determine whether the Model EPD is applicable to a particular supplier's admixtures, a confidential guidance document has been produced which allows the supplier to assess, through a point scoring system, whether its admixture formulation is covered. Having done this the supplier can then declare to customers, that it meets the requirements for the EFCA EPD to be applicable to its products.

It is important to note that, because the life cycle analysis data that underlies the EPD was provided only by EFCA member associations and their companies, the Model EPD cannot be applicable to any company not within the EFCA federation structure.

2.1 Description of the EPD

The basis of the EPD is life cycle analysis, in which all of the inputs to and outputs from the product are assessed for their environmental impact with regard to a number of agreed parameters. EN 15804 recognises several stages in the life of the construction product such as concrete which are:

- The product stage
- Construction process stage
- Use Stage
- End of life stage

These are further divided into modules which are shown in Table 1.

Additionally Module D (after the product life cycle) is designed to consider any other benefits or impacts that can occur outside of the normal life cycle modules, for example for concrete this could (and should) include the benefits of carbonation of concrete in reducing atmospheric carbon dioxide, particularly after crushing and storage for recycling after demolition.

EN 15804 allows EPD to be produced which include as a minimum modules A1-A3 (product stage only) with options to include the remaining modules. For admixtures, which are constituent materials of the final concrete products, the most appropriate are Modules A1-A3 since it is difficult to assess the admixture itself directly at the later stages as it has been fully incorporated into the concrete. This is commonly described as a 'Cradle to factory Gate' EPD.

Table 1: Summary of the life cycle stages defined in EN 15804

Life Cycle Stage	Module Description	Module Ref.
Product stage	Raw material supply	A1
	Transport	A2
	Manufacturing	A3
Construction process Stage	Transport from the gate to the site	A4
	Assembly	A5
Use stage	Use	B1
	Maintenance	B2
	Repair	B3
	Replacement	B4
	Refurbishment	B5
	Operational energy use	B6
	Operational water use	B7
End of life stage	De-construction demolition	C1
	Transport	C2
	Waste processing	C3
	Disposal	C4
Benefits and loads beyond the system boundaries	Reuse, Recovery and Recycling-potential	D

For each of these EN 15804 modules a series of parameters has been established which allows calculation of the environmental impacts of using the product. Table 2 shows the range of parameters and their units. The values for each of the parameters shown in the EPD relate to 1kg of admixture and can be used as inputs to the calculation of LCA and EPD for concrete and mortar products based on the actual amounts used in the mix design.

To produce the LCA and EPD the consultant, Thinkstep has analysed data relating to raw materials and production supplied by the manufacturers using Product Category Rules (PCR)[4] established to be in compliance with EN 15804 by IBU. This analysis and a confidential background report has been submitted to IBU who will independently verify that the EPD are in compliance with the PCR and structured in accordance with EN ISO 14025. This important verification process provides the user of the EPD with confidence that the declaration is a credible representation of the environmental implication of using the admixtures as described in the relevant European Standards.

The values shown in Table 2 are preliminary, pre-verification values for 1 kg of plasticiser and superplasticiser. These are included as an indication only and should not be assumed to be the final values. The full verified Model EPD are anticipated to be obtainable for download from July 2015 either from the EFCA website at www.efca.info or the IBU website www.bau-umwelt.com.

Table 2: Preliminary results pre-verification for 1kg of plasticisers and superplasticisers

Parameter	**Unit**	**Values**
Results of LCA – Environmental Impact: 1 kg plasticisers and superplasticisers		
Global warming potential	[kg CO_2-Eq.]	1.88E+0
Depletion potential of the stratospheric ozone layer	[kg CFC11-Eq.]	2.30E-10
Acidification potential of land and water	[kg SO_2-Eq.]	2.92E-3
Eutrophication potential	[kg $(PO_4)^{3-}$- Eq.]	1.03E-3
Formation potential of tropospheric ozone photochemical oxidants	[kg Ethen Eq.]	3.12E-4
Abiotic depletion potential for non fossil resources	[kg Sb Eq.]	1.10E-6
Abiotic depletion potential for fossil resources	[MJ]	2.91E+1
Results of LCA – Resource Use: 1 kg plasticisers and superplasticisers		
Renewable primary energy as energy carrier	[MJ]	1.51E+0
Renewable primary energy resources as material utilization	[MJ]	0.00
Total use of renewable primary energy resources	[MJ]	1.51E+0
Non renewable primary energy as energy carrier	[MJ]	2.66E+1
Non renewable primary energy as material utilization	[MJ]	4.82E+0
Total use of non renewable primary energy resources	[MJ]	3.14E+1
Use of secondary material	[kg]	0.00
Use of renewable secondary fuels	[MJ]	0.00
Use of non renewable secondary fuels	[MJ]	0.00
Use of net fresh water	[m³]	6.04E-3
Results of LCA – Output flows and waste categories: 1 kg plasticisers and superplasticisers		
Hazardous waste disposed	[kg]	5.17E-6
Non hazardous waste disposed	[kg]	2.56E-2
Radioactive waste disposed	[kg]	9.00E-4
Components for re-use	[kg]	0.00
Materials for recycling	[kg]	0.00
Materials for energy recovery	[kg]	0.00
Exported electrical energy	[MJ]	0.00
Exported thermal energy	[MJ]	0.00

3. SUMMARY

The members of EFCA feel it is vital to support the concrete industry by providing Model EPD to supply credible, verified information that can be used by concrete manufacturers in the production of their own product EPD. This will be essential in future to allow concrete to demonstrate its vital contribution to sustainable construction.
To meet this commitment EFCA will publish verified European Model EPD during the second half of 2015.

REFERENCES

[1] EN 15804:2012-04+A1 2013: Sustainability of construction works — Environmental Product Declarations — Core rules for the product category of construction products.

[2] EN 934-2:2009+A1:2012 Admixtures for concrete, mortar and grout – Part 2: Concrete admixtures – Definitions, requirements, conformity, marking and labelling.

[3] EN ISO 14025:2011-10: Environmental labels and declarations — Type III environmental declarations — Principles and procedures.

[4] IBU Product Category Rules: Institut Bauen und Umwelt e.V., Königswinter (pub.): Product Category Rules for Construction Products from the range of Environmental Product Declarations of Institut Bauen und Umwelt (IBU), Part A: Calculation Rules for the Life Cycle Assessment and Requirements on the Background Report. 2013/04, www.bau-umwelt.de

PRODUCT CATEGORY RULES FOR CONCRETE TC 104 SC1 TG 20 ACTIVITIES

Jean-Marc Potier

Technical Manager SNBPE and Sales Manager at Vicat, France

Abstract

Product Category Rules (PCR) are the rules for writing Environmental Product Declarations, and many CEN Technical Committees are writing specific PCR for their product. CEN/TC104/SC1 *Concrete - Specification, performance, production and conformity* established Task Group 20 to draft PCR for concrete.

The methodology adopted was to comment on and amplify each paragraph in EN 15804 (the general PCR for construction products) and to add specific information and requirements for concrete where needed. The objective was to write a single PCR for all concretes covered by EN 206, though this is proving difficult, as TC229 has also drafted a PCR for Concrete Products, already sent for CEN Enquiry. A joint group from the two TCs is currently examining the practicality of having a single PCR.

The main topics of the PCR are: definition of the content of each life cycle stage; definition of limits between end of life stage and the period beyond the product life cycle; consideration of carbonation (a negative emission of CO_2) during use and end of life stages; precision on allocation of co-products (e.g. GGBS, fly ashes); establishment of characterisation factors for sand, gravel, clay and limestone.

Keywords: EPD; PCR; carbonation; sustainability

1. INTRODUCTION

CEN TC 350 *Sustainability of construction works* is responsible for the development of voluntary horizontal standardized methods for the assessment of the sustainability aspects of new and existing construction works and for standards for the environmental product declaration of construction products (See Fig. 1).

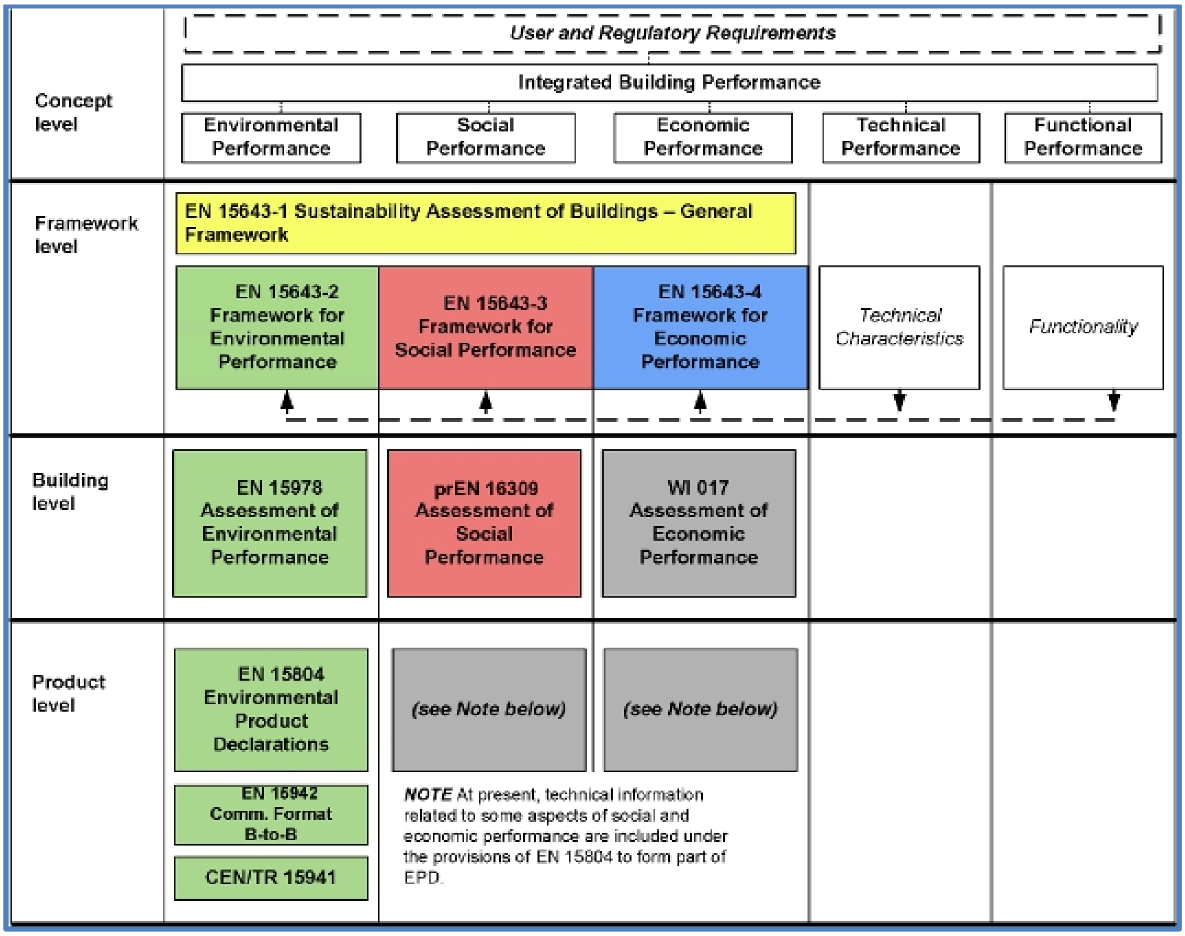


Figure 1: CEN/TC350 standard overview

In 2012 CEN TC 350 published **EN 15804**: *Sustainability of construction works - Environmental product declarations - Core rules for the product category of construction products.*

EN 15804 provides core product category rules for all construction products and services. It provides a structure to ensure that all Environmental Product Declarations (EPD) of construction products, construction services and construction processes are derived, verified and presented in a harmonised way. To be implemented, this standard needs to be completed by specific rules, Product Category Rules (PCR), for every type of construction product.

Concerning Concrete, CEN TC 104 *Concrete and related products* gave to CEN TC 104/SC1/TG 20 the task of drafting PCR for concrete with the aim of having, if possible, one PCR for all concretes covered by EN 206. ERMCO is well represented in this Task Group.

The following principles were adopted:

- to comment on and complete each paragraph of EN 15804 and add more precise information or special requirements for concrete where needed;
- to try to draft a single PCR for precast and cast in situ concrete; if not possible, two different PCR will be drafted;
- to produce a PCR in the form of a standard, and, if too much guidance is needed, instead of long notes or informative annexes, to draft a CEN TR in addition to the "short" standard.

2. PROJECT OF CEN TC104 SC1 PRODUCT CATEGORY RULES

2.1 Scope

This future standard will complement the core rules for the product category of construction products as defined in EN 15804 and will be used in conjunction with that standard.

It applies to Concrete and Concrete products for building and civil engineering.

This document will define the parameters to be reported; what EPD types (and life cycle stages) to be covered; what rules to be followed in order to generate Life Cycle Inventories (LCI) and conduct Life Cycle Impact assessment

In addition to the common parts of EN 15804, this European Standard for concrete and concrete products will:

- define allocation procedures for reuse and recycling;
- include the rules for calculating the LCI and the LCIA underlying the EPD;
- provide guidance/specific rules for the determination of the reference service life (RSL);
- give guidance on the establishment of default scenarios[1].

2.2 General aspects

The PCR will recommend the provision of an EPD based on a cradle to grave (modules A1 to C4, see Fig. 2) assessment for a defined scenario to enable the assessment of the environmental performance of construction works during their life cycle.

It will include a reminder that comparison of the environmental performance of construction products using the EPD information shall only be based only on the product's use in, and its impacts on, the whole building or civil engineering work, and shall take into account the complete life cycle (all information modules).

[1] The description of the unit (functional or declared) is the basis for developing the life cycle scenario (e.g. different scenarios for a beam and for a foundation in the construction stage).

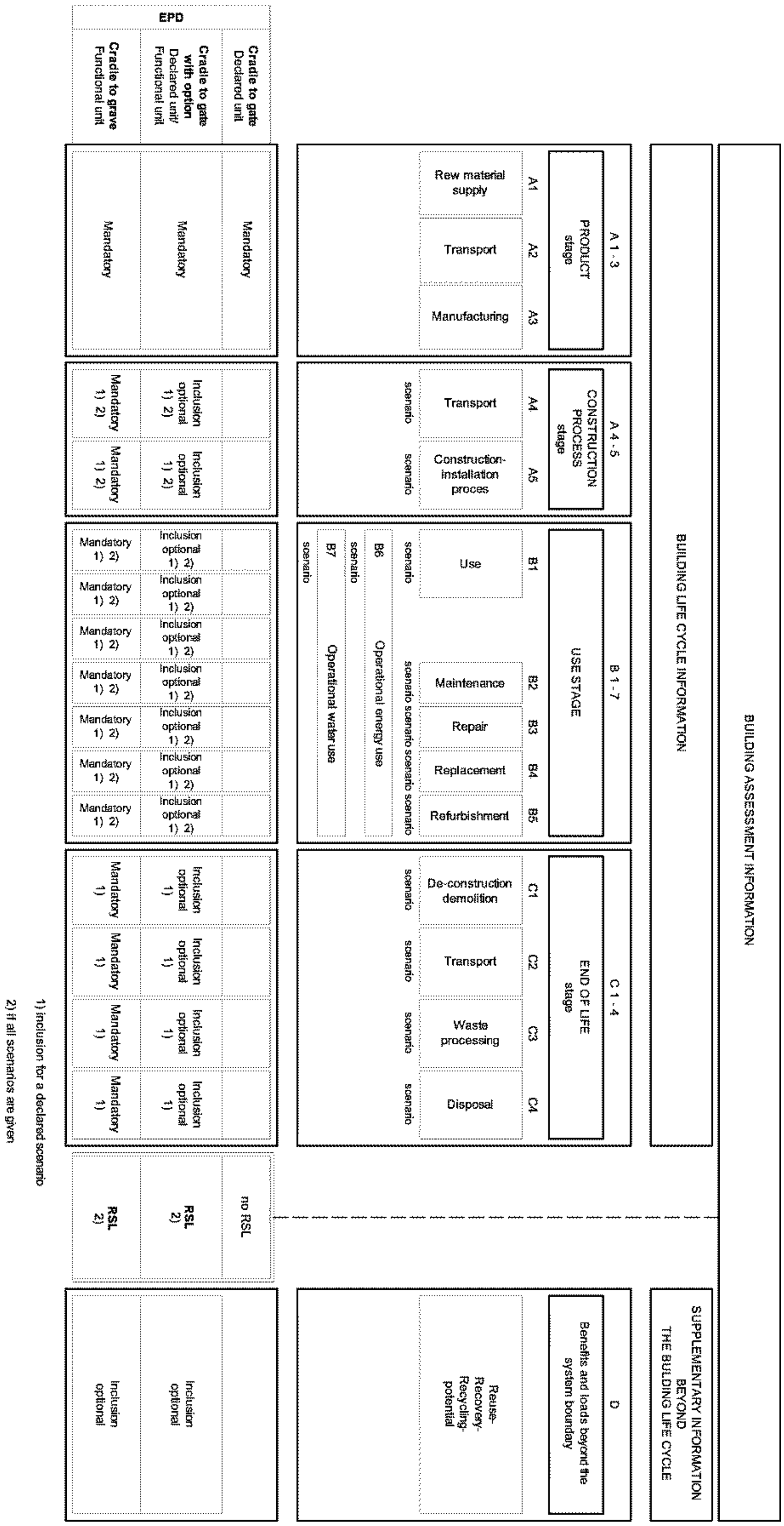


Figure 2: Types of EPD with respect to life cycle stages covered and life cycle stages and modules for the building assessment

2.3 Carbonation of concrete

The PCR proposes to take into account carbonation of concrete and give guidance for its calculation:

"Carbonation is a natural process during the life cycle of concrete that shall be considered during the use and the end-of-life stages of the product. Its rate varies considerably depending on the type of concrete, the environmental conditions in use, and the end-of-life scenario."

If no data are available, it may conservatively be considered that there is no CO_2 absorption. Otherwise, Annex D of the PCR provides several ways to assess carbon dioxide uptake through carbonation in the different life cycle stages depending on the above parameters.

Carbonation is not a degradation process of concrete. It is a natural process, actually increasing concrete strength. It is initiated at the surface, and with time, penetrates the concrete. If carbonation reaches the reinforcement, and if moisture and oxygen are available corrosion can start, due to lowered pH-value. If not, carbonation has no impact on durability of concrete. This phenomenon occurs naturally and is familiar in concrete design. The risk of corrosion is avoided by prescribing adequate cover to reinforcement or by an "equivalent performance based" approach. Such requirements are established as a fundamental part of design provisions for concrete (for instance Eurocode 2: EN 1992-1-1).

CO_2 is released during cement manufacturing. One part of the released CO_2 is due to calcination of limestone in cement clinker production; the other part is from the combustion of fuel at the cement plant. A mean value of 50% for each contribution can be taken, but the increased use of alternative fuels reduces the combustion part.

Only the calcination part can be recovered by carbonation. This means that concrete during its service life and after demolition, is potentially able to absorb a substantial part of the CO2 emissions from the calcination in the cement process.

The estimated mean CO_2-uptake [kg CO_2/m^3 concrete] for buildings and civil engineering in different strength classes is given in Table 1. The assumed service life (t) is 100 years. For service life other than 100 years the figures can be multiplied by a factor $\sqrt{t} / 10$.

Table 1: Estimated mean CO_2-uptake [kg CO_2/m^3 concrete] for buildings and civil engineering (Lagerblad 2005).

Concrete strength	15-20 MPa	25-35 MPa	≥ 35 MPa
Civil engineering structures	**12**	**7**	**5**
Exposed to rain	10	5	4
Sheltered from rain	24	13	10
In ground	4	2,5	2
House buildings	**25**	**20**	**12**
Outdoor			
Exposed to rain	24	17	10
Sheltered from rain	52	42	25
Indoor in dry climate			
With cover	29	23	14
Without	38	34	20
In ground	11	7	5

2.4 System boundaries

System boundaries for end of life of concrete are defined, and specially the limits between module C and module D

After demolition/deconstruction, the coarse concrete debris and any concrete elements retrieved from the structure are considered to be waste (cell 2 in Figure 3). The impacts of any partial separation of reinforcing steel at this stage and stockpiling of the waste shall be included in module C3, *Waste Processing*.

The crushing of the concrete till the “end of waste” status had been reached is also included in module C3

The benefits of the substitution of natural gravel by recycled aggregates are taken into account in module D, *Beyond System Boundary*.

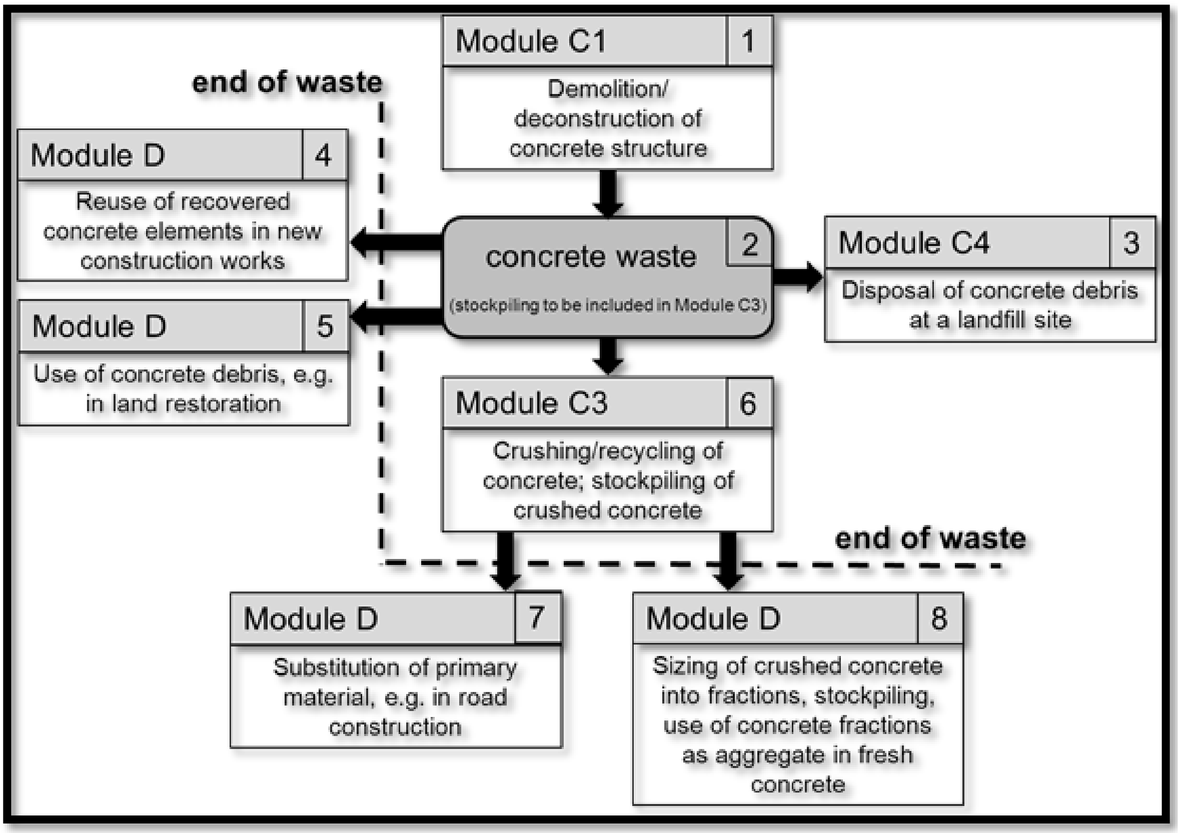


Figure 3: Typical processes at the end-of-life of concrete and concrete products and their assignment to the life cycle modules C1-C4 and D (transport processes not shown)

2.5 Co-product allocation

The PCR proposes guidance for Co-product allocation, based on paragraph 6.4.3.2 of EN15804: "Generally, according to EN 15804, for additions such as slag and fly ashes, allocation is based on economic values (or neglected) since the difference in value between steel and slag or between electricity and fly ashes is high."

2.6 Characterisation factors

EN 15804 gives only characterisation factors[2] for chemical elements (Calcium, Silicium, iron, …), as it is impossible to give the exact chemical composition of each aggregate used, in addition to table C.2 of EN 15804, the following characterisation factors are defined:

[2] : characterisation factors are used to quantify the impact of substances for abiotic depletion potential, the scarcity of each substance being expressed as "kg antimony equivalent", antimony being one of the rarest element on earth. The characterisation factor of iron is **5,24 E-08**

Table 2: Characterisation factors

Substance	Unit	Group	Initial emission or extraction	Characterisation factor kg antimony eq.	Comment
Clay	kg	element	resources	1.4 E^{-11}	assumed as Silica
Bentonite	kg	element	resources	1.4 E^{-11}	assumed as Clay
Limestone	kg	element	resources	0	assumed as Calcium
Gravel (unspecified)	kg	element	resources	1.4 E^{-11}	assumed as Silicium
Silica (SiO2)	kg	element	resources	1.4 E^{-11}	assumed as Silicium
Sand (unspecified)	kg	element	resources	1.4 E^{-11}	assumed as Silicium

3. PCR OF TC 229 AND CURRENT PROGRESS OF STANDARDISATION

TC 229 has also drafted a PCR for concrete **precast** products. Most of the text was similar to that produced by TC104, but there were also some differences. This PCR has been sent to CEN enquiry. The result was a possible acceptance as a European standard (YES 19 – NO 2 – Abstains 9), but the two negative votes, coming from Germany and Switzerland, as well as a great number of comments, were based on the demand for a single standard (CEN/TC 229 and CEN/TC 104). TC 104 itself also commented on this PCR, suggesting a single document covering both precast and in-situ concrete.

After a meeting between the two chairmen (TC 104 and TC 229) it was agreed that there should be a single standard with a common working structure endorsed by the two TCs.

As the CEN rules do not allow new joint working groups, the proposal is to give to CEN/TC 229 the lead of the proposed WG with a convenor nominated by CEN/TC 104.

The proposal to allocate the work to a WG of CEN/TC 229 is based on:

- Many multi-materials CEN Technical Committees are writing products standards for precast concrete products (TC 50, TC 125, TC 128, TC 165, TC 166, TC 178, TC 226, etc.). CEN/TC 229 has been nominated liaisons organization with these TCs and has recommended that they take into account the CEN/TC 229 PCR for writing the EPD on their products.
- Most precast concrete product standards are harmonized standards under the Construction Products Regulation (CPR). Basic Work Requirement 7 *Sustainable use of natural resources* of the CPR may require the introduction in the annex ZA some essential characteristics related to « sustainability » according to national regulations.
- Some concrete products standards do not refer to EN 206 (masonry, paving blocks, pipes, etc.).

It is hoped that a single PCR document will ensure consistency, reliability and clarity for the concrete sector. Such a document could be ready for a new CEN enquiry by the end of 2015.

If the enquiry is positive, the PCR covering both in-situ (ready mixed and site mixed) concrete and precast concrete products could be published by mid-2016.

SUSTAINABILITY – A DRIVER FOR CONCRETE INNOVATION

H. Justnes (1) and T. A. Martius-Hammer (1)

(1) SINTEF Building and Infrastructure, Trondheim, Norway

Abstract

Production of cement is ranking 3rd in causes of man-made carbon dioxide emissions world-wide. Thus, in order to make concrete more sustainable concrete innovation should move along one or more of the following routes; 1) Replacing cement in concrete with larger amounts of supplementary cementing materials (SCMs) than usual, 2) Replacing cement in concrete with combinations of SCMs leading to synergic reactions enhancing strength, 3) Producing leaner concrete with less cement per cubic meter utilizing plasticizers, 4) Making concrete with local aggregate susceptible to alkali silica reaction (ASR) by using cement replacements, thus avoiding long transport of non-reactive aggregate, and 5) Making concrete with local aggregate manufactured from crushed rock.

1. INTRODUCTION

The current focus on sustainability in the society and appurtenant demands and requirements for increased life time and reduced resource consumption, energy consumption and CO_2-emission, as well as improved productivity, forces also the building sector to be more innovative in order to meet these challenges. This recognition was also the basis for the work that was conducted in COIN – the Concrete Innovation Centre in Norway (www.coinweb.no), on which results this paper is based.

Concerning concrete, the CO_2-emission associated with cement production has been pointed out as the main challenge, and therefore paid most attention. The cement industry world-wide is calculated to bring about 5-8% of the total global anthropogenic carbon dioxide (CO_2) emissions. The general estimate is about 1 tonne of CO_2 emission per tonne clinker produced, if fossil fuel is used and no measures are taken to reduce it. The 3rd rank is not because cement is such a bad material with respect to CO_2 emissions, but owing to the fact that it is so widely used to construct the infrastructure and buildings of modern society as we know it. Concrete is actually among the more environmentally friendly materials since it is composed in general of about 1 part cement, 0.5 parts water and 5-6 parts of sand and gravel (i.e. aggregate). The world's cement production was roughly 4 billion tons in 2013, meaning roughly 24 billion tons concrete or 10 billion m^3 concrete. This quantity can be translated into making a concrete cylinder of about 20 cm diameter reaching the moon and back to earth every day or building a solid concrete block with 1 km^2 base reaching higher (10 000 m) than Mount Everest (8 848 m) in a year!

A lot is done by cement producers to reduce the global carbon footprint, in particular to replace coal with waste having a calorific value equivalent to (fossil) fuel and by making blended cement where part of the clinker is replaced with supplementary cementing materials (SCMs). However, cement is a bulk product that should cover a wide range of applications and serve different customers, giving limitations on clinker replacements.

Concrete, on the other hand, is the end product where the performance criteria are already specified and depending on application more can be done to increase its sustainability. This paper discusses the potential achievements and challenges by

- Replacing cement in concrete with larger amounts of SCMs, also uncommon ones like calcined marl
- Replacing cement in concrete with combinations of SCMs leading to synergic reactions enhancing strength
- Producing leaner concrete with less cement per cubic meter utilizing plasticizers.
- Making concrete with local aggregate susceptible to alkali silica reaction (ASR) by using cement replacements, thus avoiding long transport of non-reactive aggregate

Other aspects adding to more sustainable concrete are not discussed in this paper, like:

- Making concrete with recycled aggregate from demolished concrete structures.
- Making more durable concrete with less maintenance and longer service life.
- Making slimmer structures with high strength concrete. More cement per cubic meter, but less cubic meters
- Utilizing the heat capacity of bare concrete to save energy for heating/cooling of offices/housing
- Use of fibres replacing traditional rebars

- Use of lightweight aggregates
- Use of self-compacting concrete

2. CONCRETE WITH HIGH CONTENT OF SCM

Replacing parts of the cement in concrete with SCMs, or making blended cement where clinker is partly replaced with SCMs, is the fastest short term remedy to reduce CO_2 emissions from the cement and concrete industry. Blast furnace slags as latent hydraulic SCMs or pozzolana consuming calcium hydroxide, e.g. fly ash, are the most common ones. Limestone powder is also used as filler, in particular in self-compacting concrete.

In Europe most granulated blast furnace slag is put to good use, and high quality fly ash is becoming scarce in some areas. Other pozzolana used in smaller dosages, but often in combination with others, are silica fume, rice husk ash and metakaolin. However, the latter ones are not available in sufficiently large amounts, or are too expensive, to have large impacts on lowering CO_2 emissions, even though they are contributing. Thus, there has been a search for large volumes of unexploited resources and one possibility found is calcareous clay or *marl* unsuitable for clay industries producing brick or lightweight aggregate (LWA). Since *common blue clay* is so abundant, this may also be a viable option. Both common blue clay and marl will have to be calcined at 750-850°C prior to use as pozzolana. Be aware when using clays that they can contain chlorides if they originate from marine deposits, so the chloride content must always be checked prior to use in reinforced concrete.

Justnes et al. [1] tested calcined blue clay as a stabilizer for SCC (self-compacting concrete) by replacing limestone filler. As can be seen from Fig. 1, increasing clay replacement increased yield stress while the viscosity was relatively stable. In addition, the compressive strength increased with increasing clay replacement at all ages tested as depicted in Fig. 2. The starting point was 30% by volume limestone filler and then 1/3 and 2/3 of this was replaced with calcined clay.

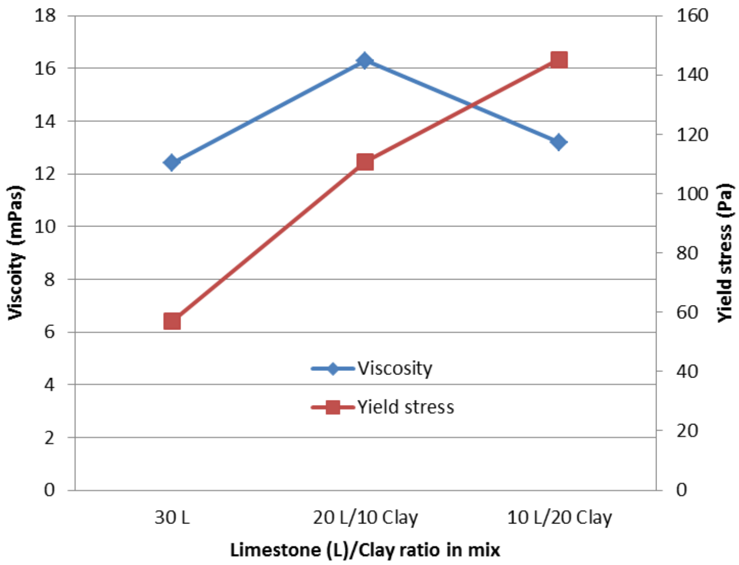


Figure 1: The viscosity and yield stress of SCC where limestone powder (L) is increasingly replaced by calcined blue clay (Clay) from Justnes et al. [1].

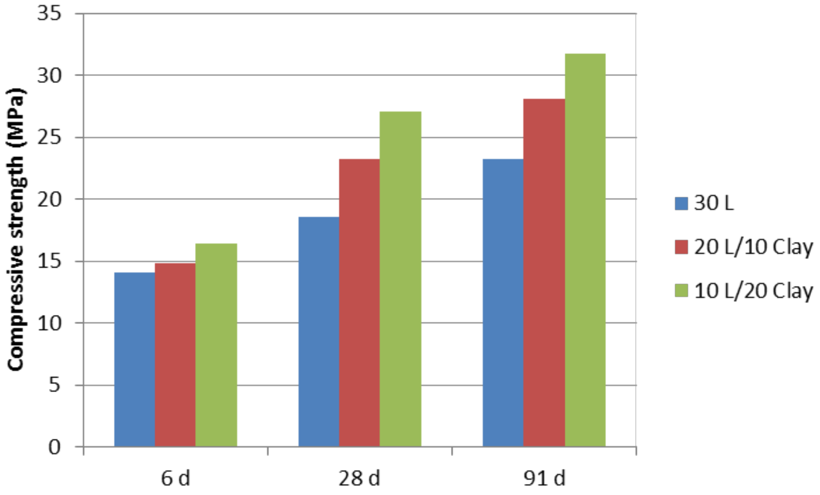


Figure 2: Compressive strength evolution of SCC where limestone powder (L) is replaced by calcined blue clay (Clay) from Justnes et al. [1].

Calcined marl as an effective pozzolan was published by Justnes et al. [2]. Compressive strengths of mortar where cement was partly replaced with marl calcined in pilot scale rotary kiln as a function of curing age is plotted in Fig. 3 for 50% calcined marl replacing cement. Note that same strength as reference is obtained at 28 days and sufficient strength (≈ 10 MPa) for demoulding at 1 day when cured at 20°C / 90% RH.

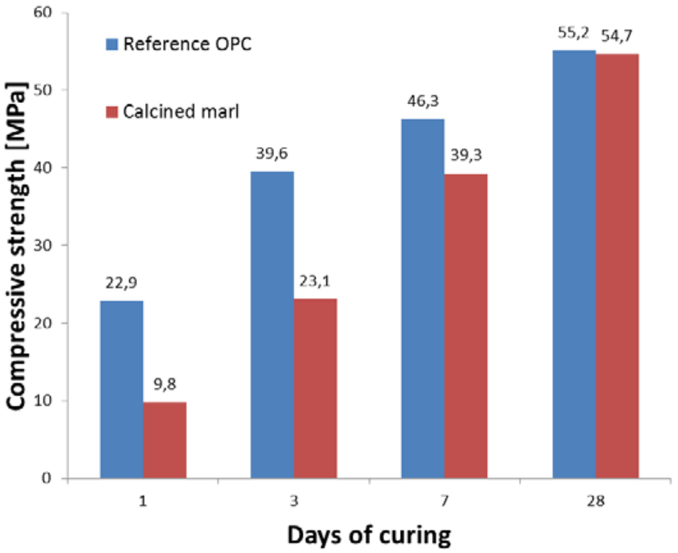


Figure 3: Compressive strength of mortars with 0 (reference) and 50 % replacement of cement by calcined marl cured for 1, 3, 7 and 28 days [2].

The compressive strengths of mortar with cement partly replaced by marl calcined in a full scale (industrial) rotary kiln as function of replacement level (up to 65%) and age (up to 2 years) has been published by Justnes and Østnor [3] together with durability properties and microstructure of the same mortars.

When large amounts of cement are to be replaced by SCMs, the challenge is usually rather low early strength that might be needed to be boosted in order to maintain productivity or construction rate. This can either be done by *mechanical* or *chemical* activation. Hoang [4] showed that it was possible to increase the strength of mortar where cement has been replaced by 30% fly ash by 60% after 2 days at 5°C and by 30% after 1 day at 20°C using a ternary accelerator at a dosage of only 0.35% of cement/fly ash mass.

3. CONCRETE WITH TERNARY BINDERS

Limestone is widely available and either used as raw meal for clinker making or as filler either in cement or in concrete. Fly ash from coal fired energy plants is still abundant in many countries like India and China. Combining fly ash with limestone will give a strength increase as a result of a chemical synergy. The theoretical consideration of combining limestone and siliceous fly ash is based on a delicate balance (Matschei et al. [5-7]) between ettringite, calcium monosulphoaluminate hydrate and calcium monocarboaluminate hydrate, and it is all about maximizing water binding. Since the cement has not enough aluminate to utilize this effect fully, a secondary aluminate forming SCM had to be introduced, e.g. fly ash. Hence, cement, limestone and for instance fly ash could then be denoted a *ternary binder*.

Lothenbach et al. [8-9] used thermodynamic modelling to calculate the phase assembly and porosity of cement with limestone and found good correlations with strength evolution.

De Weerdt and Justnes [10] studied fly ash – limestone - calcium hydroxide mixes prepared with a high alkaline solution (pH = 13.2). A clear interaction between fly ash and limestone powder was observed. More water was bound relative to the fly ash content and the hydration products formed were altered. The calcium aluminate hydrates formed during the pozzolanic reaction of fly ash reacted with the calcium carbonate of the limestone powder and formed calcium carboaluminate hydrates. The interaction between the calcium carbonate of the limestone powder and the aluminate phase of clinker has been investigated by several researchers (e.g. Lothenbach et al. [9]). In the presence of small amounts of limestone powder, hydration products alter. This can cause the total volume of the hydration products to increase (Matschei et al. [6], Lothenbach et al. [8]) resulting in an increase in strength and a decrease in permeability. The effect of this interaction in OPC is however not so pronounced due to the limited aluminate content. Fly ash on the other hand can have relatively high aluminate content. The chemical interaction between calcium aluminate hydrates and calcium carbonate might therefore be of greater importance in cement with high fly ash content or other alumina containing SCMs leading to increased amounts of calcium aluminate hydrates.

This means that fly ash and limestone powder not only compensate for each other's shortcomings when it comes to short- and long-term strength (i.e. additive effect), but there is also a true chemical interaction between the calcium carbonate of the limestone and the calcium aluminate hydrates formed by the pozzolanic reaction (i.e. a synergic reaction) as now thoroughly documented by De Weerdt et al. [11-13].

De Weerdt and Justnes [10] showed that limestone will react with the pozzolanic product of fly ash to form calcium carboaluminate hydrate and also that this reaction contributes to strength [11]. Fig. 4 from the latter study shows that 5% limestone in combination with 30% fly ash replacement of Portland gives higher 28 day strength that with 35 % fly ash replacement alone, and even higher than 30% fly ash replacement only (marked as square). This means that 5% limestone in this case has the same effect on strength as 5% cement!

The preceding synergic principle will probably work for limestone in combination with other SCMs producing calcium aluminate hydrates in their reaction; e.g. blast furnace slag, metakaolin and calcined blue clay as indicated by the results in Fig. 2. Calcined marl is a natural combination of calcined clay and calcium carbonate providing it is calcined at <850°C The synergic reactions between the calcined clay part and the remaining calcium carbonate contribute to the excellent strength documented in Fig. 3 when calcined marl replaces cement.

4. MAKING LEANER CONCRETE

The principal different ways of using plasticizers or water reducing admixtures (WRAs) in concrete technology is sketched in Fig. 5 [14]. However, the most sustainable way is to save cement and water by the use of WRA while maintaining strength and workability relative to reference without WRA.

Most concrete producers use WRA to save cement for economic reasons, but at the same time it gives ecological benefits towards more sustainable construction. As a rule of thumb with modern polycarboxylate based superplasticizers (PCS); 1 kg PCS reduces 20 kg water per m^3 concrete. Using this rule for a concrete recipe with 350 kg cement and 1.8 kg PCS /m^3 and keeping w/c = 0.60 constant, means then 290 kg cement (i.e. 60 kg or 17% cement saved). Similar calculations can be done for any WRA knowing how many litres of water can be saved per kg WRA (often stated by the admixture producer). The recipe is often economically optimized by having cement and water to a slump of 30 mm and then adding WRA to obtain a 200 mm slump. Collepardi [15] showed experimentally that the rule of thumb above is valid.

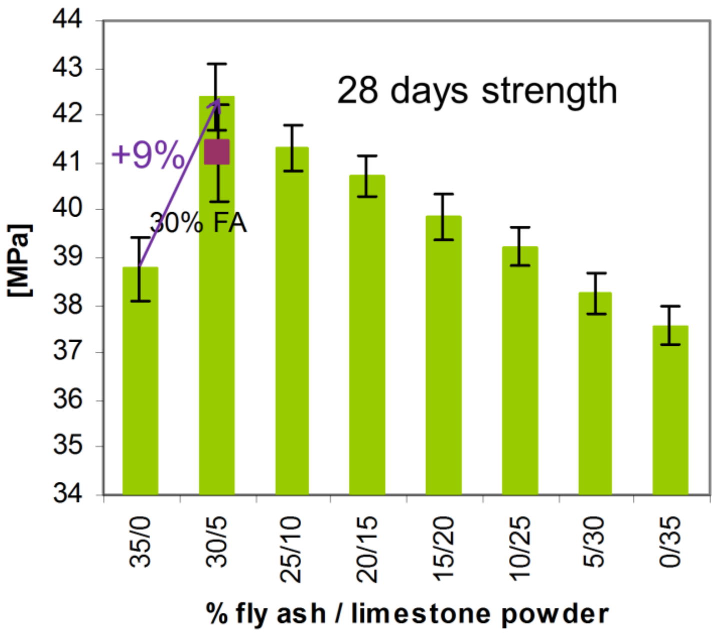


Figure 4: Comparing the 28 day compressive strength of composite cements with different combination of fly ash (FA) and limestone powder (L) from [11].

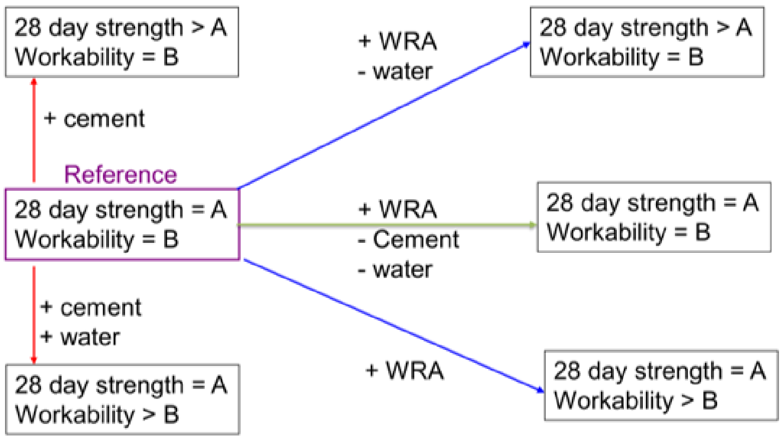


Figure 5: A sketch of different ways of utilizing water reducing agents (WRA) in concrete technology from Rixom and Mailvaganam [14].

5. CONCRETE TOLERATING ALKALI-REACTIVE AGGREGATE

The immediate conversion of alkali hydroxides to silicates by silica fume (SF) will result in reduced alkali aggregate reactions. However, Bérubé and Duchesne [16] showed that SF merely postpones expansion due to AAR. Nevertheless, SF as a remedy against AAR, together with other improvements in construction procedures, has found its application in Iceland where all cement has been interground with 7-8 % CSF to combat the problem [17]. In Norway, the cement is interground with 20% siliceous fly ash in order to be able to use ASR-prone aggregate. Such solutions will contribute to more sustainable solutions as long transport distances of high quality aggregate is avoided and high quality deposits will last longer.

6. MANUFACTURED (LOCAL) AGGREGATE

A fast-approaching coming shortage of traditional aggregate resources, firstly sand and gravel, has led to a need for alternative sources and technologies. Norway has seen a development towards more crushed aggregates from hard rock. Sustainability concerning access to and handling of aggregate resources is a challenging issue for an industry supplying highly needed materials, an industry which also is responsible for building up huge stockpiles of un-sellable crusher fines. Important elements of sustainability are resource efficiency, no-waste production, recycling and effective logistics. Even though production of manufactured sand requires more energy than corresponding production of natural sand, the vicinity to the market, with less transport, will make manufactured sand environmentally favourable. A new and different development approach has been attempted in COIN by in bringing together and facilitating the crucial interaction between the professionals from the different involved industries (quarrying machinery supplier, aggregate producers, concrete producers and concrete contractors) and the academic people from universities and research institutions, in order come up with a better crushed sand solution for the future. The concept has been a zero-waste technology for aggregate production, where instead of reducing the amount of the crushed fines their properties are rather engineered to crucially increase the overall performance of the sand in concrete. The project also involves collaboration with a state-of-the-art aggregate production plant where the new technology has already been implemented. The production process there is based on the new engineered sand concepts successfully supplying 100% all of the produced fractions to concrete and asphalt producers [18].

7. CONCLUSIONS

Concrete can never be made sustainable since it is based on non-renewable mineral resources. However, concrete can be made more sustainable (or less un-sustainable) by replacing cement with supplementary cementing materials (SCMs) based on industrial by-products like slag and fly ash. Larger amount of SCMs can be used if loss in early strength is counteracted by finer grinding or special grinding (mechanical activation) or accelerators. Cement with interground silica fume and/or fly ash can even allow the use of alkali reactive aggregate and further save environment (i.e. local deposits and less transport). New SCMs like calcined blue clay or marl can replace fly ash in areas where high quality fly ash becomes scarce or is unavailable (other than by long transport). Water reducing agents can be used to make concrete with less overall binder and same strength and workability, and thereby more environmentally friendly. Making more concrete with less cement clinker content means less CO_2 emission to the atmosphere and less use of limited natural resources and thereby more sustainable construction.

ACKNOWLEDGEMENTS

Financial support from the Norwegian ready-mix concrete association (FABEKO) is greatly appreciated, while most of the results comes from COIN (see www.sintef.no/coin).

REFERENCES

[1] Justnes, H., Østnor, T. A., De Weerdt, K. and Vikan, H., "Calcined Marl and clay as mineral addition for more sustainable concrete structures", *Proceedings of the 36th International Conference on Our World in Concrete & Structures*, 14-16 August 2011, Singapore, 10 pp. (ISBN: 978-981-08-9528-0)

[2] Justnes, H., Østnor, T. A. and Danner, T., "Calcined Marl as Effective Pozzolana", *Proceedings of the International RILEM Conference on Advances in Construction Materials Through Science and Engineering*, RILEM PRO 79, 5-7 September, 2011, Hong Kong, China, 8 pp.

[3] Justnes, H. and Østnor, T.A., "Durability and microstructure of mortar with calcined marl as supplementary cementing material", *Proceedings of the XIII conference on Durability of Building Materials and Components (DBMC)*, Sao Paulo, Brazil, September 3-5, 2014, pp. 771-780

[4] Hoang, K.D., "*Hardening Accelerator for Fly Ash Blended Cement*", Doctoral Thesis at the Norwegian University of Science and Technology (NTNU), Faculty of Engineering Science and Technology, Department of Structural Engineering, Trondheim, Norway, No. 2012:366, 195 pp.

[5] Matschei, T., Lothenbach, B. and Glasser, F.P., "The AFm phases in Portland cement", *Cement and Concrete Research*, 37, 118-130.

[6] Matschei, T., Lothenbach, B. and Glasser, F.P., "The role of calcium carbonate in cement hydration", *Cement and Concrete Research*, 37, 551-558.

[7] Matschei, T., Lothenbach, B. and Glasser, F.P., "Thermodynamic properties of Portland cement hydrates in the system CaO - Al_2O_3 - SiO_2 - $CaSO_4$ - $CaCO_3$ - H_2O", *Cement and Concrete Research*, 37, 1379-1410.

[8] Lothenbach B., Le Saout G., Galluci E. and Scrivener K., "Influence of limestone on the hydration of Portland cements", *Cement and Concrete Research*, 38, pp. 848-860.

[9] Lothenbach, B., Matschei, T., Möschner, G. and Glasser, F.P., "Thermodynamic Modelling of the Effect of Temperature on the Hydration and Porosity of Portland Cement", *Cement and Concrete Research*, 38, 1-18.

[10] De Weerdt K. and Justnes H., "Microstructure of binder from the pozzolanic reaction between lime and siliceous fly ash, and the effect of limestone addition", *RILEM Proceedings PRO 61, Microdurability Conference*, Nanjing, Vol. 1, 2008, pp.107-116.

[11] De Weerdt, K., Justnes, H., Kjellsen, K.O. and Sellevold, E., "Fly Ash-Limestone Ternary Composite Cements: Synergy Effect at 28 days", *Nordic Concrete Research*, 42 (2), 51-70 (ISBN: 978-82-8208-023-1).

[12] De Weerdt, K., Sellevold, E., Kjellsen, K.O. and Justnes, H., "Fly ash – Limestone Ternary Cements – Effect of Component Fineness", *Advances in Cement Research*, 23 (4), 203-214.

[13] De Weerdt, K., Ben Ha-Ha, M., Le Saout, G., Kjellsen, K.O., Justnes, H. and Lothenbach, B., "Hydration mechanism of ternary Portland cements containing limestone powder and fly ash", *Cement and Concrete Research*, 41 (3), 279-291.

[14] Rixom, M.R. and Mailvaganam, N.P., (1999). "*Chemical Admixtures for Concrete*", 2nd Ed., 1999, Spon Press.

[15] Collepardi, M., "Combined use of chemical admixtures and polymer macro-fibres in crack-free industrial concrete floors without wire-mesh", *Proceedings of the Jean Péra Symposium on Special Cements and Sustainability Issues*, Ed. Kamal H. Khayat, held in conjunction with the *9th CANMET/ACI International Conference on Recent Advances in Concrete Technology*, Warszawa, Poland 23-25 May, 2007, pp. 53-65.

[16] Bérubé, M. -A. and Duchesne, J., (1992). "Does Silica Fume merely postpone Expansion due to Alkali-Aggregate Reactivity?", *Proceedings of 9th International Conference on Alkali-Aggregate Reaction in Concrete*, 27-31 July, 1992, London, pp. 71-80.

[17] Asgeirsson, H., (1986). "Silica Fume in Cement and Silane for counteracting of Alkali-Silica Reactions in Iceland", *Cement and Concrete Research*, 16 (3), 423-428.

[18] Cepuritis R., Danielsen S. W. (2014): "COIN Project: Towards a Zero-Waste Technology for Concrete Aggregate Production in Norway" – Geophysical Research Abstracts Vol. 16, EGU2014-14394, 2014 – EGU General Assembly 2014.

COMBATING ASR TO ENABLE USAGE OF LOCAL AGGREGATES IN TURKEY

Robert C Lewis (1) and Elif Bayrak (2)

(1) Elkem Silicon Materials, UK

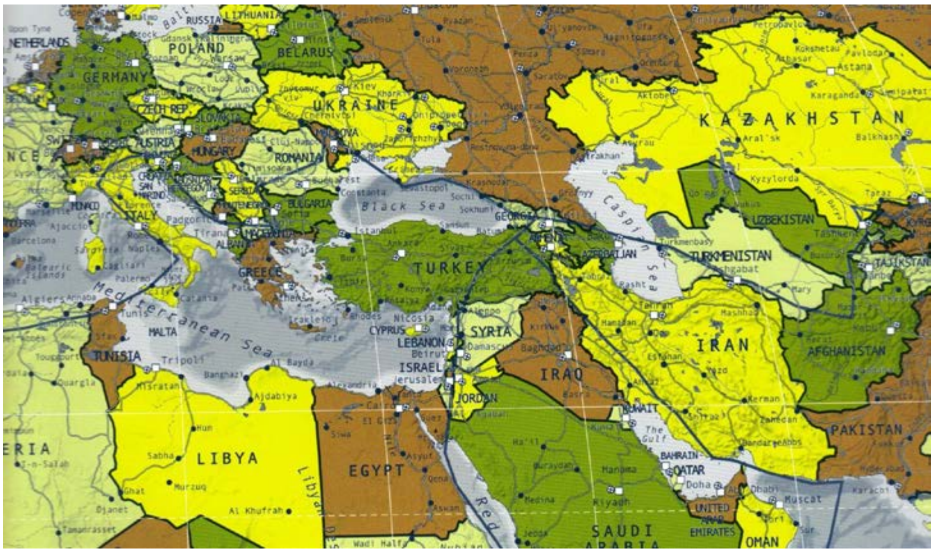
(2) KGM, Ankara, Turkey

Abstract

This paper will look at the use of silica fume to negate the high reactivity of aggregates in Turkey. Reference will be made to initial testing for ASR resistance in the early 1970's, then to local research by KGM in 2013/2014 and further testing in Iceland in 2015. The paper will show that the addition of silica fume in the area of 8% by weight of cement will control the ASR within the specified limits and enable the use of local aggregates, with known reactivity potential, in major concrete construction. Further research is expected to establish levels at which even higher reactivity aggregate may be safely used.

Keywords: Alkali silica reaction (ASR), pozzolans, silica fume, expansion values.

1. INTRODUCTION



A central position – from the ancient to the modern world.

Turkey is at the junction of the European and Asian continents – a crossover for travelers for millennia – and to maintain that "fulcrum point of the world", is looking to improve its infrastructure and communications. To do this means a lot of building from scratch and a lot of upgrading. This includes airports, dams, roads, bridges and tunnels, ports and harbours and power services.

The airport network alone is a huge system to upgrade (from Turkish Airlines)

Such construction will use huge quantities of concrete, nominal and high performance, which will in turn mean vast tonnages of aggregates. The drawback to this is the reactivity level of the majority of aggregate in Turkey. The potential for Alkali Silica Reaction (ASR) from these aggregates means that a number of precautionary steps must be taken to prevent the reaction happening and causing damage and possible failure of structures. The alternative is to ship in low reactivity aggregates to blend with local materials and thus reduce the potential for ASR. With the aggregate tonnages being registered in many tens of millions, such action would be costly and unfriendly to the environment. Readymix Concrete Production Volumes for the past few years have been approximately:

- 2010 ~ 80 million m^3
- 2011 ~ 91 million m^3
- 2012 ~ 93 million m^3
- 2013 ~ 100 million m^3
- 2014 ~ 110 million m^3

At approximately 1,800kg of aggregate per cubic metre, the last couple of years have approached some 200 million tonnes of aggregate consumption. At even a 50:50 blend with a low reactivity aggregate, the import potential would be around 100 million tonnes. The use of pozzolans has long been known to combat ASR with silica fume being widely acknowledged as the highest effectiveness in resisting this form of concrete deterioration. The target behind the research discussed in this paper is that being able to control the ASR potential, using silica fume, will mean that more local aggregates can be used, negating the need for import and thus reducing both the cost of the construction and the damage to the environment.

2. HISTORY

Since the early 1970s when large scale filtering meant that sufficient volumes of silica fume were available for the concrete industry, research has been conducted on how to make the best use of its pozzolanic action. The superfine nature and high silicon dioxide content of the silica fume gave it two advantages in combating the ASR attack. The small size and perfect spherical particles gave both a packing effect – filling the voids between cement grains – and a ball bearing effect to give cohesion but also a thixotropic effect. Once the calcium hydroxide was produced by the cement hydration, the pozzolanic reaction took over filling the void space with calcium silicate hydrates adding more bond within the matrix and blocking the pores in the cement. This meant a dense concrete with high water resistance and the pozzolanic action mopped up excess alkalis ions and calcium hydroxide (Fig 1). Thus, three parts of the quartet required for ASR were greatly reduced or negated and thus the ASR was controlled. It was in 1979 that Iceland, with its highly reactive aggregates and high alkali cement ruled that all concrete made with local materials must contain 7 to 8% silica fume to combat the ASR (Fig 2).

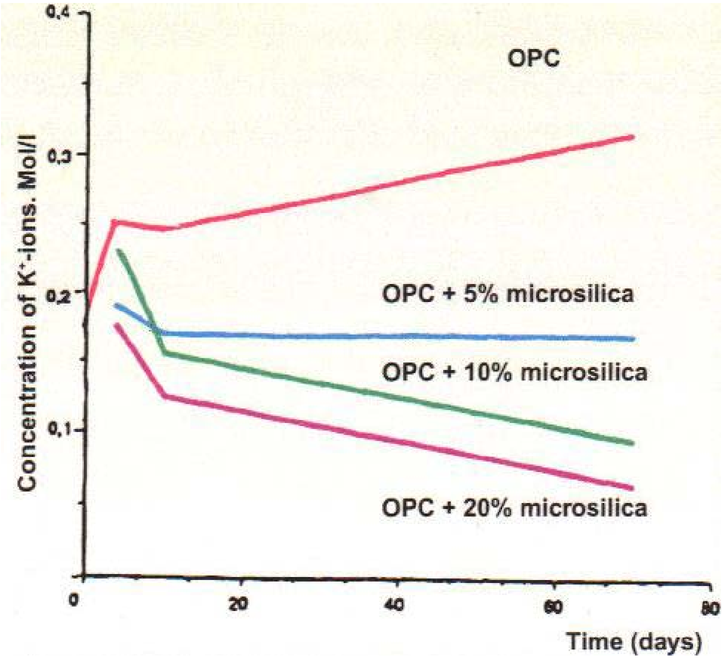


Figure 1: Reduction in Alkali ions with the addition of silica fume [1]

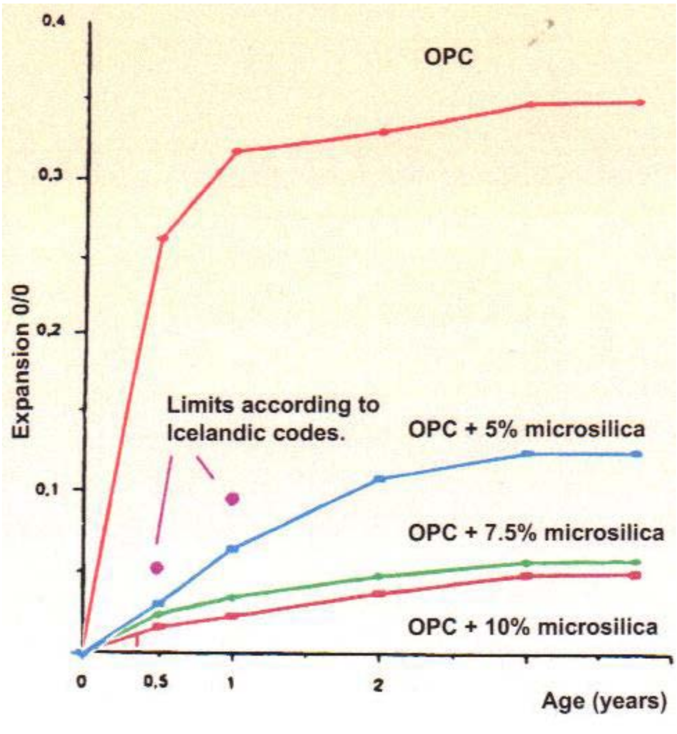


Figure 2: Reduction in Expansion with the addition of silica fume [2]

3. RESEARCH IN TURKEY

The use of concrete for roads and bridges and similar infrastructure in Turkey is covered by the current Highways Manual. The limitation on ASR expansion in this manual is 0.20% to allow aggregates to be used. As many of the local aggregates are around 0.30% to 0.50% expansion this becomes a difficult target. With some aggregates tested at around 0.80% expansion, the need for a solution to be able to use these increases. The Government Authority, DLH, through the KGM Highways department decided in 2013 to run some testing on the use of silica fume as an addition to concrete to combat the potential ASR with local materials. Using the Standard 14 day test and measuring the expansion for a series of silica fume dosages, the KGM research showed a very effective pattern of suppressing the ASR expansion, Table 1.

Table 1: Expansion values (%) by age and addition of silica fume [3]

Age \ SF	0	8%	10%	12%	14%
3 days	**0.051**	**0.016**	**0.014**	**0.016**	**0.013**
7 days	**0.161**	**0.015**	**0.010**	**0.008**	**0.004**
14 days	**0.334**	**0.060**	**0.025**	**0.014**	**0.008**

The values obtained showed that the use of silica fume at an 8% addition (by weight of cement) would limit the expansion of this level of reactive aggregate (0.30~0.40%) down to below 0.10%. This is half the target maximum of the Highways Manual. The values also showed that for each additional 2% of silica fume, the expansion at 14 days reduced by 50% from the previous level. The findings suggest that it may even be possible to use a dosage of between 8 and 10% silica fume to control reactive aggregates with values of 0.60 to 0.80%, reducing that expansion down to below 0.1%. A graph of the data is shown in figure 3.

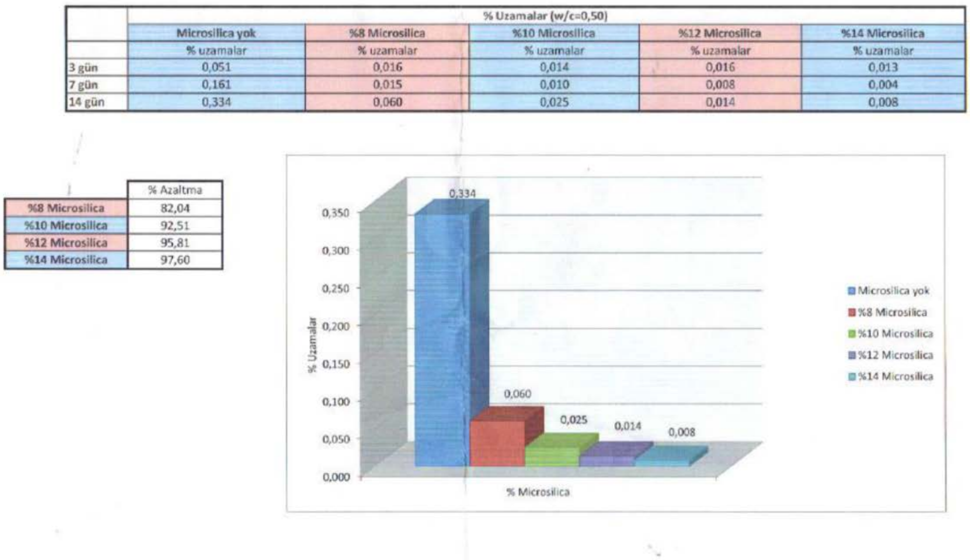


Figure 3: Tables and graph reproduced from KGM data [3]

The research at DLH / KGM is continuing to refine the levels of addition of silica fume needed for different types of reactive aggregates. The work may also include the use of other pozzolans such as fly ash or slag – also known to provide a level of resistance to ASR. The combination of these supplementary cementitious materials can achieve very high durability as well as numerous improvements to the concrete quality. It is hoped that this research will lead to the adoption of silica fume use within the Highways Manual.

4. WORK IN ICELAND

Although Iceland uses silica fume to suppress the ASR, it is known that new sources of aggregates can require changes in the addition rate to maintain a level of confidence in the concrete quality. Over the years since 1979 the addition rate of silica fume has reduced slightly – with the use of other materials and better superplasticisers – to around 6% by cement weight. Like Turkey, the idea is to blend high and low reactivity aggregates to get the best environmental sustainability and use the silica fume to ensure low expansion. A new sand source (Stokksnessandur) became available recently and testing showed that this had a potential expansion of between 0.40 and 0.50%. A 50% blend of this sand with a relatively inert sand (Raudamelssandur) would be the normal method of helping to control the ASR with the use of silica fume. Tests run at the Innovation Centre Iceland’s ‘Rheocenter’ showed that even at the 50% blend the addition of silica fume would have to be increased to approximately 8% (Figure 4, Ref 4.). To use the Stokksnessandur sand at 100%, rather than 50:50, would require an addition of silica fume at 9% to stay below 0.20% expansion or 11% to reach less than 0.10% expansion.

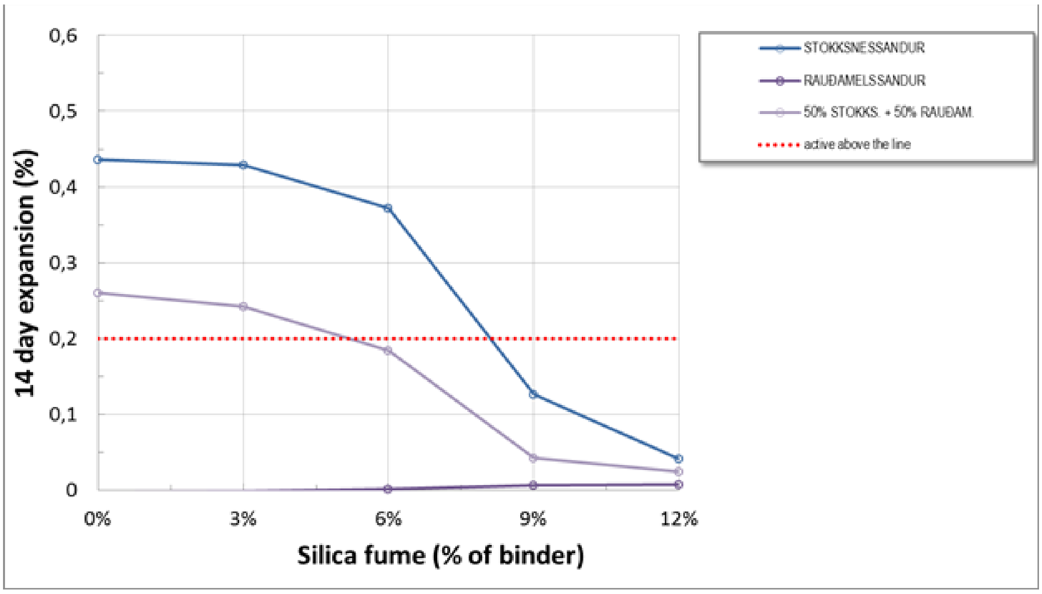


Figure 4: Expansion of individual and blended sand mixes with increasing silica fume [4]

5. SUMMARY

The current work in both Iceland and in Turkey substantiate the early results from the 1970s and 1980s, that show ASR can be resisted using silica fume as an addition to concrete. With ‘moderately reactive’ materials (0.30~0.40% expansion), this addition rate is around 8% by weight of cement. Higher reactivity aggregates can also be controlled in this manner, although test runs should be made to determine the optimum addition of silica fume.

The use of silica fume in concrete in Turkey should be highly considered by the authorities, in order to allow previously 'unusable' aggregates to be used, thus saving costs, energy and the environment through non-import of less reactive materials.

ACKNOWLEDGEMENTS

The primary author wishes to thank Elif Bayrak, the team at KGM and the DLH authority for their work on the research in Turkey and for allowing him use of the results so far achieved.

REFERENCES

[1] Diamond, Sidney: Journal of the American Concrete Society; Vol 66, No. 5, 1982.
[2] Asgeirson, Haraldur: Cement and Concrete Research; Vol 16, pp423-428, 1986.
[3] Elif Bayrak and team, KGM, Ankara – ongoing research, unpublished.
[4] Wallevik and Hjartarson, ICI, Iceland – ongoing research, unpublished.

TOOL FOR GENERATION OF EPDS IN NORWEGIAN CONCRETE INDUSTRY

Mie Vold (1), Trond Edvardsen (1) and Anne Rønning (1)

(1) Ostfold Research, Norway

Abstract

The cement and concrete industry in Norway has introduced Life Cycle Assessments (LCA) as a method to document the environmental properties of products and reduce the emissions of climate gas emissions. Environmental Declaration (EPD) is a proven effective way to document the environmental profile for a product or service. It is based on a life cycle perspective for a product or process.

Concrete producers typically produce different products for which the concrete composition varies from project to project. However, there are usually a limited number of ingredients that are mixed in different proportions. Thus, the Norwegian Ready Mixed Concrete and Precast Concrete Federations initiated development of a tool that calculate EPDs for their members' products. Ostfold Research was commissioned to develop this tool.

Specific environmental profiles (preferably EPDs) for the different materials as given in the recipe, are used as input data in the calculations of the concrete EPDs. Energy and transport are included based on data that are more general. The tool provides the EPD in the standard format required by The Norwegian EPD Foundation and in accordance with ISO 14025 and ISO 21930. A PCR in accordance to EN 15804 is also developed. The calculations and input data are verified in accordance with EPD-Norway's General Program Instruction 2014.

This tool gives producers the opportunity to disclose environmental information requested for tenders. In addition, the users gain a greater understanding of the environmental profiles of their products, which in turn creates a drive for sustainable innovation within the companies using the tools.

This paper describes how the EPD-generator works and how the concrete industry in Norway utilise the tool and its results.

Keywords: Environmental product declaration, EPD, generator, concrete, Norway

1. INTRODUCTION

Norwegian regulations for public procurement require that 'each procurement have regard to the resource implications and environmental consequences of the procurement' (The Ministry of Government Administration, Reform and Church Affairs, 1999). The law also requires that the life cycle costs of each purchase must be taken into account. The Norwegian green public procurement (GPP) performance has been assessed by Bouwer [1]. Norway was found to perform in the same class as the seven European 'front runners' in GPP (called the 'Green-7'). The public procurement regulations have been an important driver for this. However, these regulations do not specify which environmental aspects should be given priority. This requires that public authority purchasers have comprehensive competence.

Norwegian Green Building Council (NGBC) is a member-owned organization whose mission is to increase the sustainability of Norwegian buildings. They own and develop BREEAM-NOR, a method for classification of sustainable buildings. The classification system is voluntary, but it has proven popular in the market. This is a customer driven strategy to give better guidelines and increase the competence in building design phase, but it still needs objective and comparable information from the suppliers. Many of the more recent EPDs and construction products analyses have been performed in order to provide input to BREEAM-NOR projects.

In a supplier driven strategy, the suppliers provide the necessary environmental information. In the nineties the cement manufacturer Norcem, introduced life cycle assessments (LCA) as a method for decision support. Later, members of Norwegian Ready Mixed Concrete and Precast Concrete Associations adapted the method and started their own LCA-projects.

Life cycle assessments (LCA) address the environmental aspects and potential environmental impacts throughout the life cycle for a product or service, from raw material acquisition through production, use, end-of-life treatment, recycling and final disposal. Calculations are based on functional unit, describing the function of the product. In each phase of the life cycle and for each material or process involved, different environmental impacts are involved. LCA Methodology is standardised through ISO 14040 and 14044 [2], [3]. A schematic description of life cycle thinking is given in Figure 1.

Environmental declarations (EPD) present quantified environmental information for a product based on methodology for a LCA, to enable comparison between products fulfilling the same function. EPD Type III is a third party verified environmental declaration standardized in ISO 14025:2006 [4] and based on the methodology for a LCA. The ISO 14025:2006 is a general standard for all products and services. There is also a specific standard for EPDs for building materials, ISO 21930:2006 [5].

EPDs are used for calculation of environmental profile for a complex construction, but also for comparison between potential suppliers. General rules for comparability are given in the standard EN15804:2012 [6]. Comparison of the environmental performance of construction products using EPD information shall be based on the product's use in and its impacts on the building, and shall consider the complete life cycle. It is also possible to compare at the sub-building level. However, care must be taken that comparisons are made based on identical technical and functional performances. Clause 5.3 in the standard outlines, that the basis for comparative assessment in such cases is the entire building. Figure 2 is taken from the standard and shows the different modules during service life for a building.

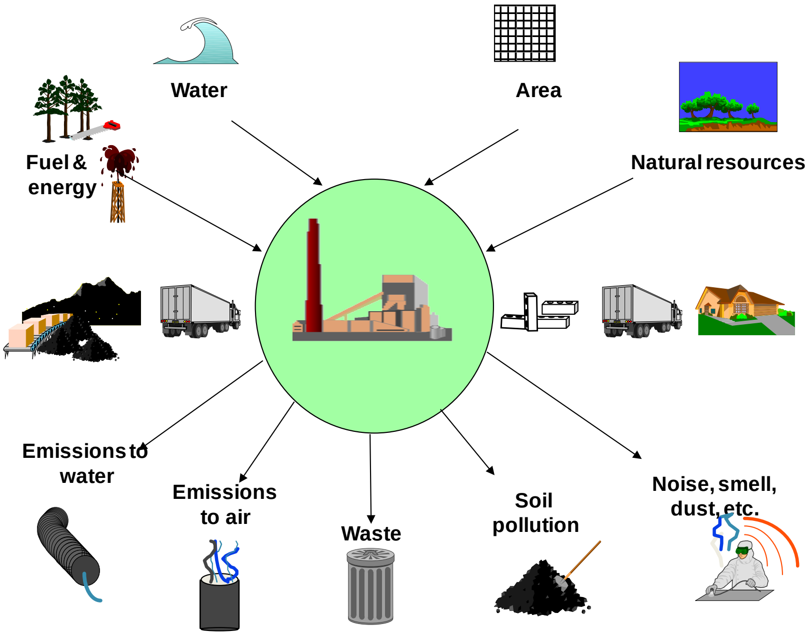


Figure 1: A life cycle for a building material

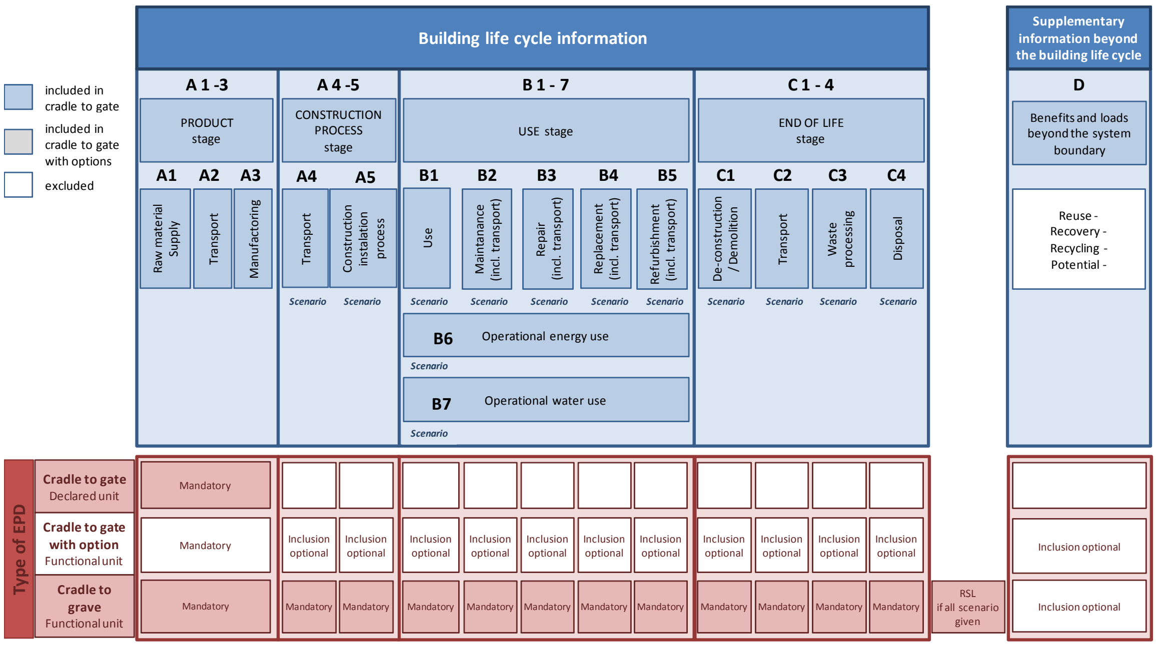


Figure 2: Types of EPDs with respect to life cycle stages and modules included in the building assessment (EN15804:2012)

A set of PCRs are developed [7], [8] to give guidelines for development of EPD for concrete products and to further specify the underlying requirements of the LCA. The core rules valid for all construction products are given in standard EN15804.

EPDs have to be understood by a broad audience and not only by LCA experts.. Norway has chosen to use a fixed format for the EPDs. Thus, EPDs are objective and comparable as they are developed using a standardized methodology and published in the same format. They are verified by a third party, which gives credibility in the market.

2. WHAT IS THE NEW TECHNOLOGY/PROCESS/PRODUCT?

During the first decade of the 2000's, Ostfold Research Foundation developed four different EPD-generators for the concrete industry. They cover "Ready mix concrete", "Concrete paving products", "Concrete construction products" and "Concrete infrastructure products". The earlier Norwegian tool was a Microsoft Excel worksheet application. Each producer could produce their own project specific EPD, based on data for their own production and life cycle data for the specific cements and other raw materials they use. This was a success and many EPDs were developed and used in the building sector.

Using Excel as the basis presented a number of challenges. These challenges were related to preserving the integrity of (and the ability to update) background data libraries, ensuring compatibility and usability across a varied range of computer platforms across the user base.
Moving to an interactive web-based solution alleviated most of these difficulties. In the new application, input from the user are data concerning the concrete recipe, transport of raw materials, their own production process, transport to building site and optional scenarios for construction phase, demolition and after life treatment [9], [10]. Figure 3 shows an illustration of the tool.

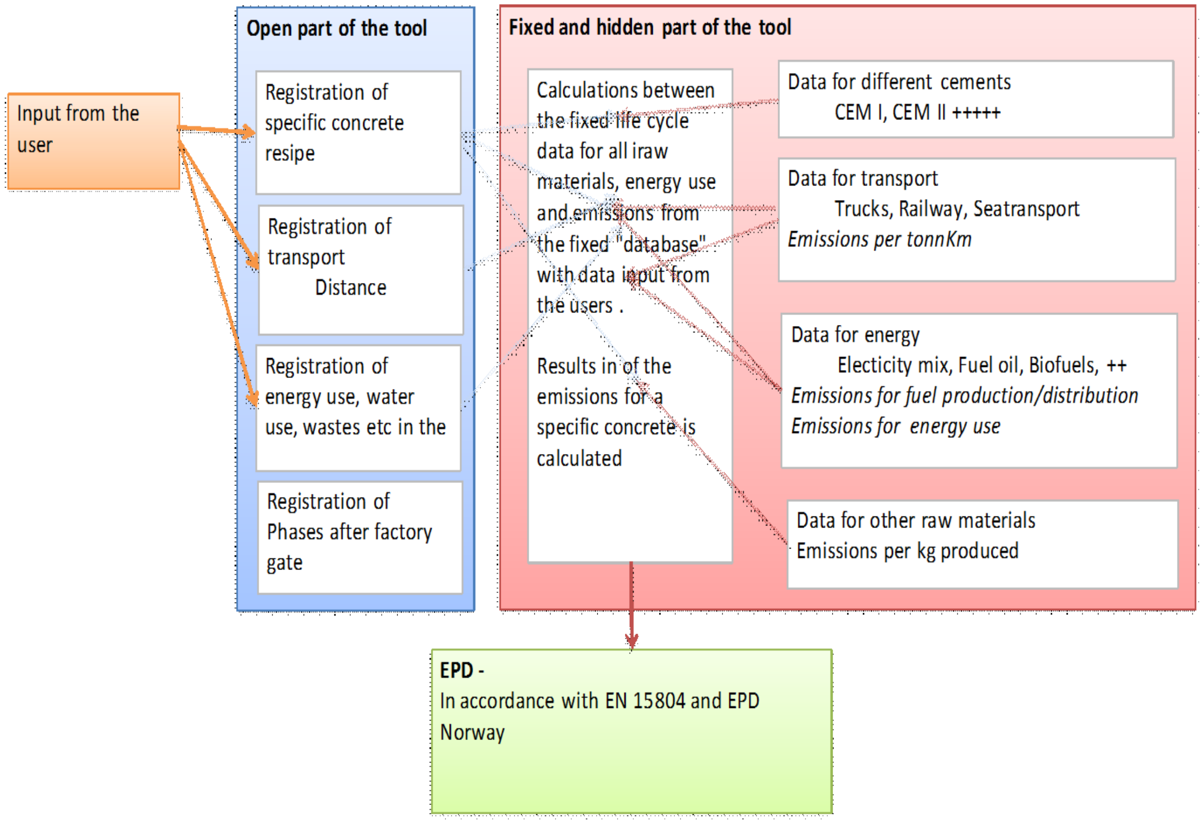


Figure 3: A sketch of the tool

The web server communicates with the EPD generator application/database that is centrally located on Ostfold Research's server. The data in the database is from Ostfold Research database which in turn is the result of LCA analysis performed using SimaPro. For the main raw materials, such as cement, the data are verified EPDs. For other commodities, are specific data from vendors also collected and recorded. Generic data from LCA database data (e.g. Ecoinvent) are modified for Norwegian conditions and used for fuel and electricity consumption, and production of raw materials which are of minor importance for the overall EPD .

The client-server configuration allows updates both to the application itself and to background data without the need for (re)distribution of code or data. Furthermore, user compatibility is more easily ensured simply by specifying the type and version of web browsers that are to be used with the tool.

2.1 What is the innovation and its relations to the areas addressed?

The EPD-generator gives the concrete producers the opportunity to provide environmental information requested for tenders. In addition, the users increase their understanding of the environmental profiles of their products, which in turn creates a drive for sustainable innovation within the companies using the generator.

This generator provides specific environmental profiles of the actual product being delivered a given project. This is because the input data in the database are actual environmental profiles (preferably EPDs) for all important commodities. In addition, environmental profiles for production and use of energy and transport are included. The database is customised for the concrete industry.

3. WHAT CHALLENGE DOES IT SOLVE?

The application enable the producers to respond to the clients requirements for EPDs immediately without using a lot of time and efforts. In addition, the provided EPDs can be project specific.

Norwegian concrete producers are typically small enterprises and often have limited resources dedicated to environmental affairs. They also produce many different products. The products are derived from common raw materials but in differing amounts. Life cycle assessments and EPD development were found to be both time and cost consuming. The existing LCA tools had a user interface that was designed for LCA experts, and they did not give the opportunity for specific calculations for each manufacturer or product.

The new EPD-generator has simplified the process and given the concrete manufacturers the opportunity to perform EPDs for their products themselves. Earlier was disclosure of EPDs to tenders only possible for large producers. Now are also small and medium sized producers in the position to present this information.

4. WHO NEEDS IT?

Environmental documentation will be and is a significant competitive argument for suppliers. Being able to provide this kind of information for a given product within a day, to a specific project will strengthen the competitiveness.

The EPD-generator is so far adapted to the concrete sector and some related industries. A configurable EPD generator will be of major importance for building material industry and other sectors. It is assumed that other industries would see the interest of adapting the generator for external environmental documentation.

Also industries outside the construction industry have shown interest in the tool, both nationally and internationally. The platform is material and industry independent. The database will have to be expanded if other industries or material groups want an application. Beyond this is the product usable to an unlimited group of producers and products.

The use of the tool for EPD generation in the concrete industry has shown that the tool gives valuable input to decision processes by enabling the producer to calculate the environmental impact for different recipes for their concrete products. It enables them to quantify the improvements of product development and shows the impact of choosing different input materials and energy use

REFERENCES

[1] Bouwer, M. (2006). Green Public Procurement in Europe. 05/06 Status overview. Report prepared for GRIP, Norway. Haarlem: Virage Milieu & Management.

[2] ISO 14040 – Environmental management – Life Cycle Assessment – Principles and Framework

[3] ISO 14044: 2006 – Environmental management – Life Cycle Assessment – Requirements and Guidelines

[4] ISO 21930: Sustainability in building construction - Environmental declaration of building products
[5] ISO 14025:2006Environmental labels and declarations –Type III environmental declarations – Principles and procedures,
[6] EN 15804:2012 Sustainability of construction works – Environmental Product Declarations – core rules for the product category of construction products.
[7] PCR 2005:7: www.environdec.com (2005): PRODUCT-CATEGORY RULES(PCR) for preparing an environmental product declaration (EPD) for Product Group “Concrete”, PCR 2005:7
[8] NPCR 2011:20: EPD-Norge.no (2012): PRODUCT-CATEGORY RULES (PCR) for preparing an environmental product declaration (EPD) for Product Group “Precast Concrete Products”, NPCR 20/11
[9] Vold, M. and Edvardsen, T. (2014): ”EPD-generator for betongindustrien, Bakgrunnsinformasjon”, OR.04.14, Østfoldforskning, Fredrikstad.
[10] Vold, M. and Edvardsen, T. (2014): ”EPD-generator for betongindustrien, Brukerveiledning”, OR.05.14, Østfoldforskning, Fredrikstad.

HOW CONCRETE SCORES IN THE MAIN ENVIRONMENTAL ASSESSMENT PROGRAMS

Kajsa Byfors (1) and Malin Löfsjögård (1)

(1) Swedish Concrete Federation

Abstract

Interest in environmental certifications has increased significantly since the first building in Sweden was certified as Green Building in 2005. Knowledge and facts about how concrete can help to meet the requirements of the systems have been compiled and published in four guides, one for each of the systems Green Building, Environmental Building Certification (Swedish system), BREEAM and LEED. The guides address to professions actively working in construction projects to be environmental certified. Some of the aspects where concrete can contribute to good performance are energy consumption, contents, recycling and waste, service life, flexible use and indoor environment.

In 2014 the Swedish Concrete Federation conducted a study of certified projects in Sweden showing that the choice of the system can be connected to the building type. The study also shows that more than 70% of all certified projects, wholly or partially, have a concrete frame. Many of them also received very high grades, confirming that concrete contributes positively too many criteria in the systems and can meet high demands from many environmental aspects.

Keywords: Environment, assessment, guide, concrete, certification

1. INTRODUCTION

Environmental certification of buildings in Sweden have greatly increased the last 2-3 years. That is positive because it helps to balance the many properties that affect a building's environmental impact. This makes governing for sustainable construction possible. But there is a lack of knowledge to translate the detailed requirements in the various certification systems to the election of sustainable building technology solutions. The Swedish concrete industry has therefore developed guidelines to describe how concrete can help to meet the requirements and thus contribute to the buildings sustainability.

2. CHOOSING SYSTEM

At an environmental certification a building's environmental impact is evaluated according to a predetermined method. In environmental certification systems, e.g. Miljöbyggad, BREEAM or LEED, many details are reviewed and then added to a combined result. Green Building is an energy certification scheme and differs from the other systems by assessing only energy performance.

The fact that environmental certification increased in Sweden are confirmed by the statistics reported by Swedish Green Building Council (SGBC).

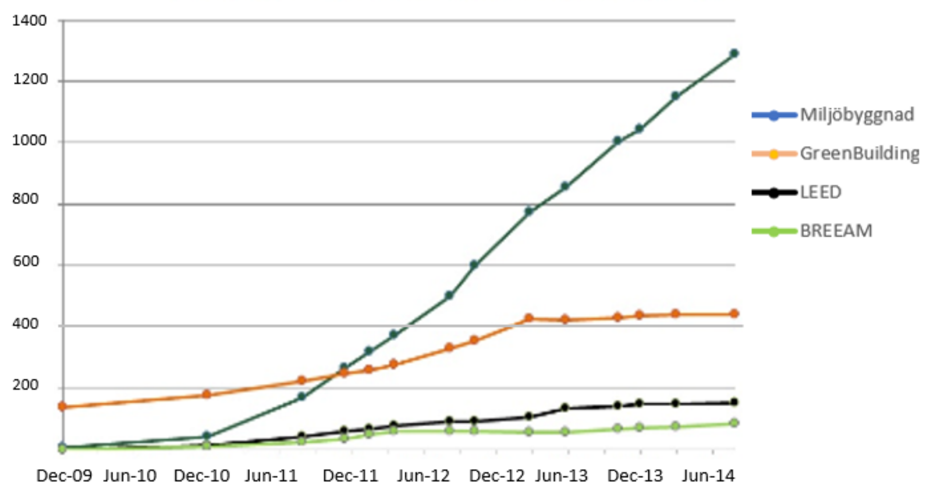


Figure 1: Number of registered and certified buildings in Sweden until august 2014, www. sgbc.se.

All the systems discussed here applies to housing. But there are also other systems such as BREEAM and LEED for cities and CEEQUAL for civil engineering structures. Which system is best suited for a particular type of building depends on several factors.

Generally, the European Green Building Certification is easiest to manage because it only takes into account the building's energy use. “Miljöbyggnad” (Green Building in English) is a Swedish system and assesses various environmental aspects adapted to Swedish condition, for example indoor environment and materials. In comparison to many other systems this system is simple and accessible and the initial cost is a little lower. Most requirements are in British BREEAM and American LEED thus also provide an opportunity to include more of the building's good qualities such as building and site design and how it interacts with its environment. They also take into account issues such as storm water design, heat island and transports. BREEAM is also available in a Swedish version, BREEAM SE, which is partly adapted to Swedish regulations and practices, while LEED is based on American rules but adapted more and more to international standards. LEED and BREEAM are more interesting if one wants that certification should be communicated in an international market. That means that the certification process will be a bit more extensive and start-up cost of the first project will be higher. But as these systems become more common, project teams gain experience and costs decreases.

Figure 2: Left: Torsplan in Stockholm is a concrete/steel building and the first commercial property in the Nordic countries that have achieved BREEAM Excellent. Illustration: Diakrit. Right: Väla Gård in Helsingborg is certified LEED Platinum, and has reached one of the highest grades in the world. Concrete secured airtightness in a simple way, which has a major impact on energy consumption. To reduce the climate impact from cement a concrete with fly ash was used. Photo: Emma Westfelt.

3. NATURAL, ROBUST AND ENERGY-SAVING FOR AT LEAST 100 YEARS

Concrete is today’s most common used construction material. The concrete strength and durability make it an excellent material to use in foundations as well as building structures and facades. The properties of concrete are very well documented, both by experience from long-term use and through basic and systematic research. Knowledge and facts about how concrete can contribute to meet the requirements of environmental certification systems have been compiled in a Swedish project and published in four guidance documents, available at www.betongforeningen.se. These guides are written for different professions who are actively working in new construction projects to be environmental certified; owners, architects, technical consultants, contractors, materials manufacturers, environmental experts, etc.

Since the different environmental certification systems differ in environmental aspects evaluated, performance levels and ways to measure and report, it is not possible to treat all aspects and requirements here - all such details are reported in more detail in the guidance documents. Here some aspects have been selected that are considered in several systems and where the concrete has significant influence and which may serve as examples of how concrete can contribute to the profit of an environmental certification.

3.1 Energy savings

Although the requirements are different, energy is a significant part in all four environmental certification schemes Green Building, Miljöbyggnad, BREEAM and LEED. In Green Building the building's energy use must be at least 25% lower than current building regulations requirements. In Miljöbyggnad and BREEAM credits are given depending on energy use relative to the regulations while LEED requires energy cost compared to a reference building. The reference building reflects the minimum requirements of American standards (ASHRAE). The credits in Miljöbyggnad, BREEAM and LEED are also affected by the choice of energy source.

Both LEED and BREEAM reward measures to minimize the impact of refrigerants in order to reduce the climate impact of refrigerants.

In all systems, it is possible to achieve high scores with concrete. With concrete the building gets a heavy heat storing structure. This means that the house gets a built-in capacity to store excess heat (or cool) that can be used when the temperature goes down. This reduces the total energy demand. The concrete's thermal properties reduces and moves power peaks in time which is beneficial both environmentally and economically. In practice this means that a concrete house does not need to be heated as much, and not at the same time as other houses. In the same way these thermal properties can be used to reduce the need of energy for cooling at hot temperatures, such as in offices. Cooling of buildings is very energy intensive and here the thermal properties of the concrete provide a significant benefit.

3.2 Content, recycling and waste

Concrete is 100 percent recyclable. It comes from limestone, rock and stone and usually returns as filler material when life is over. And concrete does not contain any substances hazardous to health or environment.

In BREEAM recovery and reuse is rewarded in several different ways. Here is an opportunity to earn credits for Responsible sourcing of materials. A concrete building has good credit opportunities as it contains relatively few materials and all are traceable. This is important as concrete represents a large part of the material used in the building. BREEAM also encourage the use of recycled aggregates in order to limit the use of virgin materials. A similar requirement exists in LEED, Recycled content. Here, concrete help to score by various waste materials, such as when fly ash and slag replaces certain part of cement and filler in concrete, something that is common today. Crushed concrete can also be recovered end partially replace new aggregates in concrete.

Waste management at the job site is addressed in both BREEAM and LEED in section Construction waste management. Both systems reward measures for reducing the amount of construction and demolition waste at the workplace, e.g. through recycling. Here, concrete contribute to credits because the amount of waste concrete is very small. Precast concrete is ready to install when delivered to the construction site and any waste or leftover of ready mixed concrete could easily be taken back to the plant. If concrete waste would occur, for example from demolition of end of life buildings, this can either be recycled as filler or as aggregate in new concrete.

In LEED section Regional materials, credits is given for locally produced building materials. The aim is to restrict transports. Since concrete and the most of the raw materials are produced locally, concrete can contribute to this score.

Phasing out hazardous substances score in the systems Miljöbyggnad, LEED and the Swedish version of BREEAM. Most of the systems follow the REACH legislation. There are no hazardous substances associated with hardened concrete.

3.3 Service life and flexibility

Buildings of concrete usually have a very long service life, at least 100 years, but requires a minimum of maintenance. The limit of the lifetime of a concrete structure is not really set by the material but rather building design based on user needs. And that's another good concrete quality because it allows for buildings with large spans and thus great flexibility in disposition of the premises.

BREEAM treat the lifecycle costs – LCC - with the aim to improve the design, system selection, and operation and maintenance throughout the life cycle. It's evaluated for 30 and 60 years life time and scores when the solution that results in the lowest life cycle cost is selected. As a properly designed concrete structure has low operating and maintenance costs, and lasts much longer than 60 years it contributes to low costs and can contribute to several points in this aspect. A longer time period in the calculation would give even better result for a concrete building.

BREEAM also considers Designing for Robustness in order to minimize replacing of material due to wear and damage. Concrete is a material with high strength and wear resistance, and that does not change with time. This means there are opportunities to award credits.

Reuse of entire buildings or building parts is considered in BREEAMs Re-use of facade and re-use of structure and LEEDs Building reuse – maintain existing walls, floors and roofs. Reuse of entire buildings is positive in several ways: the service life of the existing building increases, resource consumption decreases, cultural values are preserved and construction waste reduced. These points are awarded for rebuilding and additions and depend on what part of the building that is recycled. As concrete is very resistant and have good load carrying capacity thus gives excellent prospects for reuse. Many concrete buildings also has an unused load capacity to handle the additional loads. Combined with large spans it provides great flexibility to dispose the premises. Today there are also methods for structural strengthening and additional insulation that makes it possible to meet new demands on performance.

3.4 Indoor environment

Concrete is moisture-proof - a property which comprises during the building's entire life time. The risk of mold formation in a concrete structure due to moisture is insignificant, partly because concrete has a high alkalinity, and partly because organic materials are present in very small quantities.

Moisture is treated in both Miljöbyggnad and BREEAM SE, with the aim of reducing the risk of future moisture and water damage in the building. To score in the systems, action should be taken to prevent moisture problems. The possibility with concrete is very good. It is about ensuring that the concrete is properly desiccated and avoid solutions where moisture sensitive materials are in direct contact with concrete. Today there are methods and tools to manage this.

Miljöbyggnad and BREEAM SE consider acoustic performance. The rating is determined by the buildings noise reducing capacity according to Swedish standards. Achieving good

acoustic performance with concrete is normally not a problem and accommodation in concrete buildings usually experience the acoustic environment as very good.
Thermal comfort is treated in Miljöbyggndad, LEED and BREEAM, reflecting the perceived temperature and climate in a building. In order to meet the requirements in the systems, the thermal climate are calculated and presented in accordance with specified methods, in some cases also confirmed by the residents. A building's thermal climate is due to a combination of many factors where the structural material properties are of great importance. The concrete opportunities to create a tight building shell and the ability to store and release heat contributes positively to a building's thermal climate.

4. CONCRETE DOMINATING IN ENVIRONMENTAL CERTIFIED BUILDINGS

The Swedish Concrete Federation has initiated a survey of all new certified buildings in Sweden. The study is based on buildings included on the official lists that are published for each system until spring 2014. Of a total of 157 projects, facts about structural materials was obtained for 129 projects, which means 82% of the total number of certified projects. The survey shows that more than 70% of certified projects are concrete or steel/concrete structures.

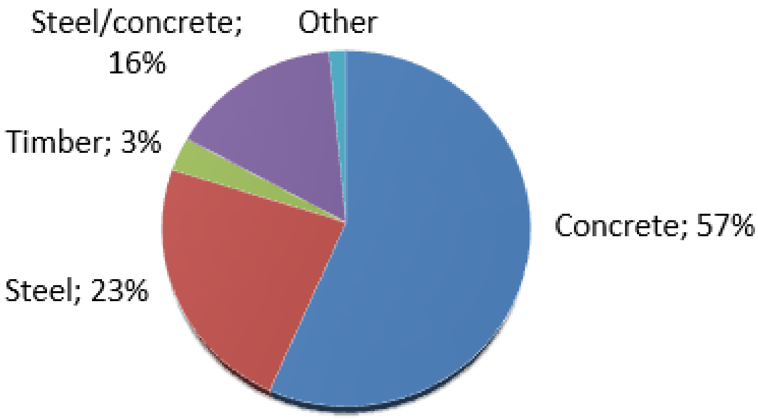


Figure 3: The use of structural materials in housing in Sweden.

This allows the concrete to be the most common material in environmental certified buildings in Sweden. It can be explained by the concrete's many good qualities and potential, for instance in terms of energy requirements, indoor climate and acoustics. In addition it was clear that most of the buildings that received the highest score in the certifications were concrete structures. It confirms that the concrete contributes positively too many criteria in the systems and that the material can meet high demands from many environmental aspects. Examples of Swedish buildings that have achieved very high ratings are office buildings “Väla Gård” in Helsingborg certified as LEED Platinum and “Torsplan” in Stockholm who achieved BREEAM Excellent.

REFERENCES

[1] SGBC: http://www.sgbc.se
[2] Swedish Concrete Association (2013): Sustainable building with concrete. Guides to environmental certification BREEAM, LEED, Green Building, respectively. Green Building (in Swedish). www.betongforeningen.se
[3] Öhrling, P., Engqvist, L., Jonestrand, S., Rönneblad, A., Byfors, K., Larsson, R., Peterson, M., During, O., " Möjligheter med betong vid miljöcertifiering (in swedish)", Bygg & Teknik 7/13, 2013, 44-48
[4] Rönneblad, A., Byfors, K. Trög betongstomme minskar energianvändningen och möjliggör fördröjning av effektuttag (in swedish). Bygg & Teknik 7/12, 2012, 53-56
[5] Westfelt, E., Byfors, K., "Betongstomme vanligast i miljöcertifierade byggnader" (in swedish), Bygg & Teknik 7/14. 2014, 72-74

SUSTAINABILITY AND GREEN TRENDS IN CONCRETE VERSUS COMMON SENSE

Michal Števula

Secretary of Concrete Producers Association of the Czech Republic

Abstract

Sustainability, green and energy policies are repetitively mentioned names of recent philosophical ways in Europe. All those trends have their own meanings. There is an effort at many levels to be more sustainable and greener. Unfortunately, industry has to live under the combination of all the above mentioned trends and from its point of view, sustainability equals the term survival. The competition at our European market is not only among us, the Europeans, but involves also the rest of the world. This means: our market is merciless. In this situation it is time for some questions: will concrete be greener with another "green" certificate? Will concrete be more sustainable with another certificate of sustainability or are we just spending time playing paper games? It is clear that it is complicated to find balance between sustainability, green policy and taxes on one side and production on the other. Maybe it is high time, as history teaches us, to use common sense.

Keywords: Sustainability, certification, energy, costs, surviving of concrete industry.

1. INTRODUCTION

This paper is not a marketing or a strategy study, it is not a legislation overview, not even a scientific research. What it could be is the view of a “common man”, let’s assume a wise man, who lives on the border between two worlds: the world of legislation, i.e. to a big extent a virtual world, and the world of economics, i.e. the real world. This man could be also a concrete producer.

2. WHAT IS SUSTAINABLE DEVELOPMENT?

In literature we can find numbers of definitions of what sustainable or sustainability mean, like these:

Sustainable is something:

- able to be used without being completely used up or destroyed;
- involving methods that do not completely use up or destroy natural resources [1].

I completely agree with these definitions. Let’s see, if and how this so well-defined concept is reflected in praxis.

3. SUSTAINABILITY CERTIFICATE FOR CONCRETE – YES OR NO?

Everything can be viewed from different angles. I just want to pick up some of them.

3.1 Certificate, papers – current state

I will describe the situation in the Czech Republic; you may of course find differences in other countries. In the Czech Republic, there is a standard for concrete production, ČSN EN 206 [2]. At the same time, it is necessary to respect the Czech legislation stating that certification is compulsory [3] [4] [5]. This means that a concrete producer must have a certified production quality control. But this in not enough: if the producer wants to bid for state tenders, it is mandatory for him to have ISO 9001 [6] and OHSAS 18001 [7] certification, and often ISO 14001 [8] also.

On top of this, different areas – but not the market – start asking for certification of concrete as a sustainable material.

3.2 Certificates and papers – the past

Due to the combination of European and Czech legislation, in the Czech Republic between 2002 and 2005 there was such a situation where every concrete at every concrete plant had to be certified three times: according to the old Czech standard (ČSN 73 2400), according to the preliminary European standard (ČSN P ENV 206) and according to the at that time new European standard (ČSN EN 206-1). The same concrete, three certificates…. three invoices!!!

3.3 Communication point of view

There are different opinions on how to assess sustainability of materials. Among these, Life Cycle Assessment (LCA) is recognized as a complex and detailed method. It is also a method which for its extent does not suit very much to communicate with the general public. How then tell the layman, that concrete is sustainable? A certificate is an easy and comprehensible evidence of sustainability of concrete.

3.4 Technical standard point of view (EN 206).

EN206 Annex F, paragraph (2) states: the values in Table F.1 are based on the assumption of an intended design working life of the structure of 50 years. In the CR we have been preparing a technical standard where there will be a Table F.2 for working life of the special structures up to 100 years. Are there really doubts about the sustainability of a material which is, according to the technical standard, designed for lasting 50 or even 100 years? (fig. 1).

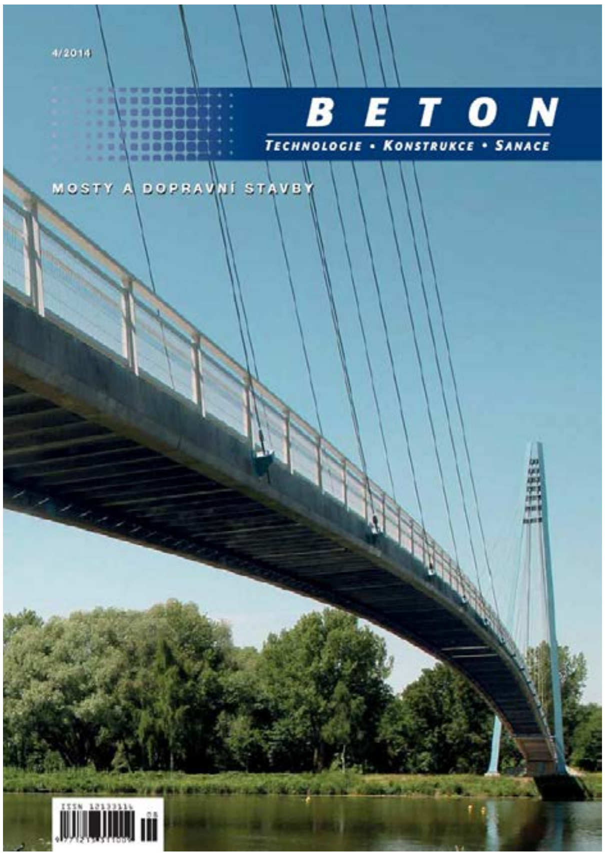


Figure 1: Example of sustainable structure footbridge in Čelákovice, near Prague, UHPC C110/130 concrete - Beton TKS magazine, issue 4/2014. (Foto J. Husák. Metrostav).

3.5 BMI view

BMI, the well-known “Body Mass Index”, could also be a point of view on sustainability and also on sustainability of concrete. Sustainability is often presented as a social demand. Based on the WHO data [9], 51.7% of adult population in the Czech Republic suffers from overweight. In the CR - and it will not be much different in other EU countries – more than one half of adults disrespects the sustainability rules if they have to apply them to themselves. Overweight and obesity do not belong to sustainability. The potential to improve sustainability of population overweight is much bigger than the potential to improve sustainability of concrete.

3.6 Jacket point of view

Most of us have at least one jacket. This jacket has a very specific colour. We have chosen the colour most probably because we liked it. Let’s say that the colour is black. Did you ask the shop assistant for a certificate stating that the jacket is really black? No. Why? Since, it is black. Do you produce concrete that is sustainable? Yes. So why do you need a certificate, if we do not need it for other commodities?

3.7 Czech market point of view

Members of the Concrete Producers of the Czech Republic produced in 2014 5.083.000 m^3 of concrete, 0,5 m^3 per capita (the CR has approx. 10,5 Mio inhabitants). This is about 75 % of overall ready mixed concrete production in the CR. I asked all chief technologists at one of their meetings: “How many cubic meters were requested either sustainable or green in 2014?” The answer was: „zero“. I posed the next question: “What do your customers want?” The answer was: “Three things: low price, lower price, the lowest price” - provided that all the technical parameters in the concrete specifications are satisfied.

3.8 Global market point of view

From all the above it is clear that costs arise for concrete producers from these obligations. Every tax or cost which an European concrete producer must pay may be considered as a discount or incentive for non-European competitors. By increasing our costs we move vessels with some constituent materials made out of Europe closer to our regions.

3.9 My personal point of view

I have been working for the Concrete Producers Association of the Czech Republic for 15 years. I would like to emphasize the “concrete producers”, not buyers of papers with stamps. I would very much appreciate if we could further on focus on production of quality, - i.e. sustainable - concretes, and not on filling in forms!

For me, sustainability is using common sense.

4. CONCLUSIONS

- Sustainability is a very wise strategy - it shouldn’t be mistaken for bureaucracy and paperwork.
- Concrete, and specifically a concrete construction, being designed for 50+ years “de facto” conforms with the sustainability principles.
- A sustainability pass for concrete should be an issue for scientific grant assignments, published in specialized media.
- Buildings, bridges and other structures are made of concrete, steel, wood or masonry - never of certificates.
- Let’s concentrate on production of quality concrete instead of hunting papers with stamps that prove the proven.

REFERENCES

[1] An Encyclopædia Britannica Company, Merriam-Webster dictionary, http://www.merriam-webster.com/dictionary/sustainable.

[2] ČSN EN 206 Concrete - Specification, performance, production and conformity, ÚNMZ, Praha, 1.7.2014.

[3] Zákon č. 22/1997 Sb., o technických požadavcích na výrobky a o změně a doplnění některých zákonů ve smyslu pozdějších předpisů.

[4] NV č.163/2002 Sb. kterým se stanoví technické požadavky na vybrané stavební výrobky, Praha 2002.
[5] NV č. 312/2005 Sb., kterým se stanoví technické požadavky na vybrané stavební výrobky, Praha, 2005.
[6] ČSN EN ISO 9001 ed. 2, Quality management systems – Requirements, ÚNMZ, Praha, 1.10.2010.
[7] ČSN OHSAS 18001 Occupational health and safety management systems – Requirements, ÚNMZ, Praha, 1.3.2008.
[8] ČSN EN ISO 14001 Environmental management systems - Requirements with guidance for use, ÚNMZ, Praha, 1.6.2005.
[9] WHO, World Health Organization, http://apps.who.int/bmi/.

ENVIRONMENTAL COSTS INDICATOR FOR CONCRETE

Leo J.G. Dekker

MSc, Mebin B.V. - HeidelbergCement Group, The Netherlands

Abstract

In The Netherlands there is an intense discussion on the environmental impact of the built environment. The main focus of this discussion is aimed towards CO_2 reduction and re-use. The Dutch Building Act requires documents on the environmental performance (CO_2 footprint and depletion of raw materials) to get a building permit for all buildings with a net available surface of 100 m^2 or more. Concrete clearly plays an important role in construction and therefore it is important to have a calculation tool to calculate the full environmental effects from cradle-to-grave. In 2014 such a tool became available (the so-called SBRCUR tool). This tool uses verified data from several sources and the Dutch National Environmental LCA Database to calculate 11 environmental LCA-aspects, amongst them CO_2 or global warming. Another calculated parameter is MKI – translated in English the so-called Environmental Costs Indicator (ECI). The ECI (€/m^3, €/project) gives the theoretical fictional costs to compensate all environmental effects based on present knowledge and assumptions. In this paper the SBRCUR tool is used to show the difference in environmental effects for two common building techniques based on ready mixed concrete for residential housing. The calculations show significant differences in CO_2 footprint and ECI between the two building concepts.

Keywords: Sustainable concrete, CO_2 footprint, environmental costs indicator, mix-design, ready mixed concrete

1. INTRODUCTION

Concrete mixes are usually designed based on known calculations. Often the well-known Abrams formula is used for that. In that formula, the concrete compressive strength is a function of the water-cement ratio, the standard compressive strength of the cement and some constants. Eventually, we will arrive at a composition on the basis of a mix design calculation that meets the required performance, such as strength class, the exposure class, and workability. In The Netherlands, in addition to these performance standards, there is an increased attention for the impact that concrete has on environmental aspects. The most important factor is the so-called CO_2 equivalent footprint. CO_2, or carbon dioxide, caused by human activity is partly responsible for a further global warming due to the greenhouse effect. The use of concrete plays a significant role in that. Moreover, the cement in concrete is globally responsible for about 5% of the CO_2 produced by human activities [1,2]. In the Netherlands this is considerably less as a result of the use of low clinker cements (CEM II, CEM III, CEM V) and the use of non-fossil fuels for the production of clinker (biomass).The contribution of cement to the greenhouse effect in the Netherlands is approximately 1%. The average CO_2-equivalent of the Dutch cement producers is 450 kg CO_2/t. The Netherlands is in this case, without any doubt, a frontrunner.

But we are not only dealing with CO_2. There are more environmental aspects to be considered for the application of concrete. This does not apply for concrete alone, but for all building materials. This is based on EN 15804. This EN 15804 refers to a LCA methodology to identify all environmental impacts of a product or process unit (m^3, t). This standard provides 24 environmental aspects. The Dutch standard (NEN-EN 15804: 2012) uses “only” 11. The LCA per unit is called MRPI (Milieu Relevante Product Informatie; environmentally relevant product information) or EPD (Environmental Product Declaration). The MRPI does not use 24 environmental aspects but uses the 11 most important (table 1):

Table 1: 11 environmental aspects used in The Netherlands

Environmental effect	Equivalent unit	Fictional price (€/kg Eq. unit)
Abiotic depletion, minerals	Sb eq	0.16
Abiotic depletion, raw materials	Sb eq	0.16
Greenhouse effect (CO_2)	CO_2 eq	0.05
Stratospheric ozone depletion	CFK-11 eq	30
Acidification	SO2 eq	4
Eutrophication	PO4 eq	9
Human toxicity	1,4-DCB eq	0.09
Eco toxicity, aquatic, water	1,4-DCB eq	0.03
Eco toxicity, marine water	1,4-DCB eq	0.0001
Eco toxicity, terrestrial	1,4-DCB eq	0.06

Many of these environmental aspects are fairly complex and are expressed in units that can only be understood by experts in this field. Also, the calculations are complex. Moreover, there is not always sufficiently reliable information available to carry out reliable calculations.

2. LCA TOOL

Recently a solution is available in The Netherlands. SGS Intron BV was commissioned by SBRCURnet to develop and design an Excel based tool for “Green Concrete”. With this tool it is possible to calculate the environmental aspects of concrete mixes and further processing on a relatively simple manner. This tool takes into account: the amount of raw materials used, transportation to the factory, production, transportation of ready mixed concrete or prefabricated elements to the construction site, the construction phase and the demolition phase. The data used by the tool comes from several public databases like: the Dutch Cement and Concrete Centre, the Dutch national environmental database, validated MRPI certificates and the international Ecoinvent database. The result of a calculation is a table that displays the 11 environmental aspects, completed with a graphic representation.

To compare several alternatives the tool introduces a new expression, the environmental costs indicator (ECI). In this ECI all the aforementioned environmental aspects, including CO_2, are 'capitalized'. Each environmental aspect represents a fictional unit amount of environmental impact that can be calculated in Euros. If we add all individual environmental aspects in Euros we get an environmental profile consisting of one single amount of money (Euros). This represent the amount of money that would be required to actually compensate these environmental aspects (see Table 1). The higher the amount, the more damaging the product in terms of environment. In the example in this paper, version 3.2 was used for calculations.

2.1 Example: residential concrete building system, with and without external heating

To demonstrate the possibilities of this tool an example using in situ concrete for a residential concrete building system, with and without external heating is considered. The technique involves a one day cycle for erecting formwork, placing reinforcement and electricity pipes, casting and remove formwork. To make this possible the concrete must have a compressive strength of at least 14 MPa after 16 hours. This is achieved by either using a concrete with predominantly CEM III/B and external gas heaters (warm technique) or using a concrete with predominantly a CEM I cement (cold technique).

Figure 1: site for building residential houses with heating facilities (source: VOBN)

The following example of the cold and warm technique is based on a situation with an average 24h temperature of 9^0C. The mix compositions used in the calculation with the LCA tool are shown in Table 2.

Table 2: mix compositions in kg/ m^3

	Warm technique	Cold technique
CEM III	250	
CEM I	75	380
Sand (river)	802	750
Gravel (river)	1053	1033
Water	164	178
Superplasticizer	0,52	1,52

The tool also needs information relating to the circumstances of the production and the application on the construction site. This, of course, can be endlessly varied, but for this example the following parameters were used:

- For the transport of the raw materials from the extraction site to the production plant, default values were used. These are national averages that are included in the database of the tool.
- Transport distance of 20 km with an average truck mixer (single trip).
- Gas consumption RMC plant: 0.25 m^3/ m^3
- Electricity use concrete plant: 3.9 kWh/ m^3 (green power)
- Diesel consumption internal transport: 0.2 l/ m^3 (bobcats)
- Gas heaters in case of the warm technique (propane gas)

The results of the LCA calculation are presented in table 3, figure 2 and 3.

Table 3: CO_2 (kg/ m^3) en ECI (€/m^3) for mixes in Table 1

	Warm technique	Cold technique
Raw materials	139	315
Transport raw material	26,5	30,2
Production concrete	3,4	3,4
Transport concrete	4,5	4,5
Building site (propane gas)	27,1	0,0
Demolition phase	5,5	5,5
Total CO_2	206	359
ECI	€ 17,69	€ 28,94

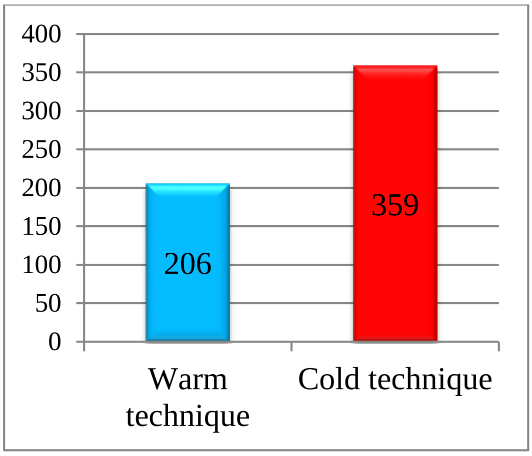


Figure 2: CO_2 (kg/m^3)

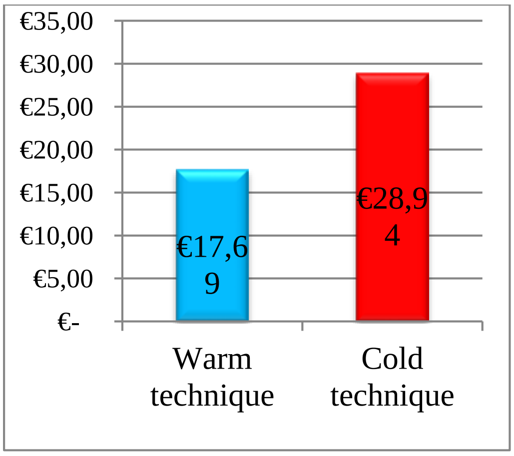


Figure 3: ECI (€/m^3)

3. DISCUSSION

3.1 Influence of raw materials

The difference is obvious: the warm technique scored significantly better in terms of CO_2 footprint and ECI. So the warm technique is the most environmental friendly alternative. This is to a large extent caused by the fact that predominantly a low clinker cement is used, in this case a CEM III. It should be noted that this can be further improved by using 100% CEM III. This is possible if the average day temperature rises (> 12ºC). On the other hand, the LCA tool uses an average value for CEM III in the Netherlands. Calculating with the actual environmental values for CEM III/B cements with a slag content of 70% makes an even more realistic and favorable image.

All separate contributions to the CO_2 footprint are shown in figures 4 and 5. It can be seen that there are two other major components that determine the CO_2 footprint. The heating of concrete on the building site in the case of warm-forming and the transportation of the raw materials.

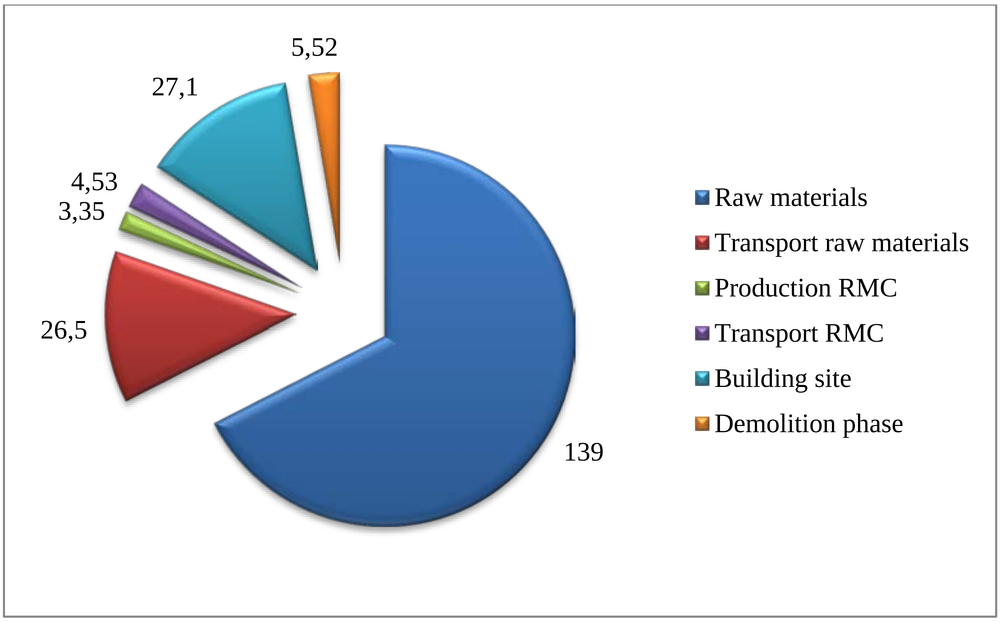


Figure 4: separate contributions for warm technique (kg CO_2/m^3)

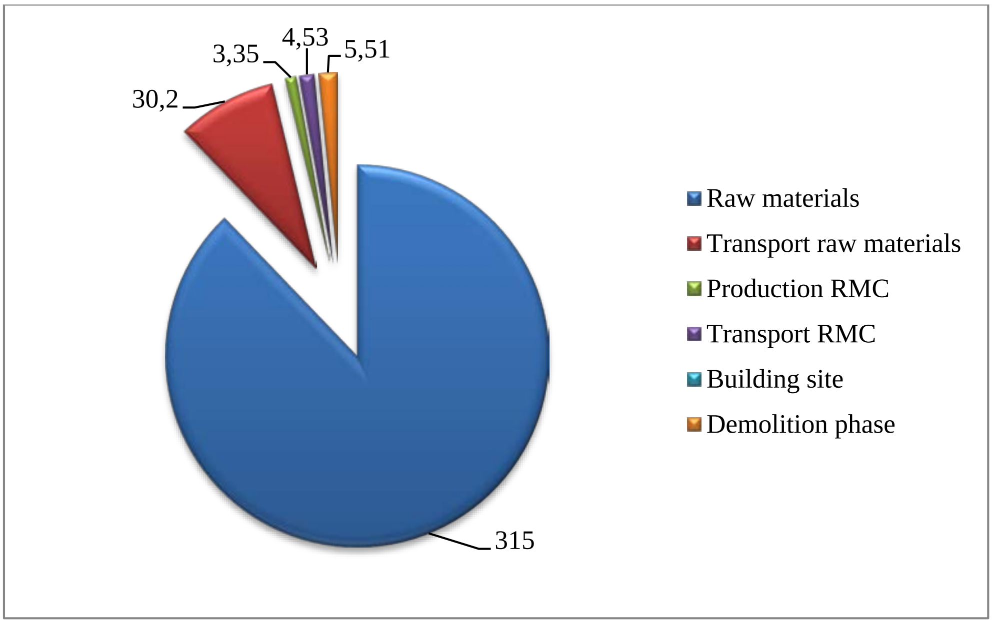


Figure 5: separate contributions for cold technique (kg CO_2/m^3)

3.2 Influence of heat addition (warm technique)

The CO_2 for heating concrete in the formwork is derived from the combustion of propane gas. Depending on the ambient temperature, this value may be higher or lower. The aim is, after all, to be able to remove the formwork after 16 hours at a compressive strength of 14 MPa. CO_2 emissions can also be reduced by simply waiting longer to remove the formwork. In most cases there is no time for that in a tight construction schedule. This is, of course, taken into account for the weekend. There is more time to reach the strength of 14 MPa. Mix compositions, and heating conditions are adjusted in this case. A 'slow concrete' is good enough in this case. **The most sustainable concrete is always poured on a Friday.**

3.3 Influence of transport

A third important contribution to the CO_2 footprint is the transport of raw materials. In the Netherlands this is predominantly by ship. As previously stated the calculation was made with default distances. These distances are from a national database of SKB and are averages for the Dutch market. It could very well be that, when a concrete plant is located near a cement plant or a sand/gravel extraction site, significantly lower values are found. For this factor you can find big differences, but it does show that is it worth to maximize the use of locally sourced materials.

4. CONCLUSION

There is more to calculate with concrete, than we are used to in daily life. Concrete producers will increasingly be held accountable for the environmental aspects of concrete in BREEAM or LEED certification systems. With the LCA tool use it is quite easy to calculate complex environmental issues in a simple way and make it visible. The example of a warm and cold technique for residential concrete building is just one example of many cases that could have been worked out. It invites us to work out further questions to compare mix design and working methods. What is the effect of recycled concrete? What is the effect of concrete with fly-ashes? What is the effect of ready mixed concrete compared to prefabricated concrete? The challenge is there!

REFERENCES

[1] Duurzaam beton – Trending topics, Betonplatform, 1994, ISBN 978-90-71806-00-1
[2] Cement, beton en CO_2, feiten en trends – Cement&BetonCentrum

PRODUCT TRANSPARENCY: NORTH AMERICAN EXPERIENCES TOWARDS EPDS, CSRS AND HPDS FOR CONCRETE

Tien Peng

National Ready Mixed Concrete Association, USA

Abstract

The trend towards product transparency has reached the green building arena in North America. The new LEED v4 rating systems and other standards such as the International Green Construction Code (IgCC) and the Architecture 2030 Challenge for Products have new reporting requirements for product manufacturers. Environmental Product Declarations (EPDs), Corporate Social Responsibility Reports (CSRs) and Health Product Declarations (HPDs) are new reporting requirements in the LEED v4 that are changing the building products industry, including concrete. A growing number of architecture and engineering firms have established formal policies requesting these reports before inclusion into their firms' product library and project specifications.

The National Ready Mixed Concrete Association (NRMCA) has developed several programs to help its members meet these new transparency requirements for sustainability reporting. NRMCA is an EPD Program Operator and has developed an industry-wide average EPD and establishing baselines for concrete producers. In addition, NRMCA is worked with the Cement Sustainability Initiative to develop a Responsible Sourcing Certification System that will result in CSR for concrete producers. NRMCA has also developed a guide to help concrete producers and suppliers meet the new HPD reporting system.

1. INTRODUCTION

The construction industry has a major effect on sustainable development. Not only does it have some of the biggest direct effects on water, resources, land use, and greenhouse gas emissions and indirect effects on the environment by affecting transport systems, but it also affects communities' economy and even public health. Of course, construction is also a major contributor to the overall growth in our economy. Consequently, consumers and specifiers are demanding more responsibility from the materials used in the construction of our buildings and cities.

Concrete companies are now competing more hotly for specifiers' attention when it comes to matters such as green building materials. In the past, the focus of material impacts has been on single attributes such as recycled content, rapid renewability, or regional materials. While these attributes are important, architects and engineers are beginning to understand that they only tell part of the story. A product could perform well in one attribute but poorly in another. As a result, a more holistic look at materials and their environmental impacts is starting to take hold. Moreover, with the rise in social media channels, corporate responsibility is becoming too important a concern for manufacturers to depend the media or NGOs to uncover for improvement. The public is getting smarter, and concrete businesses in the growing green building marketplace need to proactively provide assurances that materials have reduced their environmental impacts, are responsibly sourced and have minimized hazardous ingredients.

2. LEED DRIVES TRANSPARENCY

Since its inception, the LEED (Leadership in Energy and Environmental Design) green building rating system has been used to reduce negative environmental and human impacts of the built environment. LEED has been a market transformation device affecting all sectors of the construction industry, including concrete production and construction. The system is credit-based, allowing projects to earn points for environmentally friendly strategies employed during the design and construction process.

With each new version, the US Green Building Council (USGBC), developers of LEED, has increased the targets and scope for reducing environmental impacts related to building design, construction and maintenance. LEED v4, released in November 2013 (USGBC, 2013), continues the momentum with a number of advancements that will likely change the way design professionals, contractors and product manufacturers do business. Many credits, such as Stormwater Management, Heat Island Reduction and Optimized Energy Performance, are refined but maintain the same intent. Others, such as Material and Resource (MR) credits, challenge concrete manufacturers to disclose their environmental, social and health impacts in third party validated reports.

3. TRANSPARENCY THROUGH ENVIRONMENTAL PRODUCT DECLARATIONS

Environmental Product Declarations (EPDs), also known as Type III environmental declarations, present quantified environmental information on the life cycle of a product to enable comparisons between products fulfilling the same function. Widely used in Europe, EPDs have been maturing since the Environment Directorate General of the European Commission commissioned a study by Environmental Resources Management to examine

Environmental Product Declaration schemes in 2001. Now many countries have national EPD programs:

- French Agency on Environment and Energy Management (ADEME) National Experimentation for the Environmental Display on Products
- British Standards Institute
- German Institute of Construction and Environment (IBU)
- Swedish Environmental Management Council (SEMCo)
- The Norwegian EPD Foundation

This is not the case for the US market. While regulators demanded disclosure for automobile fuel efficiencies and food nutrition, very few construction products had to share their environmental impacts and no national EPD Program emerged. Now with the credit inclusion in LEED and the challenge from Architecture2030 for products (Arch2030, 2011), the materials industry has started to develop private EPD programs and gradually certifying products.

To help projects meet the new LEED v4 Materials & Resources credits, the National Ready Mixed Concrete Association (NRMCA) became an EPD Program Operator in 2013 (NRMCA, 2013). The objective of the NRMCA EPD Program is to facilitate the development, verification and publishing of certified EPDs for ready mixed concrete products. To maintain third party objectivity, NRMCA ensures that an independent verifier reviews and verifies each EPD developed under the program. Verifiers are individuals or verification bodies with knowledge of the concrete products industry and related environmental aspects, with process and product knowledge and have expertise in LCA methodology.

Since the EPD Program inception, NRMCA members have verified (certified) nearly 2,000 concrete products through the program and more are on the way. Central Concrete, a U.S. Concrete Company located in northern California, was the first to publish an EPD for nearly 1500 concrete products in 2013 (CSC, 2013). In 2014, Ceratech, Cemex and Argos USA followed bringing the total to nearly 2000 concrete products with verified EPDs. And the list will continue to grow as more companies publish EPDs.

NRMCA also took a leadership position and commissioned and published an Industry Wide EPD with nearly 2400 plants listed and literally millions of products with EPDs. With this inventory data, NRMCA developed the average environmental impacts for eight different regions in the U.S. so that concrete producers could compare their impacts and contribute to additional points of the LEED v4 EPD credit. Concrete is the first industry to develop its industry-wide averages in the US. Without a doubt, the concrete industry is leading the way in producing third party verified EPDs.

4. TRANSPARENCY THROUGH RESPONSIBLE SOURCING SCHEMES

Responsible sourcing is a holistic approach to the sustainable assessment of materials. It considers a wide range of sustainability issues across the entire material supply chain, and by doing so encompasses various elements of resource stewardship, corporate responsibility and procurement practices. A large portion of a company's efforts toward sustainable manufacturing happens behind the scenes. For many concrete companies, the largest opportunity for improving sustainability performance is in its supply chain.

The timber industry was the first materials industry to feel the pressure to take responsibility for their supply chain. In 1993, the Forest Stewardship Council (FSC), an international not for-profit, multi-stakeholder organization, was established to promote responsible management of the world's forests (FSC, 2015). Blindsided by the universal appeal of the certification, the timber industry responded in 1994 with their own Sustainable Forestry Initiative (SFI) program, launched by the American Forest and Paper Association. Unfortunately, the validity of the industry program has been questioned ever since and despite vigorous efforts to be included, SFI is still not accepted by environmental groups nor the US Green Building Council's Leadership in Energy and Environmental Design (LEED) rating system (USGBC, 2015).

The LEED v4 Material and Resource credit - Building Product Disclosure and Optimization: Sourcing of Raw Materials - will now reward all material manufacturers with third-party verified Corporate Social Responsibility (CSR) Reports that provide relevant information on environmental and social impacts from the extraction to manufacturing of materials. While CSR's are published as a matter of course for the larger multinational firms, many small to medium sized enterprises do not have such third–party verified documents.

The National Ready Mixed Concrete Association had previously developed a number of programs that helped the industry foster improvements and support responsible sourcing objectives. These include the *Quality Management System, Green-Star Certification, Safety Course and Certification Programs,* the *Sustainable Concrete Plant Certification* and a *c*omprehensive continuing education program to aid member companies to development its employees (NRMCA, 2015). However, these programs were not package for adoption by the US Green Building Council and did not inherently follow the approved frameworks ((USGBC, 2015).

Recognizing the need to address these new reporting requirements a group of international organizations initiated by the World Business Council on Sustainable Development/Cement Sustainability Initiative (WBCSD/CSI) came together in October 2013 to begin the process of developing an international responsible sourcing certification system for the concrete, cement and aggregates industries. A new organization, the Concrete Sustainability Council (CSC), was established to develop, launch and administer the certification system.

The CSC was formally launched in February 2014 when industry representatives met to initiate the development of a responsible sourcing scheme (RSS) for concrete during the International Concrete Sustainability Conference, organized by the Ibero-American Ready-Mixed Concrete Association (FIHP) and the National Ready-Mixed Concrete Association (NRMCA) held in Medellín, Colombia (CSC, 2015). The CSC is a standard rating system that allows organizations to evaluate, benchmark and report their performance at a regional level. This standard will seek to protect labor rights, impact on communities while managing the fair flow of capital, equitable use and sharing of benefits, economic impacts on the organization and its stakeholders.

The CSC assessment is divided up into four categories: (1) Management which includes policies, quality and chain of custody; (2) Environmental which includes Land use, energy, water, secondary materials and biodiversity; (3) Social which includes communication, community, labor practices and health and safety; and (4) Economical which includes local economies, ethics and innovation (CSC, 2015).

The CSC program will offer concrete manufacturers the ability to demonstrate and, more importantly measure their performance as a responsible organization. Ultimately, the concrete manufacturer and its suppliers should establish policies and metrics with consultation with local community stakeholders affected by extraction and acquisition of raw materials, and/or energy and material production, and/or manufacture of its assessed product, as appropriate to the purpose and activities of the company. By focusing on stewardship at every stage, from initial planning of a quarry, to mixing of concrete, to final restoration of the land, the responsible company is maintaining a sound business while respecting the environment and supporting local communities.

5. TRANSPARENCY THROUGH HEALTH PRODUCT DECLARATIONS

From brominated flame-retardants (BFRs) in home furnishings to bisphenol A (BPA) in baby bottles, consumers are more concerned than ever before with the health effects of everyday products they come in contact with. While there are many sources of indoor pollution, the materials used to make buildings are one key source of the problem. Architects and the rest of the design community began to take notice of chemical constituents of products in 2002 with the publication of *Cradle to Cradle: Remaking the Way We Make Things* by architect William McDonough and chemist Michael Braungart. In their provocative book, they challenged the notion that industry must damage the natural world in the manufacturing process (McDonough, 2002). These visionaries believe all products can be designed so that, after their useful life, they provide “nutrients" that safely re-enter the environment or recirculate within closed-loop industrial cycles.

This drive towards green chemistry in building products inspired many organizations to seek out building products that avoids toxic chemicals:

- In 2006, the International Living Future Institute’s Living Building Challenge a rigorous green-building rating system similar to the USGBC’s Leadership in Energy and Environmental Design (LEED®), raised the bar by requiring projects to eliminate 18 substances that it has identified on its material Red List (Imperative 11) (LFI, 2006).
- That same year the Healthy Building Network unveiled its Pharos Project, a materials evaluation tool. In presenting it the 2009 Environmental Award, the EPA touted Pharos as “a revolutionary on-line tool for evaluating and comparing the health, environmental and social impacts of building materials in a comprehensive and transparent way” (EPA, 2009).
- Perkins+Will, a leading architecture firm with 24 locations worldwide, released in 2009 its Precautionary List, a list of 25 substances with supporting evidence of the human health impacts that should be avoided in projects (PW, 2015).

Under the European Union (EU) REACH legislation, standard processes and tools have been used to communicate exposure scenarios. But no such standard exists in the US. One path adopted by the USGBC to disclosing a product’s constituent ingredients is through a new standard - Health Product Declaration (HPD) - developed by the HPD Collaborative. The Collaborative is a joint initiative between the Healthy Building Network and Building Green, Inc. and is one path to achieve material ingredient reporting. The HPD Open Standard is “a format for reporting product content and associated health information for individual building products and materials” and is freely available (HPD, 2015).

Similar to EPDs, the intent of the standard is to create a format so that building material ingredients will be reported in a consistent manner. However, instead of reporting on environmental impacts, HPDs reports ingredients and potential human health effects. It also helps manufacturers standardize the information provided to practitioners and it will also compliment EPDs in assisting practitioners to compare and contrast the performance of building materials.

6. CONCLUSION

To generate the cement and concrete our growing population needs, it is necessary to extract raw materials from the earth. In order to minimize the impact its activities have on the environment and create a sustainable business, concrete companies must put in place principles and best practices across its integrated supply chain. By focusing on stewardship at every stage, from initial planning of a quarry, to mixing of concrete, to final restoration of the land, the responsible company is maintaining a sound business while respecting the environment and supporting local communities.

Yesterday's compliance and minimum disclosure requirements are becoming just the price of entry into these new markets. Full product transparency is just beginning to have its moment. And not just in the environmental sense. Leading companies are starting to recognize that sustainability is an innovation and social imperative, and that as the savvy architects and engineers ask the questions, the leading concrete companies can leverage existing standards and industry programs to offer many value-generating pathways.

REFERENCES

[1] US Green Building Council (USGBC), "LEED v4, the Newest Version of LEED Green Building Program Launches at USGBC's Annual Greenbuild Conference", Nov. 20, 2013, http://www.usgbc.org/articles/leed-v4-newest-version-leed-green-building-program-launches-usgbc's-annual-greenbuild-confe, Accessed Mar 20, 2015

[2] Architecture 2030 (Arch2030) website, "Architecture 2030: Valentine to the Planet: Unleashing the Power of the Pen with the 2030 Challenge for Products", Feb 14, 2011, http://architecture2030.org/downloads/NEWSrelease_021411.pdf, Accessed Mar 20, 2015

[3] National Ready Mixed Concrete Association (NRMCA) website, EPD Program, http://www.nrmca.org/sustainability/EPDProgram/Index.asp, Accessed Apr 1, 2015

[4] Central Concrete Supply Company (CCSC) website, "Central Concrete Announces First Environmental Product Declarations (EPDs) for Concrete Mixes in the United States", April 23, 2013. http://centralconcrete.com/news-events/central-concrete-announces-first-environmental-product-declarations-epds-for-concrete-mixes-in-the-united-states/#.VSe7_1y24n8, Accessed Mar 20, 2015

[5] Forest Stewardship Council (FSC) website, "History: An Innovative Idea Takes Root", https://ic.fsc.org/our-history.17.htm, Accessed Apr 1, 2015

[6] US Green Building Council (USGBC), "LEED BD+C New Construction, v4-LEED v4", http://www.usgbc.org/credits, Accessed Apr 1, 2015

[7] NRMCA website, http://www.nrmca.org/operations/default.htm, Accessed Apr 1, 2015

[8] Concrete Sustainability Council (CSC), http://www.concretesustainabilitycouncil.org, Accessed Apr 1, 2015.

[9] McDonough, W and Braungart, M, "Cradle to Cradle: Remaking the Way We Make Things", North Point Press, 2002

[10] Living Future Institute (LFI), Living Building Challenge 2.0, Apr 2010, https://ilbi.org/lbc/LBC%20Documents/LBC2-0.pdf, Accessed Apr 1, 2015 Redlist substances include: Asbestos - Cadmium - Chlorinated Polyethylene and Chlorosulfonated Polyethlene - Chlorofluorocarbons [CFCs] - Chloroprene [Neoprene] - Formaldehyde [added] - Halogenated Flame Retardants - Hydrochlorofluorocarbons [HCFCs] - Lead [added] - Mercury - Petrochemical Fertilizers and Pesticides45 - Phthalates - Polyvinyl Chloride [PVC] - Wood treatments containing Creosote, and Arsenic or Pentachlorophenol

[11] Perkins+Will (PW) website, Precautionary List, http://transparency.perkinswill.com/default.cshtml?url=/PrecautionaryList, Accessed Apr1, 2015

[12] Environmental Protection Agency (EPA) website, “Pharos Project Recognized for Taking Green Labeling to New Heights”, Jul 30, 2009, http://blog.epa.gov/blog/2009/07/pharos-project, Accessed Apr 1, 2015

[13] Health Product Declaration Collaborative (HPD) website, http://www.hpdcollaborative.org/index-2.html, Accessed Apr1, 2015

SESSION TWO

Contribution of concrete to our society

CONCRETE PUMPING OPERATIONS FOR 3rd BOSPHORUS BRIDGE PROJECT

Göktuğ Aktaş (1), Hayri Pirgon (2), Hüseyin Bulu (3), Tarkan Büyükbaşaran (4), Abdullah Ceylan (5), Seyhan Satılmış (6) and Yılmaz Akkaya (7)

(1,2,3,4,5) Akçansa Çim. San. Tic. A.Ş., Turkey

(6) Ictas-Astaldi J.V., Turkey

(7) Istanbul Technical University, Turkey

Abstract

The reinforced concrete pylons of the 3rd Bosphorus Bridge in Istanbul, will be the tallest suspension bridge pylons in the world, standing over 322 meters tall. The tower structures are built with reinforced concrete up to a height of 305 meters, and the rest is a steel saddle for main cables. The compressive strength of the concrete used in the project is classified as C50/60 and it fulfils various durability requirements since it needs to provide at least 100 years of service life under rough environmental conditions. Including foundations, 104.000 cubic meters of concrete was used for the construction of 4 pylons of the bridge.

This paper describes the methods applied during development, testing and implementation stages of concrete pumping operations in the 3rd Bosphorus Bridge Project. Concrete pumping until the top height was completed with a single pump line for each pylon without any transfer point. In order to have a smooth concreting operation, properties of fresh and hardened concrete and elements of the pipe line equipment are tested before and after pumping. During the project, pumpability of the concrete was continuously monitored by checking measuring the pressure in the hydraulic system of the pump and the pipe line. This method allowed continuously recording the condition of the concrete and pumping equipment and also suitability of the mixture design.

As a result of this systematic work, concrete has been delivered to the highest man made reinforced concrete structure in Turkey by pumping.

1. CONCRETE PROPERTIES

Concrete used for the construction of pylons is a special product from Betonsa, branded as 100+ BETON with C50/60 compressive strength and suitable for a structure which requires min. 100 years of service life. Chloride migration value, according to NT Build 492 test standard, was below $3x10^{-12}m^2/s$ and chloride content of concrete was below %0,1 of binder content for providing higher chloride resistance for durability purposes. Fresh concrete was permitted to have a temperature between 10°C and 28°C after pumping. Initial setting time was tailored through admixtures within pours especially during the first 208 meters of slip form construction. Due to high durability requirement binder composition consisted %50 ground granulated blast furnace slag (GGBS) and %50 CEMI 42,5R LA cement (OPC), where concrete benefitted from the chloride binding capacity of GGBS. Natural sand (S1), crushed sand (S2) and two coarse limestone crushed aggregates (4-16 mm for G1 and 16-22 mm for G2) were used in the mixture design. Mixture proportions are given in Table 1.

Table 1: Mixture Design Proportions for 1 m^3

$\frac{\text{Water}}{\text{Binder}}$	Water (kg)	OPC (kg)	GGBS (kg)	S1 (kg)	S2 (kg)	G1 (kg)	G2 (kg)	Admix. Type and Amount
0,37	160	217	217	457	432	540	360	Phosphonate (%1,1 of Total Binder)

2. PROJECT

3rd Bosphorus Bridge Project has 4 concrete pylons, 2 on each side of Europe and Asia. All towers will stand at least 322 meters tall when they are completed. First 208 meters of the towers were constructed by slip forming and the rest was constructed with an Automatic Self Climbing formwork.

3. CONCRETE PUMPING SYSTEM AND PUMPING TRIALS

Based on calculations and mixture design considerations, a stationary pump, with 330 KW engine and 350 bar maximum theoretical hydraulic pressure, was selected. Max theoretical concrete pressure was 243 bars. In addition, 8,8mm thick pipes with an internal diameter of 125mm, 200 bars working pressure and 400 bars maximum pressure, were selected. These pipes are called ZX pipes with male and female connections to each other. ZX pipes proved to be much better in terms of leak proofness.

3.1 Test planning

After selection of the suitable equipment and materials based on theoretical calculations, a pumping trial was performed to test initial assumptions and actual suitability of the mixture designs. To do so, a 500-meter length horizontal pipeline was installed on the site ground. Considering 325 m of actual height of the structure,, more than 1,5 times of this actual vertical length is considered viable, since vertical and horizontal pumping exerts different pipeline pressures. Layout of the test scheme is presented Figure 1 below and actual layout and equipment used in the test can be seen Figure 2.

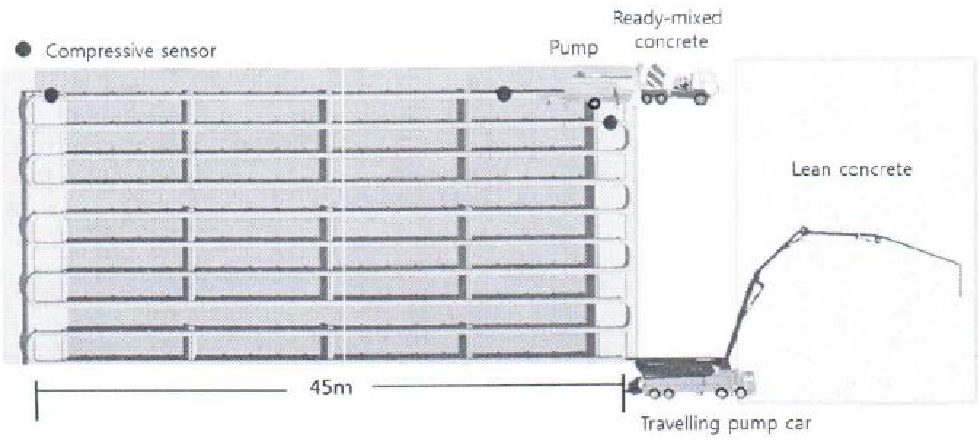


Figure 1: Pumping Test Layout

Figure 2: Pumping Test Equipment

3.2 Test operation

During the trials, 5 different concrete mixture designs were tested. Mixture proportions for these designs are presented in Table 2 below.

Table 2: Pumping test mixture designs (for 1 m^3)

ID	$\frac{\text{Water}}{\text{Binder}}$	Water (kg)	OPC (kg)	GGBS (kg)	Total Binder (kg)	S1 (kg)	S2 (kg)	G1 (kg)	G2 (kg)	Admix. Type and Amount
1	0,37	160	217	217	434	457	432	540	360	Phosphonate (1,1% of Binder)
2	0,36	158	217	217	434	441	451	542	361	PCE (1,1% of Binder)
3	0,36	158	217	217	434	448	440	550	385	Phosphonate+PCE (1,1% of Binder)
4	0,36	155	217	217	434	485	441	551	349	PCE (1,1% of Binder)

During the test, fresh concrete properties were tested before and after pumping. Also compressive strength of samples were taken before and after pumping. Practical target flow from the pump was 18m^3/hour during the trials. It took approximately 7 minutes between the discharge point from truck mixer to hopper and flowing out along the 500 meters of pipeline. Test results are presented in Table 3.

Table 3: Pumping Test Results

Test List		Mix #1		Mix #2		Mix #3		Mix #4		Remarks
		Before Pump	After Pump	Before Pump	After Pump	Before Pump	After Pump	Before Pump	After Pump	
Concrete	Flow (mm)	500	490	540	290	640	630	630	460	
	Air Content (%)	1,7	1,8	2,5	3,8	2,5	1,3	1,4	2,5	
	Temperature (°C)	26	27	27	28	26	26	26	26	
	Segregation	None		None		None		None		Visual Inspection

Table 4 (cont.): Pumping Test Results

Test List		Mix #1		Mix #2		Mix #3		Mix #4		Remarks
		Before Pump	After Pump	Before Pump	After Pump	Before Pump	After Pump	Before Pump	After Pump	
Pump	Pipe Pressure	-		-		-		-		Sensors brokedown
	Pump Pressure (bar)	158		208		138		160		
	Discharge (m^3/h)	19,2		15,5		16,1		17,1		Target: above 12
Overall Result		Complies		No compliance		Complies		No compliance		

As a result, it was proved that selected pumping equipment provides adequate capability to pump a selected concrete mixture design to the top of the pylons, and the steel pipes could endure the intended work. Another conclusion is Mix 1 and Mix 3 could be used for actual construction, however Mix 2 and Mix 4 are not suitable due to loss of workability during pumping. At the end of the test, Mix 4 became so stiff that it led to a blockage in the pipeline and, the pipeline had to be dismantled for immediate cleaning.

4 OPERATION

2 steel pipelines were installed on pylons for conveying the concrete in one lift to the top. One of the pipelines was spared as a back-up and also used for pouring elements such as crossbeams and diaphragm walls during the project. During slip forming, construction speed was around 20cm for every 2 hours and, 4.6 meters for every 5 days for the ACS formwork kit.

Until 208 meters due to slip form construction method, concrete had been pumped for 30 minutes for every 2 hours. During standby periods, it was observed that the concrete was losing its workability. In order to prevent slump loss, concrete was moved in the pipeline by sucking concrete back to hopper and moving forward. As it has been experienced stopping during the pumping operation, possesses a risk due to gravitational segregation of coarse aggregates. Another important point during stopping and pumping operations is that, the operation crew had to follow the time schedule strictly to prevent the hardening of the concrete in the pipe line. One of the few blockages during the project occurred when the prolonged work on the formwork prevented the concrete pour.

Another issue that was experienced during operation was, after a certain waiting time, first few strokes led to plugging of the pipeline just after the pump exit. After a while it was understood that this occurred whenever the pump pressure cannot move the concrete in the pipeline, first few meters is over pressurized and jammed. This issue was removed by starting at a very low flow rate in the first strokes and then gradually increasing the flow.

After 208 meters height, formwork system was changed and project has opted for ACS formwork. During the ACS formwork phase between 185 and 160 m^3 of concrete had been poured in a single pour.

5 QUALITY CONTROL

Strict quality control was employed during the pours, for every truck arriving to the site, slump, flow and temperature were measured. Slump tolerance, due to specification, was +/- 30mm, and however after 200 meters of height, for the sake of pumpability, +/- 20mm was employed. During the course of the project, some complaints were received from the pumping crew that even though slump was in acceptable range, concrete seemed to be not very pumpable. In order to check that comment, a testing schedule was employed. Firstly rheological properties of produced concretes with different slumps were investigated. Rheology is the measurement of the fundamental properties of fresh concrete, which flows under the action of shear stresses. The flow of fresh concrete can be represented by a Bingham model. In this model, parameter τ_o is the yield stress, representing the shear stress required to initiate flow. The slope of the shear stress vs. shear strain rate line is called the plastic viscosity μ, which represents the resistance to flow. These two parameters can provide a description of the flow behavior of any fluid. Fresh concrete is also a thixotropic material because shear stress required to initiate flow is high if the concrete is at rest, compared to a lower shear stress required to maintain the flow. When the applied stress reaches the static yield stress, the concrete begins to flow and the stress required to maintain the flow is reduced to the dynamic yield stress. With time, the static and dynamic yield stresses increase due to slump loss. Results of the rheology tests are presented in Table 4;

Table 5: Rheology tests

Truck Id	Slump (mm)	Rheological Parameters	
		□(Yield stress- Pa)	μ (Plastic viscosity - Pa/s)
41869	160	342,3	158,3
13522	170	321,6	149,0
41866	180	285,3	167,9
41862	200	215,4	107,8
41874	200	207,7	121,4

Even though strong correlation was observed between the measured slump and the yield stress, viscosity seemed to follow a different trend. Therefore, after an extensive investigation it has been found that even the slump is same, flow of the fresh concrete varied. Whenever the flow is on the lower end, pumps yielded significantly higher pressures. As a result, a different testing approach was taken and water to binder ratio of the concrete was changed by lowering the water content in the laboratory mixes to simulate the production variances. In these tests, it has been observed that slump correlates to the shear yield stress and flow correlates to the viscosity. Test results of those trials are presented in Table 5 below. As a result, flow of the fresh concrete was also carefully monitored and, an acceptance limit of minimum 340mm was employed.

Table 6: Rheology Tests

Slump (mm)	Slump Flow (mm)	Rheological Parameters		W/B
		□(Yield stress - Pa)	м (Plastic viscosity - Pa/s)	
190	310	165	82,2	W/B=0,35
210	400	117,5	47,1	W/B=0,37
220	380	103,8	62,2	W/B=0,36

Due to strict slump and flow restrictions during the production, pipe pressure and hydraulic pressure (during pumping and at rest) values were recorded at around a similar slump value (210mm). Hydraulic and pipe pressures, during the stand by and during the pumping are tabulated in Figure 3. Pipeline pressures were monitored by a sensor placed at 200th meter. Up to 200 meters, only the hydraulic pressure was recorded.

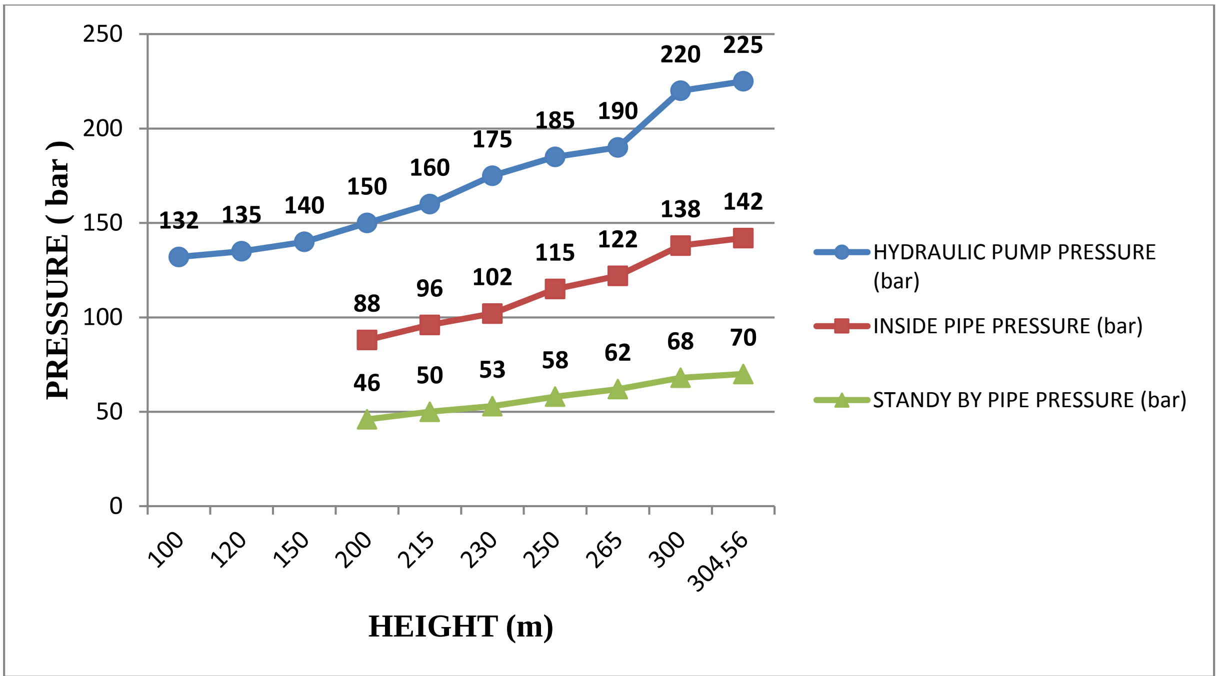


Figure 3: Height vs Pressure Chart

Figure 4 and Figure 5 shows hydraulic pressure and pressure inside the pipes during the pumping and standby conditions, while pumping concrete to 305 meters height. Due to the stroke movement of pump, there are sharp ups and downs, which represent loading and release of pistons. Flat lines between zig-zags are sta0nd by pressure of concrete between changing truck mixers. In Figure 5, a close up of pressure chart during standby can be observed.

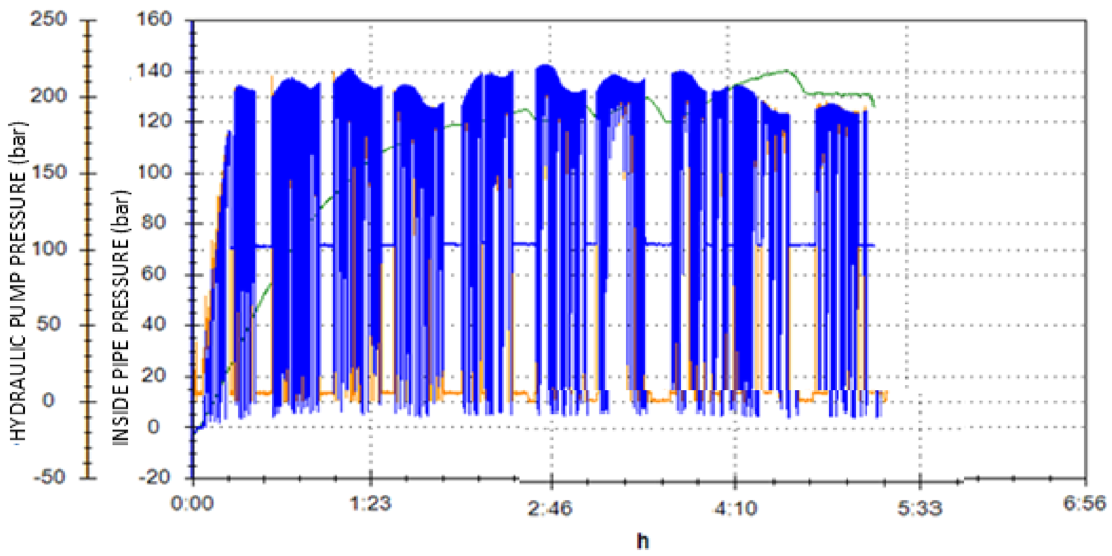


Figure 4: Hydraulic pump pressure and pipe pressure during pumping

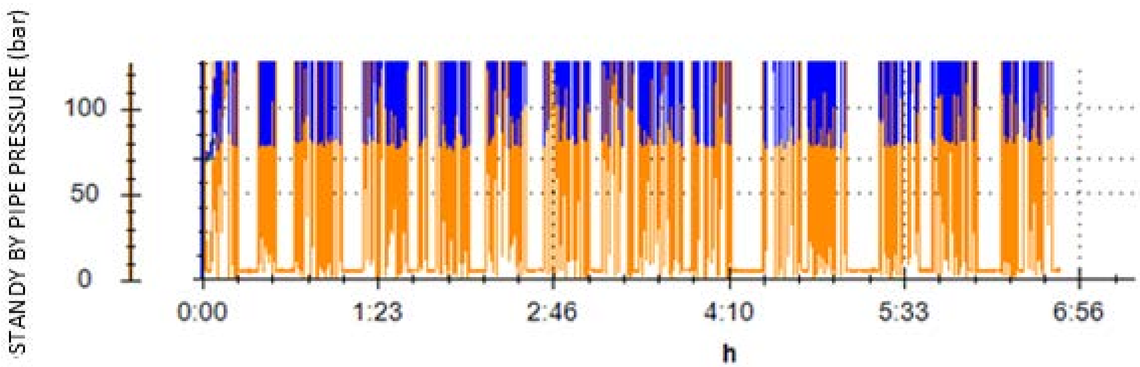


Figure 5: Pipe pressure during standby

6. CONCLUSIONS

- It was observed that, even though the hydraulic pressure during the test was 160 bars, the real hydraulic pressure reached up to 225 bars. This implies that, even the trials are a good indicator, vertical and horizontal pump lines can be very different in terms of pumping pressure.
- Once correlated with the actual conditions, and wearing of pump is taken into account, hydraulic pressure can be a good indicator of pumping capability.
- Standby pressure increases with the amount of concrete in the pipeline. Actual increase per meter of pipeline is observed to be correlated to the density of the concrete.
- Increase in the pipeline pressure per meters during pumping is almost twice the amount of increase in the stand by pressure. This could be accounted for increasing pressure head of concrete and increasing friction due to the amount of concrete.
- Flow can be taken into account as an indicator of viscosity of concrete.
- During the pumping operations, plugging is most likely to occur when the concrete in the line is moved after a period of stopping. Therefore continuous pumping can be considered as a much safer and steadier operation.

THE PANAMA CANAL EXPANSION PROJECT: 100 YEARS DIVIDING THE LAND AND UNIFYING THE WORLD

Gary de La Rosa Insignares

Chairman of the Board, Iberoamerican Federation of Ready-Mixed Concrete (FIHP)

Abstract

This paper presents the history of the Panama Canal, the biggest concrete construction 100 years ago, and details of the expansion project that is expected to be in service January 2016, in which concrete is again the most important construction material of the works. Special technical provisions for concrete have been set and huge industrial facilities and placement equipment were installed to deliver enough material for the project. This US$5.25 billion project is also having an impact on the regional ports system of all the Americas, helping to facilitate world commerce and the integration of our cultures.

Keywords: Panama Canal, marine concrete, expansion, FIHP

1. INTRODUCTION

The Panama Canal was one of the first large scale uses of concrete in the world when its construction began in the early 20th century. Due to the strategic location of the Isthmus of Panama and the short distance separating the Atlantic and Pacific oceans, from the early 19th century there were attempts to plan a route that linked the two bodies of water. It was completed and inaugurated in 1914. Almost 100 years later, Panama decided to expand its operation with a second set of locks to adapt it to modern maritime traffic, allowing the canal to accept bigger ships and lowering the crossing times for all traffic. In this presentation, we will review some aspects of the expansion project, to be completed early 2016. The total requirement for concrete in the project should be close to 4,6 million cubic meters with special provisions to guarantee its durability for another 100 years.

2. THE IMPORTANCE OF THE CANAL

2.1 Basic facts

The existing Panama Canal is 80 km in length. It opened on August 15th 1914 and is still working. The crossing of a typical ship, needs 52 million of gallons of water and ships are required to climb 26 meters to pass the Gatun Lake at the middle part of the project. As the typical transit time can be nearly 11 hours, the waiting time has been increasing from year to year due to the expansion of world commerce, reflecting the need to develop new solutions. In addition, the existing canal, designed for the biggest ships in the early 1900s, is not able to allow the passage of the most modern ships, known as “Pospanamax” in the maritime world. If you see Pospanamax ships today, you know that they are not able to use the Panama Canal, they are simply too wide.

The project of the new locks for Panama Canal, was begun at the beginning of World War II, when the United States, at that time in control of the Canal, started a project to duplicate the only locks existing then, trying to make a safer crossing for their military ships. However, with the end of the World War II, the project never really advanced and remained as just an idea, until 2006 when the Government obtained the approval of the Panamanian citizens to develop it.

The Canal expansion program has seven main components:

- Design and construction of new Postpanamax locks: one on the Pacific side and one on the Atlantic.
- Deepening and widening of the entrance from the Atlantic, with 17,66 million cubic meters of excavation.
- Widening and deepening of Gatun Lake and expansion of the navigation channels Culebra Cut.
- Construction of a 6,1 km long access channel .
- Deepening and widening of entrance areas in the Pacific.
- Expansion of the navigable areas, allowing the transit of larger vessels.
- Raising the current operating level of Gatun Lake, ensuring permanent water supply for both the existing and new locks.

In 2007, the Panama Canal Authority ACP, started the works and awarded the first contract. Actual construction started in 2007, and the whole project was initially scheduled to finish in 2014, to coincide with the 100 year ceremony of the opening of the old canal.

Several international contractors are participating in the project, including Belgian, Spanish, Italian, Korean, and Panamanian contractors.

2.2 Technical facts

All the numbers of the canal are amazing. Here is some of the information:

- Estimated project cost, US $5,25 billion, mainly financed by tolls charged by the Canal to all ships that use the canal.
- Dry excavation of nearly 50 million cubic meters of material.
- 165 million cubic meters of additional water added to the Gatun Lake after its enlargement.
- 27,000 direct jobs created since 2007.
- 4,6 million cubic meters of concrete.
- 192,000 tons of steel.

2.3 Environmental aspects

The environmental aspect is very important in the Canal expansion; the local communities and related interests were consulted at every stage and explanations were given on everything related to the project - its environmental implications, and the stages of work and all activities to be developed. A program to mitigate the environmental impact of the work was implemented. The Panama Canal Authority created an environmental monitoring program for each stage of the project that makes a report for evaluation and ensures that everything works correctly; the program activates warning signs before any foreseen eventual environmental damage, with tools to allow it to take preventive and corrective actions in time.

2.4 The concrete project

As mentioned, a large amount of concrete was specified for the expansion project, it being considered as the best material to guarantee the durability and sustainability of the construction for the future, as had been demonstrated for the old canal.

The main use of the concrete has been in works at the locks sites (Atlantic and Pacific ocean terminals), where the ships need to go up/down to reach the Gatun Lake level, 26 meters above the sea. 2,2 million cubic meters will be placed on the Atlantic Side and 2,6 million cubic meters on the Pacific side, only for the locks.

The locks complex includes walls, huge box culverts and emptying/filling systems. Several parts will be in permanent contact with sea water, so structural concrete for the marine environment was specified, including:

- 100 year lifespan.
- Maximum W/C ratio of 0,4.
- Permeability limit of 1.000 coulombs (ASTM 1202).
- Permanent Control of internal/external temperature during pouring.

Also, special formworks were designed for the project, including 33 steel forms of 14.000 lbs each with a lifespan of 5 years, to build the walls in the chambers which are equivalent to a six storey building.

Cementos Argos supplied the cement for the Atlantic Side and the mixes were designed by the contractor. The mix specifications for the concrete include 12 - 25% pozzolans, 5 - 8% microsilica, and basaltic aggregates.

There are several types of concrete specified into the project:

- Structural Marine Concrete.
- Structural Marine Concrete with high abrasion resistance.
- Mass concrete for interior side (encapsulated).
- Structural concrete.
- Levelling concrete.
- Pervious concrete for some drains.

The *structural marine concrete* must achieve 35 MPa @ 90 days with no more than 20 °C temperature difference between the surface and the interior of the concrete. *Structural marine concrete with high abrasion resistance* is specified in places (such as the drains of the locks) where the rate of water flow exceeds 8 m/s, the specified strength for this concrete is increased to 55 MPa.

2.5 Quality procedures

The Panama Canal Authority established a quality plan including several check lists, inspections and independent laboratories. The testing required in the quality plan includes:

- Cement.
- Aggregates of all sizes.
- Mix temperature.
- Strength.
- Slump test.
- Permeability.
- Shrinkage.

3. CONCLUSIONS

Just as happened 100 years ago, the new Panama Canal project is teaching us a lot of lessons which will benefit concrete construction around the world. Several of the challenges at that time remain evident today and our product is still providing ways of connecting the world in an efficient and sustainable way.

For the information provided, thanks to the Panama Canal Authority, Cementos Argos and Revista Noticreto of ASOCRETO, a FIHP member

PERMEABLE CONCRETE PAVEMENTS: THE REQUIREMENTS, USE AND METHODS OF APPLICATION

Aldona Wcisło

Lafarge Kruszywa i Beton Sp. z o.o.

Abstract

Permeable concrete is an interesting supplement to solutions available on the market, involving concrete, vibropressed or prefabricated concrete blocks. It enables us to shape our environment with a particular emphasis on sustainable construction. A properly designed and constructed permeable concrete surface or sub-base has many advantages. One of the most important advantages is the speed of draining the rainwater from the outer surface, which minimizes the risk of slipping.

In permeable concretes, the composition of ingredients is the most important part of the whole project. Application of aggregate without a constant gradation curve, or even with mono-fraction porosity, adequately durable and resistant to densification is very complicated

The most important feature parameters of permeable concrete include porosity and water conductivity. These two factors determine the choice of the aggregate (in terms of quality and quantity), and the quantity and quality of the slurry.

The proper construction of a permeable concrete surface allows for its wider application, not only because of its durability but also because of environmental aspects.

1. INTRODUCTION

Modern concrete surfaces should meet a number of requirements, such as durability, safety, comfort of use and economy of construction, or cost-effectiveness and impact on the environment and surroundings. Concrete is an interesting alternative to complement the existing range of products enabling the construction of local roads, bicycle paths, pedestrian streets, parking places in parking lots and drainage layers for each type of surface. Permeable concrete is an interesting supplement to solutions available on the market, involving concrete, vibropressed or prefabricated concrete blocks. It enables us to shape our environment with a particular emphasis on sustainable construction.

2. SHAPING THE SURFACE

Permeable concrete consists of carefully selected quantities of ingredients which allow you to create a structure with high porosity. A small amount of water, in combination with a binder, has to produce an optimum volume of paste of adequate liquidity. You have to ensure that the paste precisely covers every granule of the aggregate, does not run off, and thereby firmly combines the other components. The use of a minimum amount of slurry results in the formation of a system of voids between the granules of the aggregate. Thus, water can freely migrate from the concrete surface to the lower-lying layers. Not every type of porous concrete is permeable because this specific drainage system is not created by all such voids.

Typically, permeable concrete should have about 15-25% of voids, although the optimum value seems to be over 20%.

The factor determining the creation of the appropriate system of voids in hardened concrete is the type and shape of applied aggregate. The aggregate has a major impact on the quality and durability of the resulting concrete.

3. QUALITY REQUIREMENTS FOR PERMEABLE CONCRETE COMPONENTS

A properly designed and constructed permeable concrete surface or sub-base has many advantages. One of the most important advantages is the speed of draining the rainwater from the outer surface, which minimizes the risk of slipping.

Due to the possibility of application of permeable concrete in all types of local roads, internal roads, pavements, driveways, pedestrian paths, bicycle paths, pedestrian streets, parking places, bus lay-bys and drainage roadsides, both the components of the concrete mixture and the hardened concrete itself must fulfil a number of requirements depending on the application.

It is impossible to disregard a number of guidelines related to the type of applied concrete with fixed mineral composition [PN-EN 197-1 Cement – Part 1: Composition, specifications and conformity criteria for common cements], selection of components in accordance with concrete standard [PN-EN 206 Concrete - Specification, performance, production and conformity] or national requirements. In order to be able to use permeable concrete as the outer surface, you have to adapt to the requirements for the components and those related to the parameters for hardened concrete, i.e. resistance to frost (F150), absorption (up to 5%), water tightness (W8). In many cases, concrete is required have appropriate flexural tensile strength (4.5 – 5.5 MPa), meet the requirements related to freezing and thawing in the presence of salts (requirements for class FT2) and have the appropriate pore microstructure

($A_{300} \geq 1.5$ %, $L \leq 0.200$mm). Unfortunately, a lot of these requirements cannot be fulfilled due to the specific nature of concrete, namely its permeability.

3.1 Requirements for pedestrian streets and roads for vehicular traffic

When using permeable concrete for surfaces designed for pedestrian and bicycle traffic, in terms of components and hardened concrete, we should refer to the requirements for materials of similar application, i.e. concrete paving blocks or concrete paving flags (slabs), specified in the following standards: PN-EN 1338_2005 Concrete paving blocks – Requirements and test methods, PN-EN 1339_2005 Concrete paving flags – Requirements and test methods.

Permeable concrete is a type of framework, where aggregate must be the most durable part, which is why, despite its failure to meet the absorption or watertightness parameters, concrete must be sufficiently durable to survive the most hazardous of all factors, i.e. cyclic freezing and thawing in the presence of de-icing salts.

3.2 Requirements for cement concrete surfaces

With regard to road surfaces, permeable concrete should meet the requirements imposed on materials and hardened concrete, i.e. standard PN-EN 13877 Concrete pavements, or PN-75/S-96015 Road and airfield pavements with cement concrete, and PN-88/B-06250 Ordinary concrete.

3.3 Requirements for substructures (sub-bases)

Permeable concrete is an ideal base for subsequent layers of surfaces of roads, bicycle paths, pedestrian streets and pavements. Undoubtedly, the advantage of this type of concrete, in its hardened form, is its resistance to deformation, and thus, its permeability. Of course, depending on the application, it should meet the requirements of relevant guidelines, specifications and legal provisions. The basis of such requirements is provided by the following standards: PN-S- 96012 Automobile roads. Substructure and improved substrate of soil stabilised with cement, PN-S- 96013 Automobile roads. Substructure of lean concrete. Requirements and testing, PN-S 96023 Road constructions. Substructure and superstructure with crushed stone. Increasingly, there are references to mixtures with hydraulic binders, i.e. PN-EN 14227-1 Hydraulically bound mixtures. Cement bound mixtures, WT-5 Specifications. Hydraulically bound mixtures for roads.

3.4 Requirements for open-graded (pervious) concrete

The most appropriate classification of permeable concrete is provided by its comparison to open-graded (pervious) concrete, and the application of identical guidelines, such as PN-91/B 06263 Lightweight aggregate concrete. Permeable concrete is an improved version of open-graded concrete, with modified composition and more precisely verified quality. Depending on the ingredients used, you can control both its strength, permeability and evenness of the surface. With these advantages, the application of permeable concrete is almost unlimited.

4. DESIGNING SPACES INTENDED FOR PEDESTRIANS

Design principles for surfaces made of permeable concrete are comparable to those applicable to standard concretes. For the purpose of zones with declared durability, you should separate areas of 25 m^2 – 50 m^2 from the designed space, where the ratio of length to width ranges between 1 – 2.5 (D/S< 1÷2.5), and the length of the designed surface cannot be greater than 25 times its thickness (Figure 4).

For the purpose of proper distribution of the concrete mixture, you can create barriers to limit the distribution, in the form of expansion joints, bricks, paving stones, logs, etc., harmonizing with the concrete surface.

The thickness of the surface and the sub-base depends on operational factors and factors related to soil and water properties. Depending on the application, you should select the proper type of permeable concrete. Pavement durability does not only involve durability of the concrete itself but the longevity of the solution attributable to proper care, expansion joints and operation. The proper design of joints affects the durability of the designed surface (Figure 5). You should always apply expansion joints around fixed obstacles and try to shape the surface in a manner preventing occurrence of any sharp edges.

Expansion joints may be cut in a conventional manner - just after the concrete has achieved the required durability. Another way is to form grooves in the fresh mixture with the use of a special cutting knife (Figure 6). Permeable concrete is supplied to the installation location in a concrete mixer truck, and the mixture is not pumped but administered through a funnel. Prior to unloading, you should moisten the substrate, and lay geotextile, if required, secure the adjacent surfaces and ensure a 2% inclination of the designed surface.

5. DESIGNING THE COMPOSITION OF PERMEABLE CONCRETE MIXTURE

In permeable concretes, the composition of ingredients is the most important part of the whole project. Application of aggregate without a constant gradation curve, or even with mono-fraction porosity, adequately durable and resistant to densification is very complicated. The amount of slurry must be carefully chosen in order to enable it to surround each granule and prevent it from running off and thus gluing the system of permeable pores. The mixture of such composition must be delivered to the place of installation and properly laid, in order to ensure its durability and proper aesthetic effects (Figure 7).

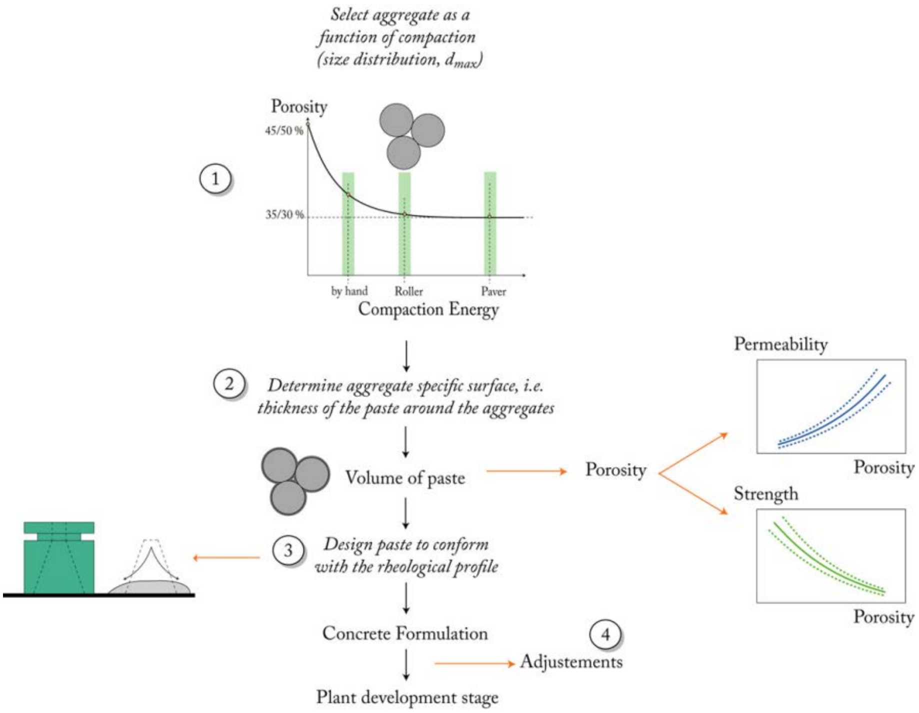


Figure 7: Permeable concrete design schame

5.1 Coarse aggregate

If you opt for permeable concrete, you should spend more time in order to find aggregate meeting both quality requirements, as well as those related to the size and shape of granules. It is very important to select the right type of mono-fraction aggregate, i.e. 2-5, 5-8, 8-11 mm (road fractions), 2-8 mm (in the case of aggregates used for standard concretes), or 2-16, 8-16, 16-22, 16-32 mm. In the case of application of granulation of 2-8 or 2-16, the risk is associated with the packing, namely, if the aggregate has been packed too densely, there is a risk of gluing the voids. For the purpose of adequate design of aggregate stack, you should use frost-resistant aggregate (depending on the application: either broken gravel or grit), with good resistance to abrasion, mono-fraction and diverse granulation, in the amount of 1500 - 1800 kg/m^3. One of the most important parameters of selected aggregate is its perviousness. In selecting it, we should pay attention to the following parameters: distribution, size and type of aggregate, and the influence of these parameters on its permeability, compactibility and resistance to compression (Table 1).

Table 1: Proces doboru kruszywa (aggregate selection process)

Step	Feature	Evaluation	Impact On		
			Permability	Compactibility	Compressive Strenght
1	Distribution Dmax/Dmin	≤2	+	+	-
		>2	-	-	+
2	Size Dmin	<4	-	-	0
		≥4	+	+	0
3	Type	Crushed	+	0	+
		Round	0	0	0
		Flat/Elongated	0	+	-
4	Lenght	L≤1.5 Width	+	0	+
		L>1.5 Width	0	+	-

5.2 Sand

The sand used in the mixture should be free of organic contaminants and have a constant gradation curve, in order to ensure the highest level of homogeneity of the mixture. You should use materials meeting quality requirements (mining sand is the most popular type). The content of sand in the mixture will vary between 0 -180 kg/m^3 and usually depends on the type of application (drainage concrete or surface concrete).

5.3 Cement

The existing specifications and regulations require the use of CEM I low-alkali type of cement, and in special cases, Portland cement, with strength class 42.5. With regard to permeable surface concrete, the best option is the application of cement with rapid increase in R strength, in the amount ranging between 260 – 330 kg/m^3, and W/C ratio of 0.25 - 0.30. In the case of drainage concrete, the class or type of applied cement is of no importance. However, you should bear in mind that migrating water may elute adverse substances. According to German guidelines, Draenbetontragschichten (DBT), the amount of cement should range between 8-12% of the aggregate mass, i.e. 150 – 220 kg/m^3. The amount of cement depends on the quality, quantity, shape and fineness of applied aggregate.

5.4 Water

You should use tap water. Each batch of delivered concrete mixture should have the same W/C (water to concrete) ratio. Usage of recycled water is not recommended for production as it may have negative influence on the aeration of the slurry and its durability.

5.5 Additives and admixtures

In order to arrive at a mixture with proper parameters, fulfilling the requirements in terms of frost resistance (F150), frost resistance in the presence of de-icing salts (FT2), and the proper pore macrostructure, you should use plasticisers, liquidisers, aeration admixtures, polymer dispersions and fibres. Obtaining slurry with the right parameters is extremely complicated. The slurry must have proper liquidity and, at the same time, proper viscosity and thickness, in order to prevent it from running off the aggregate granules.

5.6 Characteristics of the slurry

The key to success is to design the proper mixing ratio of aggregate and to select the adequate amount of slurry with proper parameters. Properly designed paste has adequate viscosity, which may be verified by using an O Funnel (individually performed test Figure 8-9), and adequate consistency, which may be verified by using a mini cone (individually performed test Figure 10-11). The whole procedure has been summarised in Table 2. The optimum amount of the slurry, designed in the manner described above, is on the surface of each aggregate granule – it does not form a rigid glue but a structure of skeleton-like connections.(Table 3)

5.7 Conditioning agent

Immediately after laying the concrete mixture, its surface must be protected from drying out. Due to the fact that the mixture will have the optimum amount of slurry with a low W/C (water to concrete) ratio, it will be susceptible to rapid evaporation of water. The first step is to secure the surface with a foil barrier preventing excessive evaporation of water. Alternatively, you may apply a non-wax conditioning agent.

Table 2: Charaketerystyka zaczynu (Paste characteristics)

Badanie (Test)	Charaketrystyka zaczynu (Paste characteristics)	Rekomendowane wartości (recommended values)
Mini O-funnel	12 - 18 [sec]	12 sec
Mini Slump	260 - 300 [mm]	280 mm

Table 3: Ilości składników dla podbudów zgodnie z DBT (The amounts of ingredients for substructures in accordance with DBT)

Składnik (Component)	Udział w % masy (Amount in % mass)	Udział w kg/m3 (Amount in kg/m3)
Cement (Cement)	8 - 12 % kruszywa (aggregates)	150 - 220
Woda (Water)	3 - 6 % cementu + kruszyw (cement + aggregates)	60 - 90
Piasek 0/1, 0/2 mm (Sand)	10% kruszywa (aggregate)	150 - 180
Kruszywo łamane 8/22, 8/32 (Crushed aggregates)	90% kruszywa (aggregate)	1500 - 1600

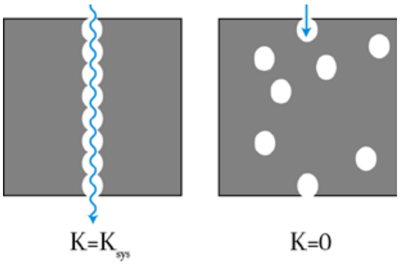


Figure 1: Difference in permeability with the same porosity

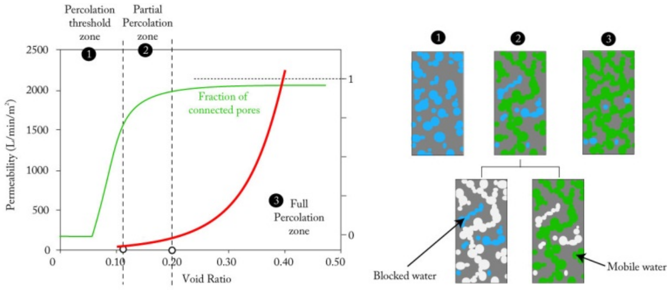


Figure 2: Optimal porosity

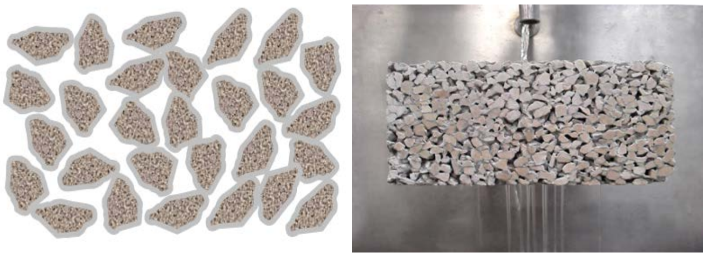


Figure 3: Pervious concrete conception

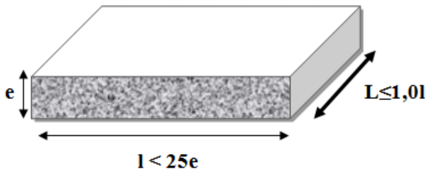


Figure 4: Scheme of designed dimensions

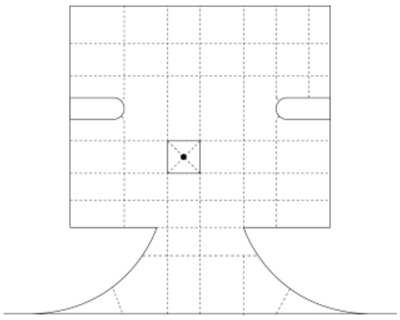


Figure 5: Joint system

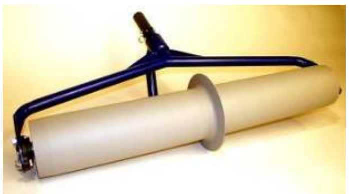


Figure 6: Joint cutter

Figure 8: Mini O-lejek (Mini O-funell)

Figure 9: Mini O-lejek (Mini O-funell)

Figure 10: Mini stożek (Mini Slump)

Figure 11: Mini stożek (Mini Slump)

6. PROCEDURE DURING CONCRETE LAYING

The technology of permeable concrete laying is very complex because the small amount of slurry forces the operator to perform laying and compaction very quickly, preferably without vibration. The concrete mixture must be laid on a properly prepared substrate, with adequate density and moisture, and complete with barriers (boundary surfaces). After the quick distribution of the mixture, the surface is initially levelled by using patches and secured with foil against excessive evaporation. Afterwards, depending on your assumptions, you should commence with pre-compaction. The purpose of pre-compaction is to ensure better distribution of the concrete mixture and the proper arrangement of the outer layer of aggregates, in order to prevent it from being pulled out. In order to minimize the risk of slurry runoff, a few methods of concrete laying and compaction have been tested – depending on the required final durability. Below, you will find descriptions of some of the simplest methods of concrete laying:

- manual paving – if only permeability parameter is required, and durability is less important (Figure 12)
- paver for concrete surfaces (Figure 13)
- mechanical compacting of the surface (Figure 14)
- mechanical compacting of the surface with the use of vibration patches (Figure 15)
- compacting of the surface with the use of a hand-roll (Figure 16)

Figure 12: Manual Paving

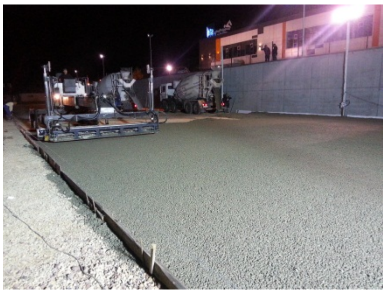


Figure 13: Placing the concrete paver

Figure 14: Mechanical compacting

Figure 15: Mechanical compacting with vibration patch

Figure 16: Compacting with hand-roll

7. CONTROL OF PERMEABLE CONCRETE PARAMETERS

The most important feature parameters of permeable concrete include porosity and water conductivity. These two factors determine the choice of the aggregate (in terms of quality and quantity), and the quantity and quality of the slurry. Depending on the application and requirements to be met by permeable concrete, you can freely decide on the arrangement of components so as to achieve the best effect.

7.1 Water permeability

A water permeability test involves measuring the time of the flow of a certain amount of water through a designated element. The measure of this parameter is K – vertical permeability [mm/s].

$$K = \frac{4\,L}{\pi\,h\,D^2} * \tfrac{M}{t} * 60$$

7.2 Porosity

A porosity test involves measuring the volume of all the voids created in the already hardened concrete. The value of this parameter is determined by p – total porosity [%].

$$p = [\ 1 - (\ V1 - V2\ ) / V\ ]\ x100$$

7.3 Segregation resistance

The purpose of a segregation resistance test is to verify the correctness of the design of the slurry. It is a measure of the amount of paste which will run off the aggregate granules under the influence of vibration. This value must not exceed 5% of the total mass of tested concrete. (Figure 17)

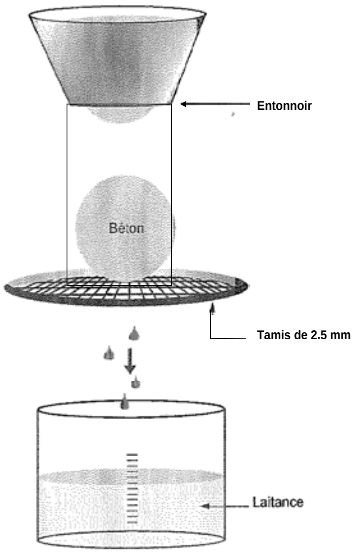


Figure 17: Segregation test scheme

8. CONCLUSIONS

Permeable concrete has a wide range of applications, depending on the ingredients used. It is a very interesting material both for the sub-base and the top layer of local roads, pedestrian streets, pavements, bicycle paths and roadside drainage surfaces. It can be freely used as a layer for draining water from tennis courts, inverted roofs or a layer for filtrating rainwater to underground tanks.

The wide range of available aggregates, dyes and possible designs of durable yet permeable surfaces allows for a wide spectrum of action.

The proper construction of a permeable concrete surface allows for its wider application, not only because of its durability but also because of environmental aspects (Multi-criteria Building Grading Systems).

REFERANCES

[1] A.Szydło, P.Mackiewicz „Nawierzchnie betonowe na drogach gminnych – poradnik"
[2] PN-EN 206 Beton. Wymagania, właściwości, produkcja i zgodność
[3] PN-EN 13877-1 Nawierzchnie betonowe, Część 1: Materiały
[4] PN-EN 13877-2 Nawierzchnie betonowe, Część 2: Wymagania funkcjonalne dla nawierzchni betonowych
[5] PN-75/S-96015 Drogowe i lotniskowe nawierzchnie z betonu cementowego
[6] PN-EN 1338_2005 Betonowe kostki brukowe – Wymagania i metody badań,
[7] PN-EN 1339_2005 Betonowe płyty brukowe – Wymagania i metody badań.
[8] Pervious Concrete Mixture Proportions for Improved Freeze-Thaw Durability – J.T.Kevern;V.R.Schaefer;K.Wang;M.T.Suleiman
[9] Durability of porous concrete for concrete pavements – E.Rickschen;E.Siebel
[10] Mix Design Development for pervious concrete in cold weather climates - J.T.Kevern;V.R.Schaefer;K.Wang;M.T.Suleiman
[11] Pervious Concrete Pavements P.D.Tennis;M.L.Leming;D.J.Akers

DETERMINATION OF TEMPERATURE VARIATION FOR DIFFERENT CONCRETE CLASSES OF AIRPORT RIGID PAVEMENTS USING LABORATORY STUDIES

Mehmet T. Seferoğlu (1), Muhammet V. Akpinar (1) and Ayşegül G. Seferoğlu (2)

(1) Karadeniz Technical University, Turkey

(2) Gümüşhane University, Turkey

Abstract

The icing on the airport concrete pavements is the main causes for the decrease in the friction resulting in a significant time and economical losses. Millions of Liras is being spent every year particularly in areas where the temperatures usually are below 0 °C although still existing methods remain incapable and an incremental cost due to planes not being able to land.

In this study, usability of heating through electrical cable method for warming the coatings to strive against snow and ice in airports was examined. Rigid pavements were chosen as the airport pavement type and three different concrete grade's (C25, C30, C35) samples were prepared. Electrical heating cables were located seven centimeters below the surface inside the samples. These samples were set to four different temperatures (-12.8 °C, -14.7 °C, -17 °C, -21.3 °C) and the surface temperatures of the samples were measured by heating the cable. The necessary minimum temperature time duration for deicing (0-4°C) concrete pavement surface was determined for the three concrete samples.

As a result of this study, it was concluded that heating concrete pavements is an effective method for deicing of concrete pavements. It was understood that stresses due to heating processes do not exceed pavement's strength. The temperature variation through the concrete pavement layer was exponential. The variation did not significantly change for the other concrete classes studied in this research program.

Keywords: Airport, rigid pavement, heating cables, deicing

1. INTRODUCTION

The risk of icing begins when the air temperature drop to below +4°C. When in the event of icing in the airport pavements, the ice reduces the frictional resistance of the surface by forming a film between the wheel and the surface (ICAO, 1983). To prevent these hazardous situations, snow and ice should be removed from the pavement surface. In our country in terms of cost and effectiveness of the traditional methods for fighting against snow and ice, attends much distress. In traditional methods which salt and some chemical solutions are using for airport pavements, have been damaged to asphalt and concrete surfaces, reinforcement in concrete, drainage pipes, the metal parts of the planes and the environment. The machinery and labor requirement of this common ice and snow control methods continues uninterrupted. The pavement heating systems began to use in our country but abroad commonly used.

There are many heating systems. These systems are divided into two categories: the systems set up into the pavement and outside of the pavement. The systems which are set up into the pavement are composed of ground source heat pipes, hot fluid heat pipes, solar water heat pipes, electrical heating cables, carbon fiber heating cables and electrical conductive concrete and asphalt pavement**.** The systems which are set up outside of the pavement are composed of snow and ice melting systems with microwave and snow and ice melting systems with infrared radiant heat.

In this study, electrical heating cables were used for deicing on airport rigid pavement surface. Temperature scale was chosen according to icing formation criteria.

2. LITERATURE RESEARCH

2.1 Electrical heating pipes and cables

The electrical heating pipe systems are snow melting systems which use the electricity for an energy source and composed of mineral isolated cables and resistant electric wires (ASHRAE, 1999; Lund, 2000). The wires and cables placed under the concrete plates and asphalt pavements can cause cracking or separation of the concrete by heating due to voltage. The heated cables or wires can be broken. To prevent this situation expansion/contraction joint, reinforcement and discharge (drainage) must be taken into account during design process (Brinkman, 1975). The thermostat and sensor must be used as part of the system for operating effectively of such ice and snow melting systems.

By controlling the temperature and humidity via thermostat and sensors the system operates at the time of snowfall begins, so is to keep the power consumption at the highest possible level down (URL 1).

Electric heating cables are used for the first time on a bridge to remove snow and ice in 1961, New Jersey-Newark (Henderson, 1963). The power density, 378 W/m^2 for bridges and 430 W/m^2 for road pavements was determined. The supplied electric current is adjusted to melt 25 mm thickness of snow per hour from the pavement surface. But in time, the electrical cables started to appear on the surface due to the wearing out of asphalt and concrete pavements because of the traffic load, so this system has been canceled. The same has been implemented in two ramps and a bridge in 1964, New Jersey-Teterboro. This system has been successful in snow and ice melting, has worked with a power density of 375 W/m^2 and an annual operating cost is identified as \$5 /m^2 (Zenewitz, 1977). Then similar systems have been implemented in Nebraska, Ohio, Oregon, Pennsylvania, South Dakota, Texas and West Virginia.

In a study, a new type of road-heating system with electrical energy was developed which is engined with a monitor that indicates the snowfall probability data generated by the Japan Meteorological Association to the four control parameters road heating system. This monitor system analyses the snowfall probability, road surface temperature, atmosphere temperature and control temperature, then arranging the surface pre-heating temperature and energy outputs sends these to the heating cables placed into the pavement (Sugarawa, 1998).

2.2 Carbon fiber heating cables

Carbon fiber cables as a heating element to melt snow and ice from the pavement surface is first time used by Zhao in 2010. Initially a study was conducted to determine the intermediate distance of carbon fiber cable which was placed into the pavement with ANSYS finite element program. And the optimal distance between the cables was determined to be 100 mm for a uniform heat distribution on the surface. Studies conducted in the laboratory, a sample with compressive strength of 47.1 MPa and 200×400×50 mm size was prepared. Carbon fiber cables were placed into the 25 mm under the sample surface and put in the refrigerator that was surrounded 10 mm thickness of insulated material and the internal temperature kept at (-25°C) at 1100×470×470 mm size. The power which has 1134 W/m^2 is transmitted to the carbon fiber cables. Five hours after the pavement surface was increased to 6-8.6 °C (Zhao vd., 2010).

In another study, a test was performed at open air and 1000×2000×250 mm size concrete sample was used when the air temperature was between (-8,-16 °C) and the humidity was between 20-60%. The carbon fiber cables were placed into 40 mm under the pavement surface. Through the temperature gauges which is placed 50 mm interval to the cross-section of the sample, the data in Table 1 was obtained after 6 hours heating process (Zhao vd., 2010).

Table 1: Air Temperature, power and pavement surface temperatures (Zhao vd., 2010).

Air Temperature (°C)	*Beaufort Wind Scale*	*Power (W/m²)*	*Pavement Surface Temperature Before the Heating (°C)*	*Pavement Surface Temperature After the Heating (°C)*
-5	2	1000	-3.5	18
-7	2	800	-6	12
-4	2	500	-3	6
-2	1	300	-0.5	8
-3	1	150	-1.6	5

3. EXPERIMENTAL STUDY

3.1 Concrete sample preparation

The concrete classes of the airport pavements in Turkey are based on the FAA procedures. The FAA recommends a design flexural strength of 600 to 700 psi (4.14 to 4.83 MPa) for most airfield applications (FAA, 2009). In this study three type of concrete classes obtained from Aşkale Çimento had at least 4.1 MPa flexural strength. The characteristics of the concrete samples are shown in Table 2. 6 samples were taken from the same mix of the big sample to determine the 3, 7, and 28 days compressive strengths.

Table 2: Mix design and compressive strength of the concrete samples.

		C25/30	*C30/37*	*C35/45*
Cement (CEM I) (kg)		340	380	430
Water (kg)		205	205	198
Aggregate (kg)	0-7 mm	984	852	805
	15 mm	376	405	410
	15-22.4 mm	464	528	528
Super Plasticizer Additive (kg)		4.08	5.7	6.88
Water Cement Ratio		0.6	0.54	0.46
Aggregate /m^3 (kg)		1824	1785	1743
Fresh Concrete Temperature (°C)		24	26	27.5
Compressive Strength (Mpa)		41	45	57

Concrete was poured in to the molds about 130 mm thick and then the cables were placed and top of that 70 mm thick concrete was poured. The FAA recommends that placement depth is nominally 2.0 to 3.0 inches below the finished surface of asphalt or concrete to maintain the desired output (FAA 2011). The total thickness of 200 mm was formed by pouring 70 mm more concrete on top of the cables. 3 mm diameter steel bars were vertically inserted at 50, 100, 150 mm depths to open holes where thermocouples can be inserted Figure 1. These bars were easily removed from the concrete after 24 hours.

Figure 1: Placement of steel into the concrete sample

3.2 Placement of heating cable

In this study, 5 m long 7 mm diameter double-conductor type 220 V with 30 W/m power capacity heating cables were used to heat 350x700x200 mm concrete block samples. The cable intervals and depth form the surface was selected based on the FAA 2011 AC-150/5370/17 (Airports Use of Heated Overlay at Airports) criteria. Cable depth and intervals specified by the FAA are 5-7.5 cm (2-3 in) from the surface and 7.5 to 22.5 cm (3-9 in) intervals, respectively.

The heating cable was placed 70 mm (2.75 in) below the surface and by 50 mm intervals in all three different concrete classes (C25, C30 and C35) as shown in Figure 2. The distance between each curling cable was 5 cm to provide homogenous heat through of the concrete sample.

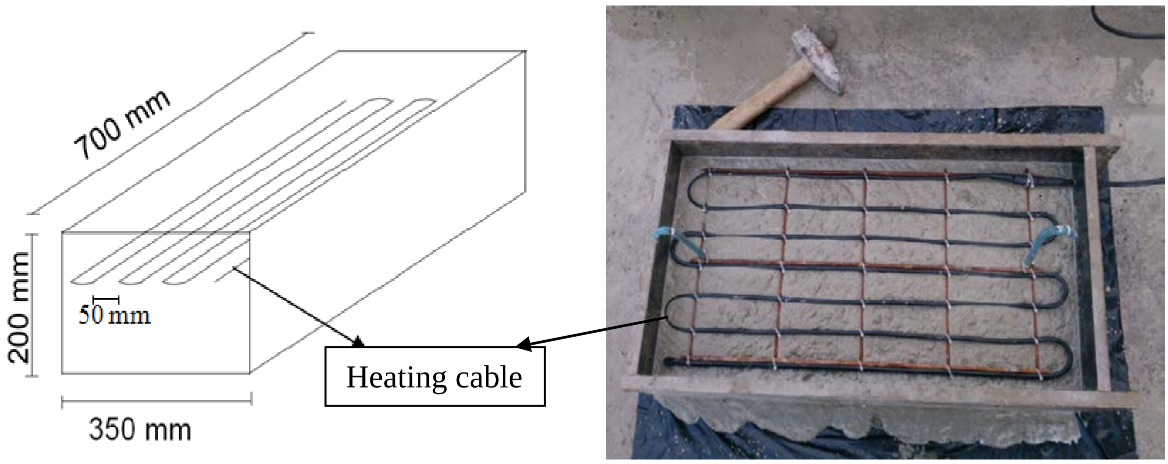


Figure 2: Heating cable positioned into sample prior to testing.

The current and voltage quantities of the heating cable were measured at room temperature (24 ° C) before starting the experiment. The corresponding data obtained was found to be 0.60 amperes and 220 volt. In this case, it was computed that 132 Watt energy will pass through 5 meters cable and will generate 65 °C heat (measured by the thermocouple connected to the cable).

3.3 Placement of concrete samples into the freezer

The testing program was initiated in the freezer to simulate the cold conditions at the airport. A freezer capable to freeze the samples between -15 and -25 °C was used in the laboratory experiments. Concrete samples were placed on top of the 20 cm thick soil layer for field condition as shown in Figure 3. The samples were isolated with 5 cm thick 16 kg/m^3 density polyester material to prevent the heat fluctuation.

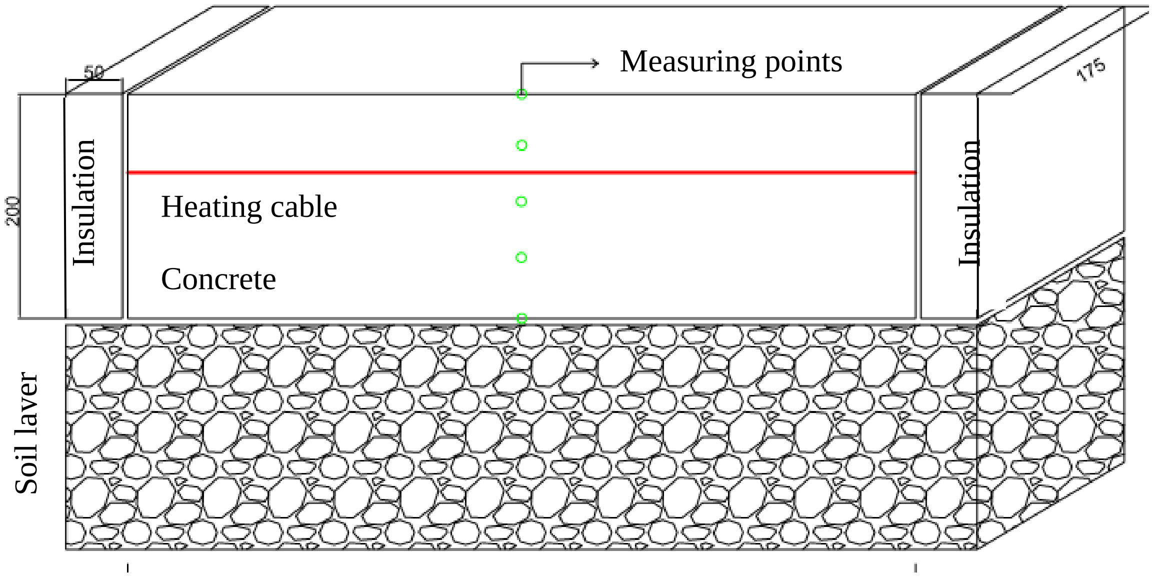


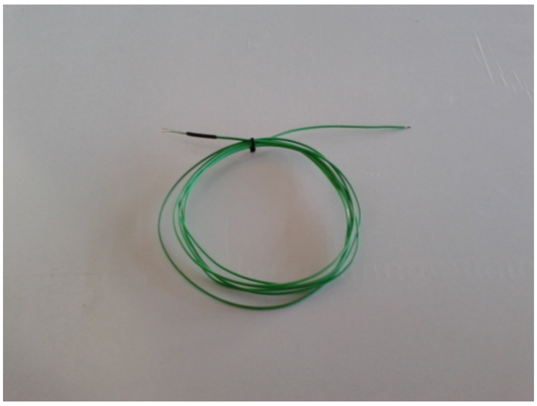
Figure 3: Concrete sample on top of the soil inside the freezer.

Once the sample was isolated grease was applied to the wholes where the thermocouples were inserted. By doing so the air fluctuation inside the wholes were prevented. Finally the top of the whole was sealed with silicon material.

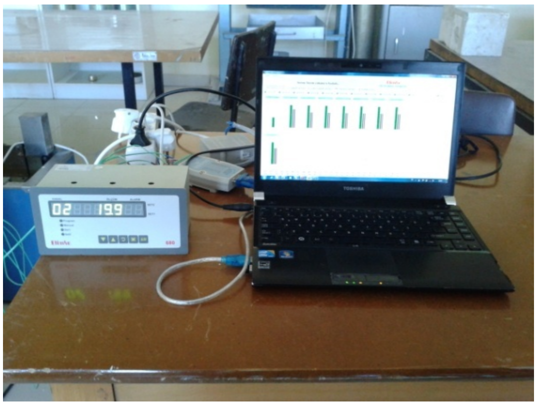
3.4 Experimental study

Temperatures were measured by the thermocouples inserted into the concrete samples and were stored into the computer using the data loggers as shown in Figured 4.b. The K-type thermocouples were capable of measuring temperatures between -200 and 1200 ° C as shown in Figured 4.a. Each sample was subjected to 12.8 ° C, -14.7 ° C, -17 ° C and -21.3 ° C, temperatures.

Samples were covered with isolated by using isolation materials and then put into a refrigerator for 1 day to maintain a homogenous constant temperature around the sample. A soon as the designed temperature were reached the cable started heating the concrete samples and the temperatures were recorded every 30 seconds.



(a)



(b)

Figure 4: a. K type thermocouple, b. Data logger and experimental setup

4. RESULTS AND DISCUSSION

Twelve laboratory experiments were performed on three different concrete classes. 132 watts of power cables placed in 70 mm below the sample concrete pavement surface heated the sample and then the temperatures on top and inside the samples were measured Attention was paid on the differences between the initial and final temperatures. The total heating time intervals for 12 samples are shown in Figure 5. A sample measurement of temperature reading is shown in Figure 6.

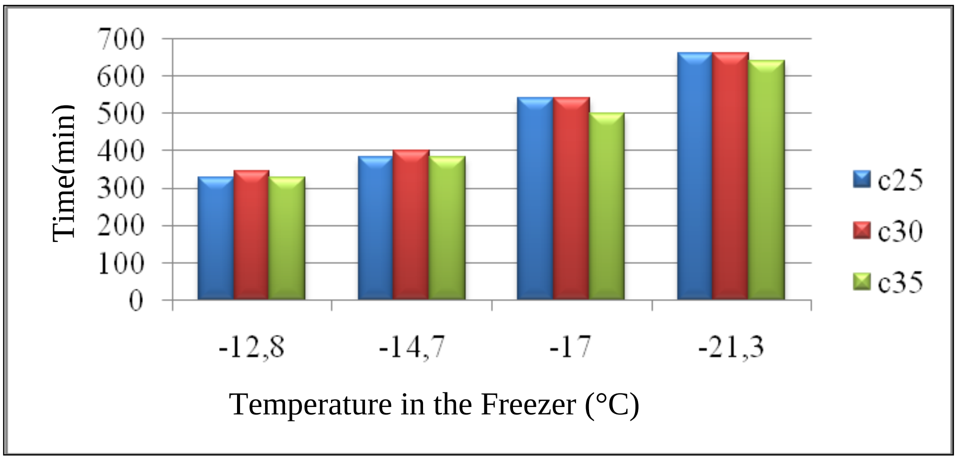


Figure 5: Heating time for concrete samples

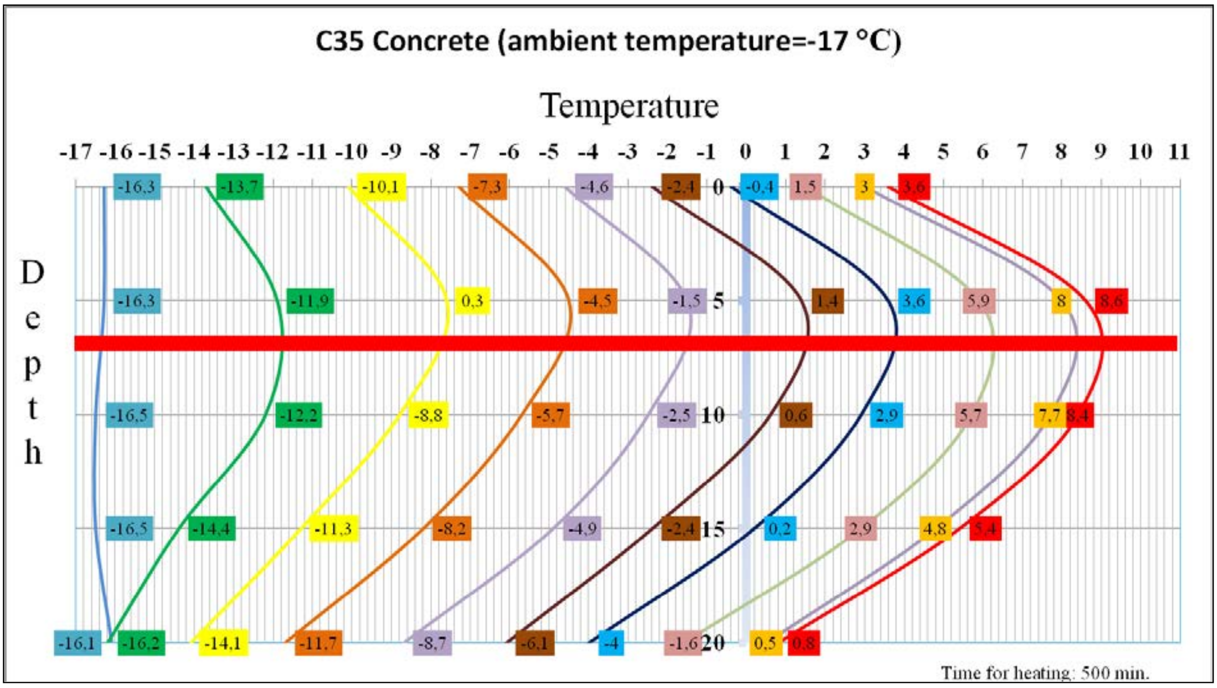


Figure 6: Laboratory temperature distribution measurements in C30 concrete (-17 °C)

The temperature distribution in the concrete samples was plotted at 1-hour intervals. The first hour is green line and last time is red line. The most prominent features of the basic shape in all graphics are that the temperature curves main structure occurs in the first 1 hour and continues to occur in the same way. This means that the sudden changes in temperature will not occur and it is continues progress does not cause any crack. According to the graphs the maximum temperature was obtained closest to the cable which was 50 mm below the concrete surface. Following 100 mm, 150 mm depths, surface and bottom of the concrete samples. In the experiments, the heating time was defined as the difference between surface starting temperature and the final temperature.

C25 concrete with 41 MPa compressive strength had the highest water cement ratio and the lowest cement content relative to C30 and C35 concretes. As it is known, the increase in the cement content has significant effect on the increase of the concrete's thermal conductivity. Also, the excessive water in the concrete later evaporates and causes the formation of voids in concrete.

These gaps, slows down the transmission of heat from the concrete. Heat conduction in the concrete would be the best when there are no voids. Accordingly, the heating time of the C25 concrete should be more than the other two concrete classes. However, based on laboratory tests, heating time for C25 concrete was higher relative to C35 concrete at temperatures -17 and -21.3 °C. C25 and C30 warmed at the same time at temperature -12.8 and -14.7 °C although the opposite was expected. However C35 was heated late at -12.8 and -14.7 °C compared to C25 concrete. As expected, C25 warmed later than C35 at all temperatures.

5. CONCLUSIONS AND RECOMMENDATION

In this study, snow and ice removal at airports by electric cable heating system built into the C25, C30 and C35 class concrete pavement layers was searched by laboratory experiments. The prepared concrete samples were subjected to four different temperatures (-12.8 °C, -14.7 °C, -17 °C and -21.3 °C) and the temperature on the surface of the samples was measured by providing electrical heating cables. The time required to reach the deicing temperature range (0 to 4 ° C) on the surface of the samples were measured. The results of this study are as follows;

- Heating concrete pavement can be considered to be a useful method in de-icing of airport surfaces.
- Laboratory studies for all samples showed that the temperature throughout the concrete layer thickness changes exponentially.
- The total heating time in all four concrete samples were 330 min, 385 min, 540 min and 640 min, to reach -12.8 ° C and -14.7 ° C, -17 ° C, -21.3 ° C temperatures respectively.
- The concrete class does not have significant effect on warm-up time for airport concrete pavements.
- When prevent icing with concrete coated airports on estimated heating cable desired temperatures from meteorological offices this system of measures can be taken to the expected low temperatures already running.
- The heating cable system can be activated based on the information provided from meteorological offices in order to prevent de-icing on the airport concrete surface. This study gives the early time durations to heat the concrete surface for varying concrete types.

REFERENCES

[1] Ashrae, 1999. American Society of Heating, Refrigeration and Air-Conditioning Engineers, Handbook of HVAC Applications, Atlanta.

[2] Brinkman, C. P., 1975. Space Age Technology for Deicing Hazardous Highway Locations, Public Roads, 89–95.

[3] FAA, 2009. Airport Pavement Design and Evaluation, AC 150/5320-6E.

[4] AA, 2011. Airside Use of Heated Pavement System, AC 150/5370-17.

[5] Henderson, D. J., 1963. Experimental Roadway Heating Project on a Bridge Approach, Highway Research Records, 14–23.

[6] ICAO, 1983. Aerodrome Design Manual, Part 3-Pavement, Second Edition.

[7] Lund, J. W., June 2000. Pavement Snow Melting, Geo-Heat Center, GHC Bullettin.

[8] Sugarawa, N., 1998, Energy Saving Characteristics of a New Type of Road-Heating System, Atmospheric Research, 113-122.

[9] Zhao, H., Wang, S., Wu, Z. and Che, G., 2010. Concrete Slab Installed With Carbon Fiber Heating Wire for Bridge Deck Deicing, Journal of Transportation Engineering, 500–509.

[10] Zenewitz, J. A., May 1977. Survey of Alternatives to the Use of Chlorides for Highway Deicing, Report No. FHWA-RD-77-52.

USE OF ROLLER COMPACTED CONCRETE IN COMPOSITE SOLUTIONS FOR HIGHWAYS

Steve Crompton

Chairman of the British Readymix Concrete Association and Board member of ERMCO

Abstract

Roller Compacted Concrete (RCC) has many attributes; it is a fast, economical and durable alternative to traditional methods of pavement construction. Increasingly clients and specifiers are turning to RCC as first build costs are competitive with asphalt whilst whole life costs are substantially lower thanks to reduced maintenance and greater longevity of concrete pavements.

However the use of RCC is limited in major highway construction as the surface regularity of RCC can make for a noisier and less comfortable ride than an asphalt pavement. There are techniques such as diamond grinding that considerably improve the surface characteristics of RCC making them more suitable for higher speed highway applications. However this paper examines the use of composite pavements which combine RCC construction with high performance thin asphalt wearing surfaces to provide economical solutions for high speed road construction.

The paper outlines construction techniques and specification requirements highlighting the benefits of composite construction in highway construction.

Keywords: Concrete, RCC, asphalt, thin surfacing, highways, roads, composite construction, durability, performance

1. INTRODUCTION

Roller Compacted Concrete (RCC) is a hydraulically-bound material with compressive and flexural strengths similar to structural concrete. It is a zero-slump material that is placed by asphalt paving machines and compacted by roller to achieve a dense and rigid pavement. It is a rapid form of construction that allows pavements to be trafficked in the same time frame as asphalt solutions.

RCC pavements are designed as rigid pavements whilst asphalt pavements are designed as flexible pavements. RCC has the structural design approach of conventional concrete pavements with a construction and mix design approach similar to asphalt.

RCC is not a new product and there are examples of its use in the 1930s but its first recorded widespread use was in the Canadian logging industry in the 1970s where the speed of construction, strength and durability led to an very economic method for the construction of access roads.

Although RCC use continued to grow in North America its use in the UK was restricted to some specialist uses in dam construction until 2010 when a number of suppliers started to investigate the benefits of RCC as an economical and durable alternative to asphalt solutions that dominate the UK pavements market.

Numerous projects have now been completed using RCC although the majority of projects have been in industrial and parking applications. It’s use in highways has been limited due to concerns over noise, skid resistance and ride comfort. This paper outlines a potential solution to these issues by combining the benefits of RCC with the surface finish characteristics of asphalt.

2. CHARACTERISTICS OF RCC

2.1 Strength

Strength is rarely an issue with and strengths above C28/35 are easily achievable with relatively low cement contents (typically below 300kg/m3) providing that the material is fully compacted and cured.

2.2 Freeze thaw resistance

RCC is not air-entrained and experience has shown that air entrainment is not normally needed, even in conditions where there is the potential for freeze-thaw damage. However the aggregates need to be freeze-thaw resistant and there has to be sufficient fine material to give a closed structure and the RCC has to be compacted as required by the specification so that the potential resistance is achieved in practice

2.3 Abrasion resistance

To achieve a high abrasion resistance, a high compressive strength class is often specified along with aggregates that have a high Aggregate Abrasion Value.

2.4 Surface characteristics

Figure 1 show the typical surface finish that is achieved. The surface finish will depend upon the mix proportioning. With such dry concretes, brushed surface finishes such as those obtained with pavement quality concrete are not possible. Consequently it is not possible to get the same skid resistance properties achieved with alternative forms of construction such as pavement quality concrete or asphalt.

Figure 1: Typical surface finish with 20mm (left) and 10 mm (right) maximum aggregate size

These properties of RCC make it a popular choice for use in industrial paving, heavy duty parking and low speed roads.

However the use of RCC in highway construction has been limited due to concerns about skid resistance, noise and ride quality.

2.5 Skid resistance

Considerable research was carried out from the 1950s to the 1970s into this phenomenon. Skidding occurs when friction between tyre and road surface is insufficient to accommodate a particular vehicle manoeuvre, such as braking or cornering (White, 2001). Properties of the road surface, tyre, vehicle and the behaviour of the driver are all significant factors, but this lecture focuses on the road surface properties.

Research found that there are two key factors that influence the skid resistance of the road surface. Firstly, at higher speeds the road surface macrotexture, or visible roughness, is key. With sufficient texture, the majority of water can drain away from the road surface / tyre interface. With insufficient texture, a thick layer of water builds up. Fast-moving vehicles will then tend to ride on the water layer, rather than the road surface. This is called aquaplaning. Clearly if the vehicle is not in contact with the road, skid resistance and manoeuvrability become negligible.

Figure 2: The consequences of low surface texture and heavy rain – Lewis Hamilton aquaplanes into the gravel trap at the 2007 European Grand Prix.

Secondly, at lower speeds the road surface microtexture, that is the microscopic texture, is the critical factor. Even if the macrotexture has allowed the majority of the water to drain, a water film will remain on the road surface. With sufficient microtexture, the vehicle tyre can break the surface water film and maintain dry contact with the road. Conversely, inadequate microtexture means that dry contact is not established and skid resistance is greatly reduced.

The road surface must provide adequate macrotexture and microtexture. If macrotexture is insufficient, high-speed braking will never become low speed braking; and if microtexture is insufficient, high speed braking will simply become a lower-speed skid. Macrotexture and microtexture are illustrated in Figure 3.

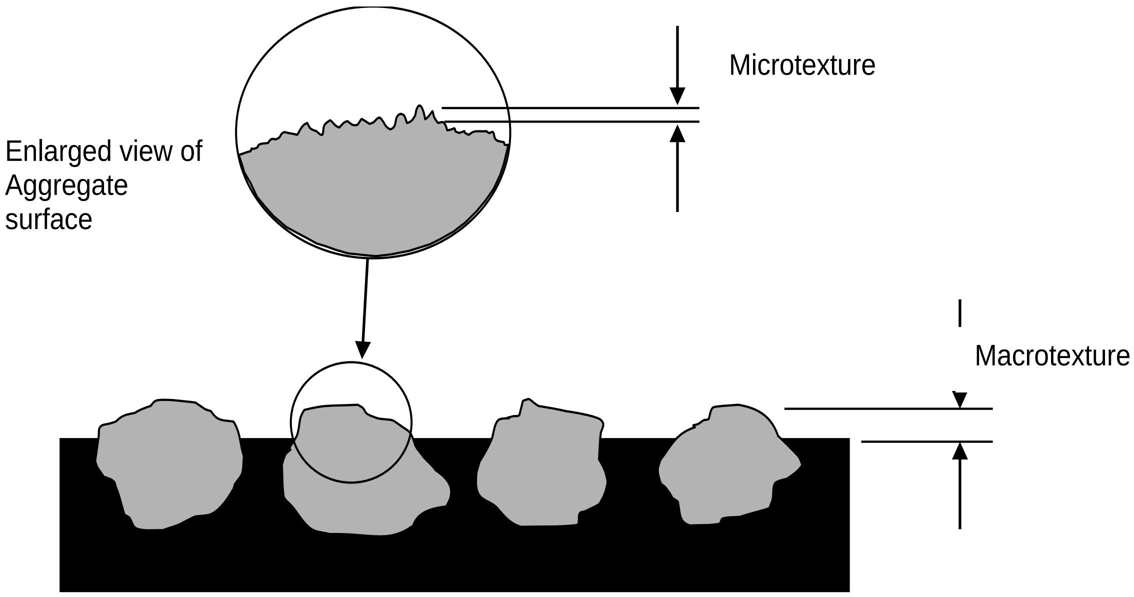


Figure 3: Diagram to show microtexture and macrotexture in the road surface

Surface friction reduces as speed increases. Surface macrotexture limits the size of the friction loss, so surface macrotexture is specified as a proxy for high speed skid resistance.

For thin surface course systems, the Specification for Highways Works (Highways Agency, 2009a) requires that the initial surface macrotexture on high speed roads is not less than 1.3mm, and that the macrotexture is maintained above 1.0mm for at least two years. Typically RCC would have a macrotexture of <0.6mm with the exact value being dependent on the upper aggregate size.

The aggregate used in RCC can be selected to ensure that the microtexture is sufficient for low speed braking but the macro texture is insufficient to provide adequate braking performance at high speed unless the surface is treated by using diamond grinding techniques.

2.6 Noise

Much of the United Kingdom is quite densely populated, and a significant proportion of the population lives close to major roads and traffic noise is an important issue. At lower speeds, traffic noise mainly originates from the engine, gearbox and exhaust of the vehicle. At higher speeds, the noise is principally generated by the interaction between the road surface and the vehicle tyres as shown in Figure 4.

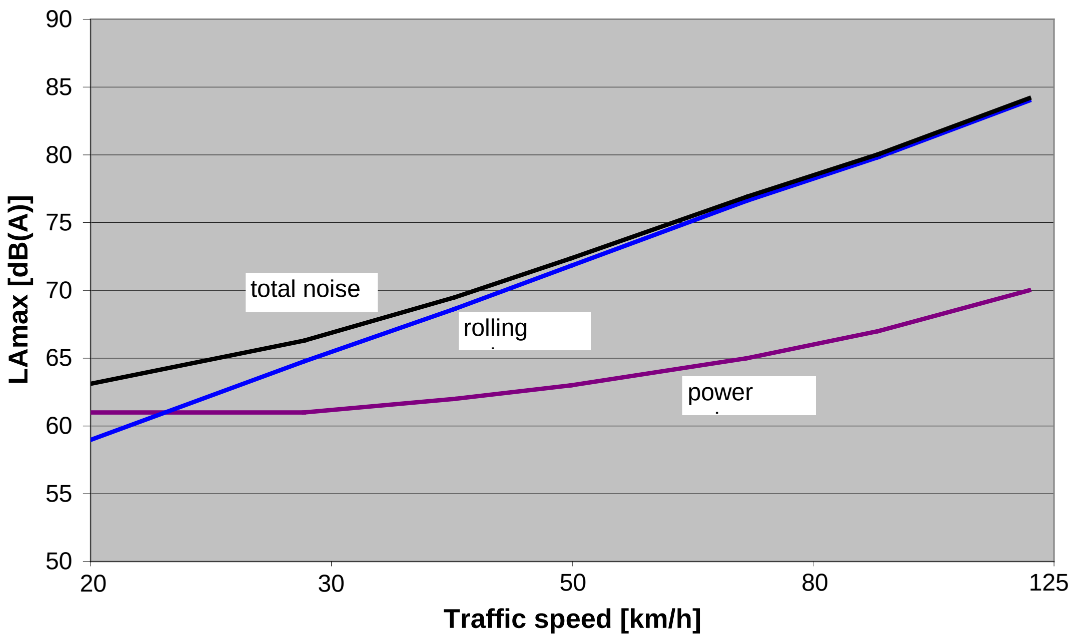


Figure 4: Noise from light vehicles at various speeds (F. Besnard et al, 2003)

The contribution of the road surface to noise is related to the macrotexture, that is, the visible roughness and megatexture which is the degree of smoothness of the surface with a wavelength between 50mm and 500mm.

Noise is generated by compression of air between the tyre and the road. The shorter the escape path for the air, the lower the noise generated. Generally, greater macrotexture gives shorter escape paths and leads to lower noise. The nature of the macrotexture, whether it is positive and negative, also has a major impact on noise. With positive texture, the tyre is constantly striking the aggregate projecting above the surface, leading to vibration of the tyre wall and additional noise. On the other hand, with negative texture, air pressure is reduced in the contact area between the tyre and the road. The voids within the surface may even absorb some sound, and the sound that escapes is at lower frequency.

The lower macro texture of RCC means that in comparison to asphalt there is much higher tyre noise at high speed. The noise levels can be reduced by diamond grinding and grooving but even with this technology it cannot match the performance of high performance asphalt solutions.

2.7 Ride Quality

Ride quality can be defined as the degree of comfort provided to road users by the road surface. This property is a combination of the evenness of the road, together with vehicle speed and the ability of the vehicle to protect occupants from any unevenness of the surface. Evenness is the degree of surface regularity at wavelengths over 500mm.

Motor car design and suspension systems usually compromise between comfort, which requires a high centre of gravity and soft suspension, and handling, which requires a low centre of gravity and firm suspension. Lorry design and suspension systems also have to allow for load carrying, which reduces comfort.

However, suspension systems can only isolate the vehicle from irregularities than are shorter or narrower than the dimensions of the vehicle, that is the microtexture, macrotexture and most of the megatexture elements of the surface. Longer wavelength unevenness may even be amplified by the suspension. The principal consequence of this is increased driver fatigue, though back pain and reduced vehicle control can also result (Granlund and Lindström, 2004).

Surface regularity is provided by the correct installation of appropriate road pavements using good surfacing practice. This vital and wide ranging subject is beyond the scope of this paper.

However The Specification for Highway Works (Highways Agency, 2009b) places tolerances on surface levels of newly constructed roads. In addition, longitudinal surface regularity is specified using a rolling straight-edge. This device is a 3 metre long frame with rows of small fixed wheels along each side. In the centre is a pair of wheels that are not fixed, but able to move up and down. Thus the fixed wheels provide the overall level over the 3m length and the moving wheels drop down into any depressions or rise up over any bumps.

It is difficult to achieve the longitudinal surface regularity requirements with RCC and it can lead to an uncomfortable ride compared to asphalt or traditional concrete pavements.

3. COMPOSITE CONSTRUCTION

RCC has many advantages as a road pavement material: Speed of application, economy, high stiffness leading to reduced fuel consumption, lower whole life cost and environmental benefits.

On the other hand asphalt pavements have advantages related to skid resistance, lower noise, reduced surface spray and improved ride quality.

The use of a composite solution utilising an RCC base with a thin high performance asphalt wearing course can utilise the strengths of both materials whilst overcoming the drawbacks associated with each material.

4. DESIGN FOR RCC COMPOSITE CONSTRUCTION

The design of a RCC composite will be influenced by the volume of traffic utilising the road, the maximum speed and the ground conditions. Factors to be considered would be the overall thickness of the road, the strength of the RCC, the type and thickness of the asphalt overlay and the need for sub base materials beneath the RCC.

There is no specific guidance for the design of RCC roads in the UK but the Highways Agency design model can be used to establish the thickness of the required road based on the volume of traffic expressed as millions of standard axels (msa).

The design chart shown in figure 5 can be used although it is based on traditional Cement Bound Granular materials which have much lower strength than RCC and therefore the recommendation for the asphalt overlay are excessively thick.

The use of RCC base coupled with high performance polymer modified surface course materials allows the thickness of the overlay to be reduced whilst still maintaining overall performance.

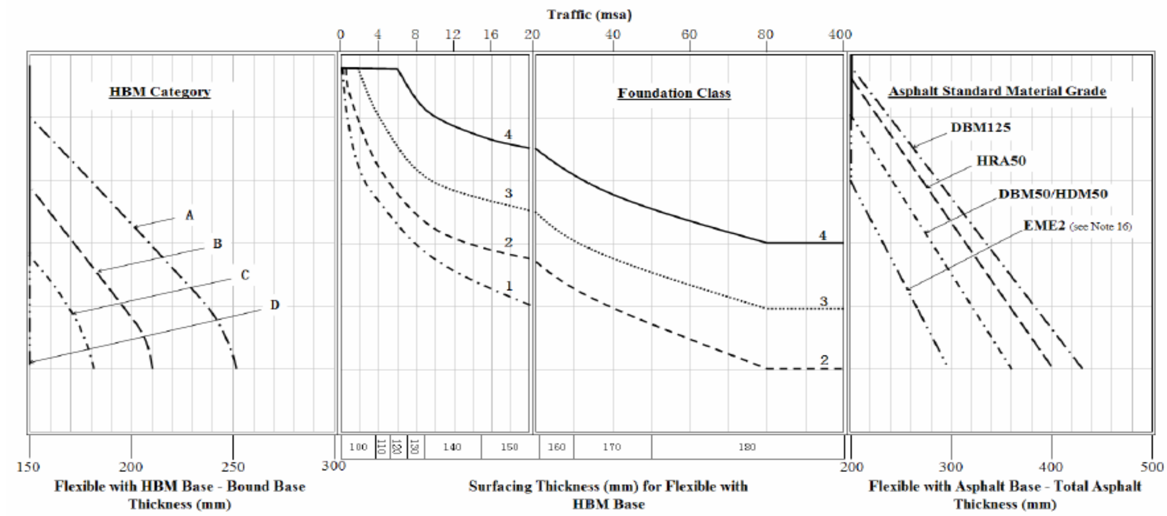


Figure 5: Design Chart for composite construction

5. SPECIFICATION OF ASPHALT OVERLAY

Stone mastic asphalt (SMA) consists of an interlocking network of high PSV coarse aggregate particles, with the voids between the coarse particles almost filled with a mastic of bitumen, filler, sand and cellulose fibres. When installed and compacted, this gives a surface course which is dense throughout most of the layer, but has an open, negative texture at its surface. This leads to a skid-resistant, deformation-resistant and relatively quiet surface.

Thin surface course systems are asphalt mixtures usually based on the use of polymer modified bitumens which give enhanced durability and flexural characteristics. They are similar to stone mastic asphalt, or very thin open asphalt concrete, and share their skid resistance, noise reduction and deformation resistance characteristics. Thin surfacings are supplied as "systems". Their design, manufacture and installation are vital to the achievement of the desired road surface properties.

A composite RCC road structure is exceptionally durable in the lower layers which means that maintenance will only require the replacement of worn out surface course. Thin surface course systems can be replaced rapidly, helping to reduce traffic disruption and other user costs associated with road works.

Use of thin surface course systems will allow the thickness to be reduced to below 50mm.

6. INSTALLATION OF RCC COMPOSITE ROADS

RCC should not be placed on standing water or placed in heavy rainfall. To minimise the risk of damage due to freezing at an early age, RCC should not be placed when the ambient temperature is less than 5°C.

RCC is usually placed with an asphalt paving machine. Conventional asphalt paving machines compact the RCC by either tamping or vibration. Combination paving machines compact the RCC by a combination of tamping and vibration. As conventional machines achieve about 80% to 85% of the design density and combination machines > 90% of the design density, further rolling is essential. Combination machines are needed to place thick layers of RCC but, when these are not available, conventional paving machines are used to place the RCC in two 100mm to 150mm layers instead of one thick layer. Detailed guidance on installation of RCC can be found in The ERMCO guide to Roller Compacted Concrete.

In order to ensure the full benefit of a composite solution it is essential to ensure a good bond between the RCC and thin surface course.

To achieve this a tack coat of a cationic bitumen emulsion should be applied directly to the freshly laid RCC as soon as possible after the final rolling of the concrete.

Following the application of the tack coat and immediately prior to the installation of the thin surfacing a hot polymer modified cationic bitumen bond coat should be machine applied at a uniform rate of $0.35kg/m^2$.

The strength of the bond coat can be measured and the results from a recent contract are shown in figure 6 confirming that the methodology outlined comfortably exceeds the minimum requirement of 0.7Mpa.

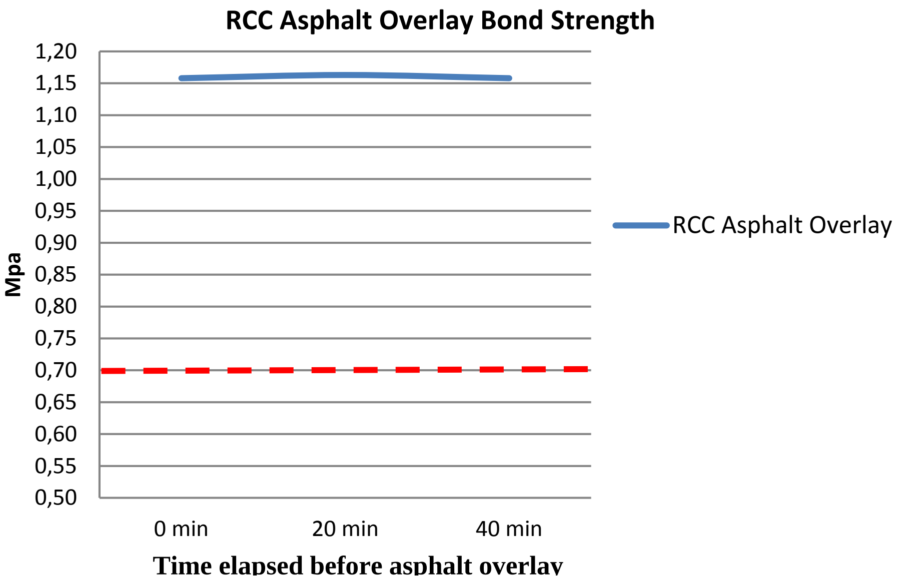


Figure 6: Asphalt overlay bond strengths

Over time reflective cracking of the asphalt overlay above the RCC joints will occur and to prevent the need for post installation repairs it is recommended that sealed joints are formed in the asphalt aligned with the RCC joints.

7. CONCLUSIONS

The use of an RCC Thin Surface Asphalt composite construction in high speed roads overcomes the limitations associated with the use of a full depth RCC or full depth flexible construction. It provides the rigid, durable and low cost pavement associated with concrete pavements combined with the low noise, high skid resistance properties associated with Thin Surface Asphalt systems.

REFERENCES

[1] Ashfield, M.J., (2014). Road Surface Properties and Treatment. CEMEX UK Materials

[2] ERMCO (2013) Guide to Roller Compacted Concrete for Pavements

[3] Besnard, F., Bérengier, M., Doisy, S., Fürst, N., Hamet, J.F., Lelong, J. and Pallas, M.A. (2003). *The procedure for updating the vehicle noise emission values of the French "Guide du Bruit".* Paper ID: 146, Euronoise, Naples.

[4] Granlund, N.O.J. and Lindström, F., (2004). *Reducing Whole-body Vibration by Geometric Repair of Pavements.* Journal of Low Frequency Noise, Vibration and Active Control, Vol 23, No. 2. Multi-Science Publishing Co Ltd, Brentwood.

[5] Highways Agency, (2008). *Design Manual for Roads and Bridges.* Volume 7 – Pavement Design and Maintenance. HD 29/08, Pavement Maintenance Assessment. The Stationery Office, London.

[6] Highways Agency, (1999). *Design Manual for Roads and Bridges.* Volume 7 – Pavement Design and Maintenance. HD 37/99, Bituminous Surfacing Materials and Techniques. The Stationery Office, London.

[7] Highways Agency, (2006). *Design Manual for Roads and Bridges.* Volume 7 – Pavement Design and Maintenance. HD 36/06, Surfacing Materials for New and Maintenance Construction. The Stationery Office, London.

[8] Highways Agency, (2009a). *Manual of Contract Documents for Highway Works.* Volume 1 – Specification for Highway Works. Series 900, Road Pavements – Bituminous Bound Materials. The Stationery Office, London.

[9] Highways Agency, (2009b). *Manual of Contract Documents for Highway Works.* Volume 1 – Specification for Highway Works. Series 700, Road Pavements – General. The Stationery Office, London.

CONCRETE PAVEMENTS IN TUNNELS: THE SUSTAINABLE CHOICE

Giuseppe Marchese

General Manager of Calcestruzzi S.p.A., Italy

Abstract

Italy has the greatest number of tunnels in the EU; worldwide, only Japan has more. Currently the Trans-European Road Network (TERN) counts more than 600 km of tunnels throughout Italy and 10% of those are strictly aligned with the safety European standards (2004/54/CE).

Concrete is considered the genuine solution for tunnels, regarding its capability to improve safety structural performances. Moreover, concrete is continually proving itself as the best answer to the economic, social and environmental sustainability of tunnels projects.

The "Quadrilatero Marche Umbria" project in central Italy has provided a very good example of concrete adapting Italian road tunnels to the European standards. The tunnels of Maxilotto 1 (SS 77 "della Val di Chienti"), 40 km long in total, have been built using concrete pavements. Unreinforced Jointed Concrete has been considered the most suitable choice for this work using unreinforced concrete slabs, separated by transversal and longitudinal reinforced joints. Joints reduce the stresses the concrete will experience during its life and greatly increase the concrete pavement's lifetime. Through the presence of reinforcement, the location and spacing of cracks can also be controlled.

By using concrete for road tunnel pavements, the "Quadrilatero" project has provided important elements for the correct assessment of the whole infrastructural work. Considering the early cash flow assigned to the work, the real cost of the project plan has been massively rewarded. This paper focuses on the economic benefit of concrete for road tunnel paving.

Key words: Concrete, pavements, tunnel, dual layer, infrastructure, public work, slip form, sustainability, road transport.

1. INTRODUCTION

Road construction culture in Italy goes back to ancient times. The Romans built great roads using deep roadbeds of crushed stone as an underlying layer to ensure that they were kept dry, as the water was able to flow out from the crushed stone instead of creating mud in clay soils.

Over the centuries Italy developed a deep knowledge of road construction. Due to the number of mountain chains all along the Italian peninsula, Italian design and building professionals increased their special experience in road tunnel construction.

Despite this expertise, till few years ago concrete did not find the space it deserves in road pavements in Italy. This was for a number of reasons such as the greater initial cost when compared to other materials, the absence of standards and specifications, etc.

In the early 2000s a number of tunnel fires showed the disastrous effects of road accidents when they occur in a tunnel. The fires in Gotthard (Switzerland), Tauern (Austria), and Mont Blanc tunnels (France/Italy) had tragic consequences in terms of human life, environment and economic impact. Since then, tunnel design has become mostly an "emergency design".

The consequences of a fire in a tunnel have clearly demonstrated the need for an appropriate choice of materials to ensure high safety and reliable availability to traffic. In the case of fire a non-combustible and non-toxic road pavement, such as on built in concrete, contributes to the safety of people (both users and rescue teams) and protects the tunnel equipment and its structure. Therefore, renewed interest in concrete solutions for tunnels has grown over recent years in Italy.

Another reason for this change of attitude could be the Directive 2004/54/EC aiming to ensure a minimum level of safety for road users in the Trans-European Road Network (TERN) tunnels. The Directive includes instructions about prevention of critical events that may endanger human life, the environment and tunnel installations, as well as provisions in case of accidents. All tunnels in the Trans-European Road Network with lengths over 500 m, whether in operation, under construction, or at the design stage have to meet requirements identified by this Directive. Though 600 km of all TERN tunnels are in Italy, only 10% of them are strictly aligned to the European Directive - so the Italian Government should upgrade more than 500 km of tunnels [1].

2. SUSTAINABILITY OF CONCRETE PAVEMENTS IN ROAD TUNNELS [2]

A concrete pavement in road tunnels shows social, economic and environmental sustainability.

2.1 Social sustainability

The choice of a concrete pavement improves both safety and comfort of road tunnels. Concrete is non-combustible (Figure 1), does not release smoke, does not change shape when submitted to high temperature and keeps its mechanical performances in case of fire. In addition, spalling is very uncommon in road pavement concrete. All these properties lead to an increase of safety for both users and rescue teams in case of fire [4] [5] [6].

Figure 1: Comparison of specimens of asphalt (left) and concrete (right) after heating to 750°C [3]

A concrete pavement is smooth and quiet over the life cycle providing constant performances in terms of travelling comfort. The low maintenance requirement, due to concrete's intrinsic durability, is another of the principal advantages of concrete pavements. This means fewer traffic disruptions and minimal accident risk. The light coloured surface contributes to minimizing accident risk as well as to improving users' comfort.
Furthermore, concrete production being a local one, it contributes to the economic growth and welfare of local communities, when road construction is carried out.

2.2 Economic sustainability

Reduced fuel consumption, lower lighting due to higher surface reflectivity and limited maintenance make concrete preferable to competiting materials. The total cost of a concrete pavement is lower than for other materials <u>if maintenance costs are included</u>. The extra initial cost of a concrete pavement is paid off after about 10 years, and after 30 years the concrete solution becomes more economic due to the low need of maintenance. Further economic advantages of using concrete in road tunnels lie in the minimal damages to the structure in case of fire.

2.3 Environmental sustainability

Concrete is itself a low environmental impact material. Durability performance ensures unvarying surface features, so grip uniformity throughout the whole road service life contributes to reducing fuel consumption and traffic pollution. In the same way, reduced traffic diversions due to maintenance activities help to bring emissions down.
In addition fluid pollutants can be easily drained into gutters, because fuel leakages do not damage the concrete pavement.
Concrete luminescence gives the advantage of lowering energy consumption for lighting; and the opportunity of using recycled materials in concrete manufacturing makes concrete sustainable. Finally, concrete pavement itself can be fully recycled to produce aggregates.

3. A CASE HISTORY: THE "QUADRILATERO MARCHE UMBRIA" [7]

The "Quadrilatero Marche Umbria" project has a key role within the Italian road network as it connects different areas in central Italy (Regione Marche and Regione Umbria). The project is divided in two lots (Maxilotto 1 and 2). Maxilotto 1 (Figure 2) - a 1.100 million euros investment has more than 35 km of 4 lanes road including 18 viaducts, 14 natural tunnels and 12 artificial ones. The final expected cost is 2 billion euros. The awarding authority is Quadrilatero Marche Umbria S.p.A., a project-dedicated Joint Stock Company.

96% of the contract has been already completed: on January 16, 2015 the General Contractor Val di Chienti S.C.p.A opened to traffic a 9 km stretch between Colfiorito (PG) and Serravalle di Chienti (MC), and the project is expected to be completed by the end of 2015.

The concrete pavement was introduced in Maxilotto 1 by the General Contractor as a technical improvement. This choice was successfully approved by the owner because of the advantages outlined above in terms of environmental, social and economic sustainability. Concrete was used in the all double tube tunnels longer than 1.000 m, for a total of 10 out of the 14 tunnels. The surface area of the two 20 km motorway lanes in each tube is 300.000 m^2.

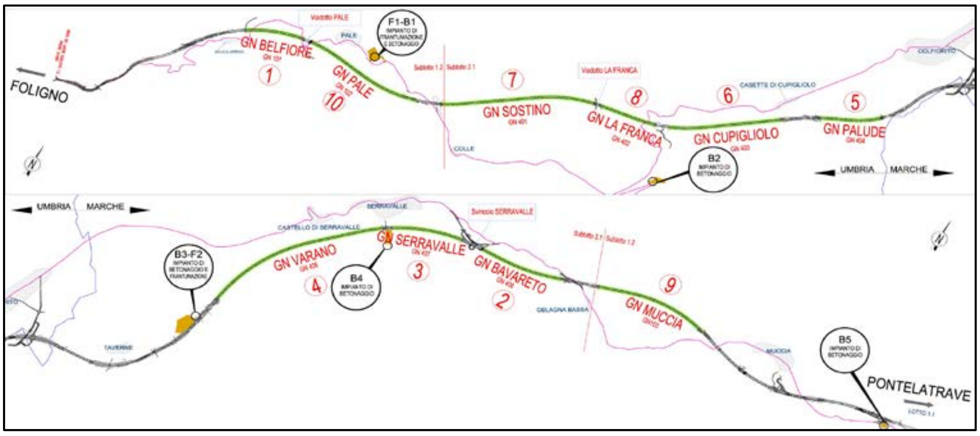


Figure 2: Quadrilatero Marche Umbria - Maxilotto 1

3.1 Tunnel pavement design and laying

The German Catalogue RStO-2001 was applied for the design of concrete pavements with heavy traffic load, resulting in an SV construction class according to the catalogue.

A service life of 30 years was fixed in agreement with relevant standards. In reality, the service life for Maxilotto 1 concrete pavements is expected to be more than 40 years. JPCP (Jointed Plain Concrete Pavement - Figure 3) type was chosen among the many technical solutions. A Joint Plain Concrete Pavement does not contain any steel reinforcement but in a given number of transversal and longitudinal joints to allow to control the location of all the expected shrinkage cracks - concrete cracks are expected at the joints only and not elsewhere in the slabs.

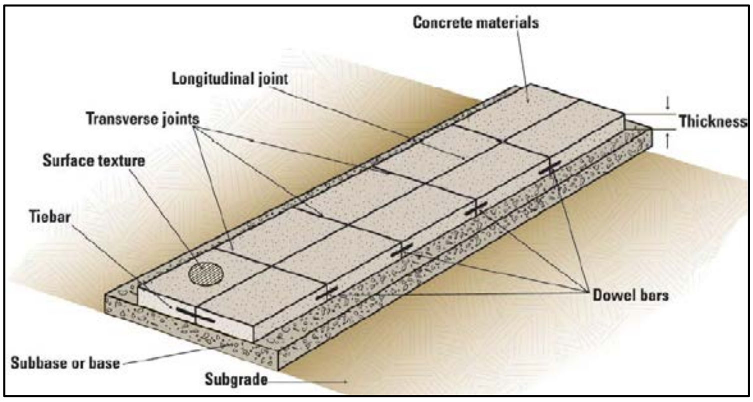


Figure 3: Joint Plain Concrete Pavement (JPCP)

In Quadrilatero tunnels the total pavement thickness - 60 cm – is made up of a 30 cm sub base, made of stabilized granular material, and a 30 cm "dual layer" concrete slab with a 25 cm lower layer and a 5 cm upper layer (Figure 4).

Different types of joint control cracking to improve pavement durability. Isolation joints at lane edges have the purpose to make concrete slabs independent from other constraints such as the presence of utilities. Cracks are controlled by including longitudinal and transverse contraction joints.

The longitudinal ones are sawed in the centre line then tied together with five 20 mm diameter tie bars placed at 110 cm centres. Transverse contraction joints are placed at 5,5 m centres, the spacing being 20 times the slab thickness.

Transverse joints have dowels placed every 25 cm where the heaviest stresses are expected, as well as in the free edges. Elsewhere the spacing is 50 cm.

The two-layer concrete pavement is placed by slipform pavers, specific for tunnel construction. A paving train lays down the two layers in just one pass using the "wet-on-wet" method, placing the upper layer concrete immediately after the bottom one. This method ensures optimal interlocking of both layers. The average production is about 300 m/day with a maximum of 500 m/day.

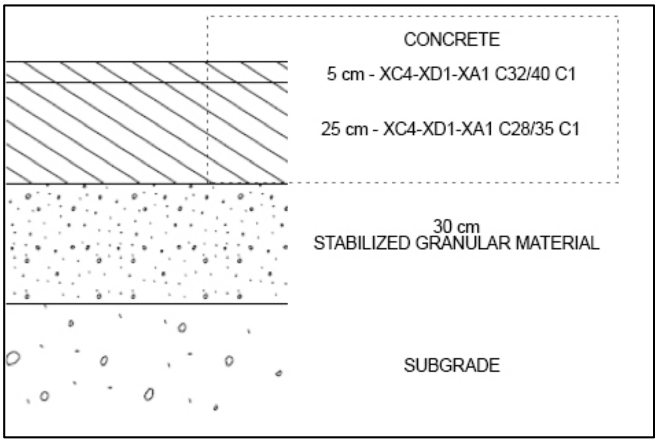


Figure 4: Road cross section

The first slipform paver is responsible for the subgrade. The 25-cm bottom-layer concrete is paved first to line and level by the slipform paver and compacted by an internal vibrator, while the slipform simultaneously embeds dowels and tie bars in it. This yields a homogeneous base for the 5-cm-thick top-layer concrete. A charging conveyor delivers the top-layer concrete to a receiving hopper on the first slip-form paver and the second paver then compacts it. Manual finishing gives the desired surface finish. To prevent rapid evaporation and cracks, immediately after paving the concrete surface is sprayed with curing compounds across its entire working width.

3.2 Concrete and materials for tunnel paving

Concrete for Quadrilatero tunnel paving is produced by a consortium whose partners are Atecap members Calcestruzzi S.p.A., Colabeton S.p.A. and Luigi Metelli S.p.A. They produce concrete for both levels, the lower layer (25 cm) is a C28/35 class concrete, whereas the upper one is C32/40.

Two different types of aggregates are used for the two concrete mixes. In the upper layer, basaltic aggregates with high abrasion resistance are provided to improve grip and reduce surface damage. In the lower level, limestone aggregates are used to comply with concrete specification.

The execution method requires an “earth-moist” (no-slump) concrete characterized by a low water content and stiff consistence. Ad hoc mix design allows the required performance to be met - tunnel pavements include a number of specific performance for concrete, such as tensile splitting strength.

Portland Cement was found to be the most suitable for road pavements, even though CEM II/B-S 42,5 or 52,5 would also be suitable. It should have consistent and established strength development and a Blaine fineness higher than 4000 cm^2/g.

Concrete consistence is really important in tunnel paving to ease placing. A minimum compaction C2 class is needed. Medium-range water reducing superplasticizers and air-entraining admixtures may help.Table 1 gives properties and constituents of the tunnel pavement concrete.

Table 1: Concrete for Quadrilatero tunnel pavements

CONCRETE PROPERTIES	LOWER LAYER	UPPER LAYER
Strength class	C28/35	C32/40
Consistence class compacted to EN 12350-4	C1	C1 and C2
w/c ratio	≤ 0,41	≤ 0,45
Exposure classes	XC4-XD1-XA1	XC4-XD1-XA1
Cement type and class	CEM I 42,5 N	CEM I 42,5 N
Aggregates	limestone D_{max} 25 mm	Basaltic
Superplasticizer	1,14 l/m^3	1,24 l/m^3
Air-entraining admixture	0,22 l/m^3	0,10 l/m^3

4. CONCLUSIONS

In Italy concrete road construction has a poor history and an unfavorable status compared with other materials.

The Italian ready mixed concrete industry is making efforts and investing substantial resources in promoting concrete as the most suitable material for road pavements especially for tunnels, where concrete's fundamental properties ensure the highest levels of safety and sustainability.

The successful "Quadrilatero" experience is a significant achievement for concrete producers. To date it is one of the most important public works in Italy and has received great media coverage. Therefore it might be a convincing way to spread the concrete industry message: "concrete in tunnels is the best for social, economic and environmental sustainability".

Figure 5: Concrete pavement in one of the maxilotto 1 tunnels

REFERENCES

[1] "*Directive 2004/54/CE on minimum safety requirements for tunnels in the Trans-European Road Network*", European Parliament and Council of the European Union, Official Journal of the European Union, 30 April 2004

[2] "*Pavimentazioni stradali in calcestruzzo. La scelta d'elezione per costruire un futuro sostenibile*", Aitec (Italian Technical Economic Association for Cement), Rome, April 2009

[3] "*Characterization of asphalt exposed to high temperature: application to fire case of asphalt pavement*", Noumoue A., Cergy Pontoise Univesity, 2003

[4] "*Aumentare la sicurezza in galleria*", Federbeton (Italian Federation for Cement and Reinforced Concrete), Roma, 2014

[5] "*Improving fire safety in tunnels: The concrete pavement solution*", BIBM – Cembureau - ERMCO, Brussels, April 2004

[6] "*Contribution of concrete pavements to the safety of tunnels in case of fire*", Eupave, Brussels, December 2010

[7] "*Rapporto 2015*", Atecap (Italian Technical Economic Association for Ready-mixed Concrete), Roma, March 2015

EXPOSED AGGREGATE CONCRETE SURFACES – DURABILITY, FUNCTIONALITY AND AESTHETICS

Aldona Wcisło

Lafarge Kruszywa i Beton Sp. z o.o.

Abstract

Concrete is valued by designers and architects, because of the virtually unlimited ways in which it can be used to design space - complicated structures, composition with other materials. In many cases concrete structural elements are left without additional finishing, which results in increased expectations regarding its visual appeal. The development of concrete technology brought about a number of ways for creation of decorative surfaces. One of the techniques is to shaping by incorporation / bond (chemical bonds retardation). The texture obtained in this way exposes the aggregate. Taking into account the availability of freeze-resistant aggregates in combination with dyes and cements, it is possible to create numerous types of surface concretes of a different colours and texture according to needs and requirements. Concrete surfaces in Poland are enjoying a growing interest and, due to their advantages, should become more and more popular, becoming a true supplement to the available technologies – asphalt, pavements carried out with stones etc. Concrete with exposed aggregate surface is characterized by features of the standard surface concrete with added decorative effect. Is the concrete with exposed aggregate relevant? Do the additional treatments to expose the aggregate from the surface discourage contractors? Will the product - exposed aggregate concrete - be attractive enough to excite architects with its decorative texture?

The paper is a trial approximation of the technology of surface forming by applying a setting time retardant, an indication of limits of material availability and a presentation of guidelines that should be valuable in production and processing. The presented paper aims to identify alternative solutions that, in the future, will enrich the portfolio of available technologies.

1. INTRODUCTION

Modern concrete surfaces should meet a number of requirements, concerning its: durability, safety, comfort of use, cost-effectiveness of construction, and impact on environment and surroundings. Concrete is a new way to look at material used for paving promenades, walkways, bicycle lanes, pedestrian passages, parking spaces, bus bays and even road surfaces. Exposed aggregate concrete is an interesting alternative and complement to the existing solutions such as: paving stones, blocks or asphalt. The visual effect, the resultant quality and durability of such surface may prove interesting and encourage realization of interesting projects.

2. SURFACE SHAPING

Chemical bond retardation is in most cases achieved by placing a surface deactivator on the top surface of a poured concrete mix and later removed of the non-bonded layer by water jet or brushing. If high-pressure washers are used - the nozzle should be directed at 45 degree angle to the surface at a distance of about 20 cm and pressure of 4-6 BAR. Washing off the top layer, depending on atmospheric conditions and agents used should begin between 6 and 24 hours from the moment the deactivator was placed.

The proposed finishing may only show the outline of the used aggregate, or if penetration is deeper, it may present up to 2/5 of the actual size of the used grade. The depth of deactivation depends on the used chemical agent. The other factors impacting the depth are also:

- the type of concrete used - with Portland cement the penetration is more shallow than with concretes with admixtures;
- quantity of concrete (the higher the concrete content the shallower the penetration);
- type and quantity of used mineral additions;
- the W/C ratio (reduction of the water quantity makes the removed layer thinner);
- admixtures impacting binding and heat treatment (decrease penetration depth);
- ambient temperature during application (penetration depth: in summer - decreased, in winter - increased)
- bleeding due to emission of water from the cement slurry or due to watery consistence, too much vibrations, or visible segregation (decreases penetration)."

It is important to select proper penetration depth in relation to the aggregate grade utilized in the cement mix. In order to ensure proper durability, the grains should be immersed to a level of at least 3/5 of their diameter. This prevents against individual grains falling out.

Aesthetic and interesting visual appeal is obtained by concretes utilizing mono-grade aggregate 0-2, 0-5, 2-5, 5-8, 8-11, 11-16 mm, however it is also possible to create surfaces with aggregate grades 2-8, 8-16, 2-16, 16-32 mm.

In order to avoid mistakes, the size and type of exposed aggregate should be well researched, as its size (quantity and size of grains) strongly impacts the visual appeal of the surface.

3. QUALITATIVE REQUIREMENTS FOR COMPONENTS OF EXPOSED AGGREGATE CONCRETE

Properly designed and created exposed aggregate surface has numerous advantages. Some of these include: good resistance to load transmission, high load-bearing, good adhesion, low abrasion (if proper aggregate is used), light colour (impacting the safety of vehicular movement and visibility), decrease in noise levels, good exploitation characteristics and low maintenance.

As exposed aggregate concrete may be used for pavements, ramps, pedestrian passages, promenades, parking spaces, bus bays and road surfaces, the components of the cement mix and the hardened concrete must meet a number of requirements, depending on their intended use.

Pavement concrete - the type used in construction of roads and communication infrastructure must be characterized by high quality, therefore it is designed using components of adequate quality. By using PN-EN 206-1 and assigning proper exposure classes to construction elements one can arrive at requirements concerning the minimal amount of cement, the W/C ratio, concrete class, air content, etc. Additionally, the designed cement mix must meet the requirements of the norm when selecting cement to a given concrete type and taking into consideration the realization of works, intended use for the concrete, maintenance conditions, size of the construction, environmental factors that the construction will have to withstand and potential reaction alkalies-aggregate. National reference documents - norms, ordinances and technical specifications - limit the ability for proper selection of cement. PN-EN 197-1, imposes usage of only CEM I low-alkali NA with determined mineral content. Another limitation is the assignment of cement strength class to a given concrete class.

When following the national requirements, in each and every case the surface concretes should be designed using cement CEM I NA, granite or basaltic grit of proper parameters, pit sand or river sand of determined through and characteristics, and a W/C ratio no lower than 0.5. Additionally, when using aerating admixtures there is also an requirement to provide additional aeration of the concrete mix (Table 1), K3 consistency. Hardened concrete should: be resistant to frost (F150), meet the absorbability to 5%, be watertight (W8), in many instances it is required to have tensile strength at bending from 4.5-5.5 MPa, as well as be freeze-thaw resistant in presence of salts (meeting requirements of class TF2) and have a proper pore microstructure ($A_{300} \geq 1.5$ %, $L \leq 0.200$mm)

3.1 Requirements for pedestrian passages and road surfaces for bike traffic

In case of use of a decorative concrete for surfaces for pedestrian and bike traffic we should relate requirements for components and the concrete itself to materials of similar use - i.e. setts, concrete paving slabs, guidelines for which are given in norm PN-EN 1338_2005 Betonowe kostki brukowe - Wymagania i metody badań[Concrete paving stones - Requirements and test methods], PN-EN 1339_2005 Betonowe płyty brukowe – Wymagania i metody badań [Concrete paving slabs - Requirements and test methods].

In the referenced norms, for every material intended for external pedestrian passages and areas of bike traffic, defined was freeze-thaw resistance in presence of de-icing salts, absorbability, resistance to bending, resistance to grinding, slippage resistance of unpolished surface.

Table 1: Examples of characteristics and requirements for aggregates for exposed aggregate concrete

Właściwości	Norma badania	Dobór	Wymagania/ Kategoria			
			Kruszywo drobne	Kruszywo grube		
			(*)	DWB	PWB	GWB(**)
Skład ziarnowy	PN-EN 933-1		G_F85	$G_C85/20$	$G_C90/15$	$G_C90/15$
Tolerancje uziarnienia	PN-EN 933-1	D/d < 4	-	G_T15	G_T15	G_T15
		D/d ≥ 4	-	$G_T17.5$	$G_T17.5$	$G_T17.5$
Zawartość pyłu	PN-EN 933-1	-	f_3	f_1	f_1	f_1
Kształt ziarn	PN-EN 933-4	-	-	SI_{50}/SI_{50}	SI_{20}/FI_{20}	SI_{15}/FI_{15}
Zawartość ziarn przekruszonych	PN-EN 933-5	-	-	-	$C_{90/1}$	$C_{100/0}$
Zawartość muszli w kruszywie grubym	PN EN 933-7	-	-	SC_{10}	SC_{10}	SC_{10}
Wskaźnik polerowalności	PN-EN 1097-8	-	-	-	PSV_{50}	PSV_{53}
Odporność na działanie mrozu w roztworze NaCl	EN 1367-6	-	-	≤5%	≤5%	≤5%
Reaktywność alkaliczna	PN-92/B-06714/46	-	stopień 0	stopień 0	stopień 0	stopień 0
Zanieczyszczenia organiczne	PN-EN 1744-1	-	$m_{LPC}0,25$	$m_{LPC}0,05$	$m_{LPC}0,05$	$m_{LPC}0,05$
Zawartość siarki całkowitej	PN-EN 1744-1	-	$S_{1,0}$	$S_{1,0}$	$S_{1,0}$	$S_{1,0}$

(*) wymagania kruszywa w odniesieniu do wszystkich rodzajów betonu objętych niniejszą ST
(**) kruszywo łamane uzyskiwane wyłącznie ze skały litej

3.2 Requirements for surfaces of cement concrete

When analyzing the road surfaces, the exposed aggregate concrete should meet the requirements concerning materials and hardened concrete in the PN-EN 13,877 Nawierzchnie betonowe [Concrete surfaces], or PN-75/S-96015 PN-75/S-96015 Drogowe i lotniskowe nawierzchnie z betonu cementowego [Roads and airports surfaces from cement concrete] .
Surface concrete should undergo freeze-thaw testing, and its freeze resistance in presence of de-icing salts should be classified according to prEN 12390-9. Another important characteristic is bending resistance at stretching. According to the Polish norms in force, the specification will impose detailed requirements concerning the utilized: cements, aggregates, absorbability, water-tightness, freeze resistance (according to PN-88/B-06250)

4. DESIGNING OF SPACES INTENDED FOR PEDESTRIAN TRAFFIC

Rules for designing exposed aggregate surfaces are similar to the ones for standard concretes. In order to achieve the required visual/aesthetic effect of proper durability, from the designed space should be separated spaces of size 25 m^2 – 50 m^2, where the length/width ratio will be 1 – 2.5 (D/S< 1÷2.5), and the length of the designed surface cannot be larger than 25 times its thickness (Figure 1).

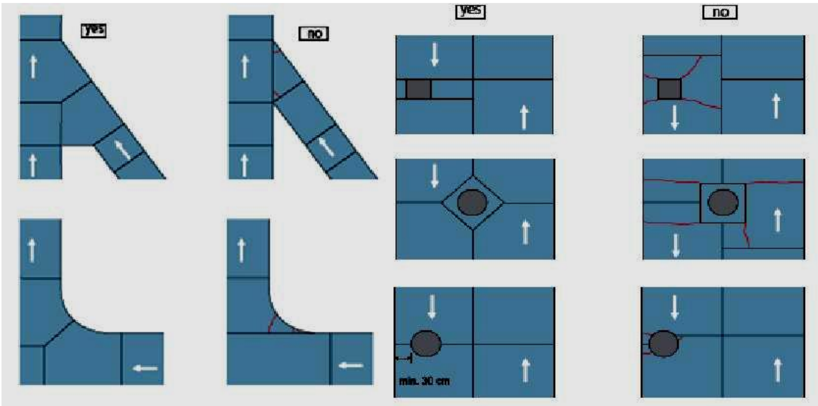


Figure 1: Examples of joint systems and limiting walls

4.1 Relations between dimensions of the designed surface

In order to properly place the concrete mix, barriers should be placed to limit the spilling in form of plastic dilatation barriers, bricks, setts, wooden logs, etc. to match the concrete surface (Photo.1).

Photo 1: Examples of dilatation barriers

Depending on the amount of the traffic, the soil conditions and the type of soil , the thickness of the poured layer is designed. Concrete surfaces intended for pedestrian and low vehicular traffic may be of thickness lower than 17-27 (as given in the catalogue of rigid surfaces). In order to ensure proper durability of a concrete surface the dilatation gaps have to be properly planned. Cracks or self-dilatation of concrete is a natural phenomenon**.** By introduction of joints the cracks appear in advance and in a controlled manner. Joints lessen effects of concrete shrinkage and allow for movement of separated concrete slabs - less stress is generated. Proper design of joints impacts the durability of the designed surface (Figure 2). The dilatations should also be created within the permanent obstacles and aim at surface shape devoid of sharp edges.

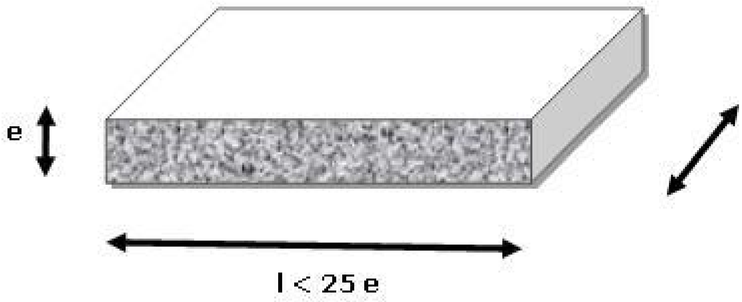


Figure 2: Relations between dimensions of the designed surface

4.2 Examples of joint systems and limiting walls

Exposed aggregate concrete, depending on the type of access to its final location may be designed in two ways: as pumpable concrete, focusing on consistence at expense of the visual effect or non-pumped which results in better dispersion of aggregate. Before unloading, the substrate should be moistened, nets should be placed (if needed), the surrounding surfaces are to be secured, and a 2% decline of the designed surfaces.

5. DESIGNING THE CEMENT MIX FOR EXPOSED AGGREGATE

In exposed aggregate concretes the composition of components is the most crucial factor determining the final look of the surface. The indicators to be taken into consideration are the type and amount of traffic, the aesthetic effect, acoustic characteristics and durability.

5.1 Coarse aggregate

When the used technology results in individual grains being exposed, additional attention should be given to selection of aggregate that not only meets the quality requirements but is also characterized by proper grain size and shape. It is crucial to select a mono-grade aggregate 2-5, 5-8, 8-11 mm (road grade), 2-8 mm (in case of aggregate for standard concrete), or 2-16, 8-16, 16-22, 16-32 mm. A significant risk when utilizing discontinuous grade curves, i.e. 0-2 plus 8-16 is exposure of too much slurry, making the overall visual effect less appealing. Grain size up to 8 mm is recommended for surfaces where the key aspect, apart the quality requirements, is noise reduction. Due to high cost of mixture intended for exposed aggregate surfaces, the IFSTTAR Technology Centre has presented a concept of placing only a 3-4 cm layer on aggregate up to 8 mm with exposed aggregate, wet poured onto concrete that meets the durability requirements but not the aesthetic ones. Utilization of aggregate above 8 mm makes the surfaces noisier which limits its uses to access roads and private residences. In order to properly design the aggregate composition, a freeze-resistant aggregate should be used (gravel or grit depending on use), with high resistance to grinding, mono-grade, with continuous grain size in quantity of 1100 - 1350 kg/m^3.

5.2 Sand

The sand used should be free of organic contaminants, with continuous grain size in order for the mortar to be as uniform as possible. The chosen material should meet the quality requirements (pit sand is most commonly used). Its content in the mix should be 550-700 kg/m^3 and usually depends on the required visual effect.

5.3 Cement

As in case of most surfaces, the specification and the ordinances in force impose utilization of CEM I low-alkali or, under special circumstances, CEM III/A low-alkali and strength class 42.5. The cement should be chosen in such a way as to meet the requirements concerning colour, when using a coloured mortar in the mix. The quantity of the cement is strongly dependent on the specified W/C or C/P ratio and the durability requirements for the surface concretes. Due to visual appearance it should not go above 350-380 kg/m^3. In case of aggregate with grades up to 8 mm the content should be increased to 420 – 425 kg/m^3.

5.4 Water

Tap water should be used. Every supplied batch of concrete mix should be characterized by the same W/C ratio. It is not recommended to use recycled water in production. The deeper the aggregate exposure, the lower the W/C ratio of the mortar must be.

5.5 Additions and admixtures

In order to achieve a mix of proper characteristics meeting requirements concerning freeze-resistance (F150), freeze-resistance in presence of de-icing salts (FT2), absorptivity (up to 5%), water-tightness (w8), flexural strength (4.5 - 5.5MPa), and proper macrostructure of air pores used should be mixes for: plasticization, flow, aeration. In some cases for achieving concrete of lower porosity probable is utilization of metakaolin.

An extremely important aspect when producing exposed aggregate concrete is proper dispersion of aggregate grains. To avoid segregation, bleeding and shrinkage - plastic cracks, synthetic fibres should be used (polypropylene). In order to increase the visual appeal of the surface the mortar may be coloured, using the dyes available in the marketplace. The visual effect will be increased if the colour of the mortar is selected to march the utilized aggregate. Every batch of concrete mix created should be characterized by the same W/C in order to keep the colour similar. Selection of quantity and form of the chosen dye depends on its content and intensity of dyeing, it is recommended not to use more than 6% of the cement's mass. When using dyes their impact on the W/C ratio and freeze-resistance should be tested.

5.6 Deactivator - chemical retardation of bonding

The retarding agent is a special layer - in form of a paste, lacquer, liquid or sheets of infused paper, that, when placed on the surface of concrete mix just after smoothing, penetrates the mix to a given, determined depth. The main objective of the deactivator is delaying or stopping the bonding process in the outer layer of the concrete mix so as to expose the aggregate when removing the top unbound layer.

During the second phase the deactivator works as a finishing-protecting layer that, during the first 24 hours, keeps 80% of water in the mix. It secures the surface against rain and later acts as a protective layer during maintenance.

Depending on the type and origin of the deactivator used it should be placed and washed off at different times. In most cases it is available as water solution that should be well stirred before use and then placed on the smoothed surface in form of a mist, covering all the surface. The placement may be realized using a garden sprayer or a different device used to spray liquid under pressure. For practical reasons, deactivators of different penetration depths have different colours.

Table 2: Examples of deactivators

<table>
<tr><th>Recommended amounts of the components in the mixture</th><th>Max size of aggregate [mm]</th><th colspan="6">Deactivators</th></tr>
<tr><td rowspan="3">350 kg cement, 1200 kg sand, 600 kg aggregate</td><td>4</td><td rowspan="5">Green – Sanding effect</td><td rowspan="5"></td><td rowspan="6"></td><td rowspan="8"></td><td rowspan="10"></td><td rowspan="12"></td></tr>
<tr><td>6</td></tr>
<tr><td>8</td></tr>
<tr><td rowspan="2">350 kg cement, 900 kg sand, 900 kg aggregate</td><td>4</td></tr>
<tr><td>6</td></tr>
<tr><td rowspan="7">350 kg cement, 600-700 kg sand, 1100-1200kg aggregate</td><td>4</td><td rowspan="10"></td><td rowspan="4">Blue – fine wash out</td></tr>
<tr><td>6</td><td rowspan="4">Violett – light and medium wash out</td></tr>
<tr><td>8</td></tr>
<tr><td>10</td><td rowspan="4">Yellow – medium wash out</td></tr>
<tr><td>12</td><td rowspan="6"></td></tr>
<tr><td>14</td><td rowspan="5"></td><td rowspan="3">Pink – strong wash out</td></tr>
<tr><td>16</td></tr>
<tr><td rowspan="3">330 kg cement, 500-600 kg sand, aggregate</td><td>20</td><td rowspan="3"></td><td rowspan="3">Grey – deep wash out</td></tr>
<tr><td>25</td><td rowspan="2"></td></tr>
<tr><td>30</td></tr>
</table>

6. PROCEDURE FOR SURFACE PROCESSING OF PAVEMENT CONCRETE

The technology of processing the surface by using a retarding agent, which allows for uncovering the aggregate hidden in the concrete is highly labour-intensive, requires high degree of precision and work quality. It is a time consuming, expensive solution that nevertheless provides a lot of satisfaction from the finished product. When creating an exposed aggregate concrete surface the indicated procedure should be closely followed.

- **Production of concrete mix and quality control**;

Photo 2: Production of concrete mix, transport and control

- **Determination of proper concrete pouring zones**
- **Impregnation of limiting surfaces, securing of surrounding surfaces**

Photo 3: Securing of auxiliary surfaces

- **Moistening the substrate and placement of reinforcement nets**
- **Pouring of concrete mix**

Photo 4: Pumping of concrete mix

- **Creation of reference surfaces**
- **Spreading of concrete mix**

Photo 5: Spreading of mix

- **Smoothing of poured concrete mix**
- **Placement of retarding agent**

Photo 6: Spraying of deactivator

Photo 7: Deactivated surface

- **Washing off the retardant film by jet stream**

Photo 8: Washing off

- **Dilatation of resultant surface**
- **Surface protection by impregnating agents**

Photo 9: Washed-off surface on aggregate up to 16 mm (granite grit) and brown-coloured mortar.

Photo 10: Washed-off surface on aggregate 8-16 mm (gravel) and black colouring.

Photo 11: Washed-off surface on aggregate w-8 mm (gravel), grey colouring

Photo 12: Washed-off surface on aggregate 2-8 mm (granite grit) and non-coloured mortar

Photo 13:. Washed-off surface on aggregate 2-16 mm (granite grit) and black-coloured mortar

7. CONCLUSIONS

Concrete surfaces are not prone to deformations such as: ruts and pits. This results in lower repair expenditures and thus increases safety and driving comfort. Surface concrete, depending on colouring of the mortar, may be a surface that is characterized by better visibility during the night hours. Thanks to this feature, the expenses for lighting of such spaces may be lower. It also decreases the amount of heat emitted to the atmosphere due to lower heat accumulation in comparison to asphalt surfaces. When analyzing concrete surfaces from the life-cycle standpoint we have to agree that in relation to the asphalt surfaces, concrete is characterized by lower level of depletion of natural resources and lesser environmental impact. To a lesser degree than asphalt roads, it adds to global warming, acidification, eutrophication, photo-chemical creation of ozone, decrease in ozone layer, human environment and quality of the natural environment. When comparing costs of construction and repair of concrete and asphalt surfaces during 30 years of exploitation, the total cost in case of concrete roads is lower.

Apart all the benefits listed above, there is an additional one that the surfaces of exposed aggregate permanently increase the roughness of surface by strengthening its macro texture. Additionally, when aggregate of up to 8 mm is used the level of noise is reduced. Taking into consideration the ability to use concrete with exposed aggregate when creating local roads, pedestrian passages, parking spaces, where the visual effect is important, it can serve as a great replacement or supplement to the previously used surfaces produced of fine elements.

A wide selection of available aggregates and dyes as well as possibilities to create surfaces from elements of larger areas allows architects for much more freedom in design.

Proper realization of surface utilizing exposed aggregate concrete allows for a range of uses not only due to its durability but also its aesthetic characteristics.

REFERANCES

[1] A.Szydło, P.Mackiewicz „Nawierzchnie betonowe na drogach gminnych – poradnik"

[2] K.Kuniczuk „Beton architektoniczny wytyczne techniczne"

[3] PN-EN 13877-1 Nawierzchnie betonowe, Część 1: Materials:

[4] PN-EN 13877-2 Nawierzchnie betonowe, Część 2: Wymagania funkcjonalne dla nawierzchni betonowych

[5] PN-75/S-96015 Drogowe i lotniskowe nawierzchnie z betonu cementowego

[6] PN-EN 1338_2005 Betonowe kostki brukowe – Wymagania i metody badań,

[7] PN-EN 1339_2005 Betonowe płyty brukowe – Wymagania i metody badań.

[8] Ir A.Jasienski Expose aggregate concrete for the renovation of public spacer

[9]Exposed-aggregate, Finishes for Flatwork, Briefing 02, Cement Concrete&Aggregates Australia, 2007

[10] Residential Concrete Driveways and Paths, Cement Concrete&Aggregates Australia, 2006

[11] IB 18 Architectural Surface Finishes, CCANZ

[12] I.Ruttmar A2 Nowy Tomyśl-Świecko, Innowacyjna technologia wykonywania nawierzchni

[13] Z.Giergiczny Zalety betonu jako materiału budowlanego w drogownictwie

[14] BTA styczeń-marzec 2009, A.Szydło „Badanie trwałości zmęczeniowej nawierzchni betonowej autostrady A4"

[15] BTA styczeń-marzec 2010, A.Karbowski „Emisja hałasu nawierzchni dróg-drogi betonowe"

[16] Materiały Budowlane 11'2011, W.BAńskowski 'Ciche nawierzchnie drogowe na świecie"

[17] BTA styczeń-marzec 2010, A.Karbowski „Dlaczego warto budować betonowe drogi w miastach"

[18] BTA kwiecień-czerwiec 2011, Ł.Szabat „Drogi betonowe a drogi asfaltowe w ujęciu metody oceny cyklu życia"

[19] BTA styczeń-marzec 2011, O.Kamiński „Szczeliny w nawierzchniach betonowych"

[20] BTA październik-grudzień 2011, D.Bebłacz „Wymagania jakościowe stawiane składnikom betonu drogowego.

CONCRETE ROAD CONSTRUCTION IN ESKISEHIR

Fulya Gülen (1), Erman Derin (1), Gökhan Arıkoglu (1) and Önder Kırca (1)

(1) Çimsa Çimento San. ve Tic. A.Ş., Mersin, Turkey

Abstract

In this study, the design of concrete road in 2 km length and in 40.000 m^2 area constructed in Eskişehir were discussed. In this project, the type of road was preferred concrete road by virtue of being more suitable for heavy load traffic, weather and ground conditions. For this purpose, the concrete road was designed by considering the characteristics of long service life, driving safety and comfort, low fuel consumption, the environmental compliance, the availability of waste materials, first manufacturing costs, total costs along the service period and, low maintenance and repair costs. Two concrete mixtures for concrete road were selected according to different combinations of laboratory and industrial trials. The use of Dowel bars, shear reinforcement, to reduce deflections between concrete roads separated by joints, tie bar, connecting roads, to connect each concrete road produced separately by finisher, prevent strip sliding, and provide a stable longitudinal joint was decided.

Keywords: Concrete road, tie bar, dowel bar

1. INTRODUCTION

The aim of the project was to obtain the transition of raw material vehicles, cement silo buses, trailer trucks carrying packed cement, service buses, coal trucks and concrete mixers from the north of the Eskişehir-Bursa highway to the south of the highway.

The concrete road construction, in Eskişehir, involves an underpass, a rural roundabout, 6.200 m^2 truck and car parking area in capacity of 63 vehicles. The length of the road is 2 km in total area of 40.000 m^2. The construction area had been used as a rubble field until this project. Another feature of this project was cleaning the environment from the rubbles. It was also an environmental friendly project due to the use of the remaining rubble as a raw material in cement production.

In this project, the type of road was preferred concrete road by virtue of being more suitable for heavy load traffic, weather and ground conditions. For this purpose, the concrete road was designed by considering the characteristics of long service life, driving safety and comfort, low fuel consumption, the environmental compliance, the availability of waste materials, first manufacturing costs, total costs along the service period and, low maintenance and repair costs.

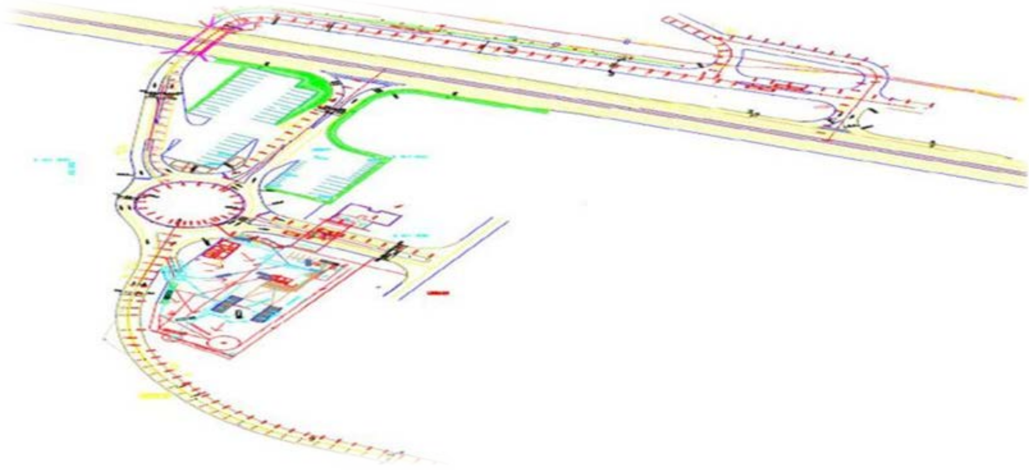


Figure 1: The plan of the concrete road construction

2. INFRASTRUCTURE OF THE CONCRETE ROAD (GROUND SURVEYS)

The following study was conducted on the sub-base thickness and on the material selection.

8 trial pits in 3 meters depth were opened in every 250 meters along the 2 km road. Disturbed and undisturbed soil samples were taken from these pits.

The water content, sieve analysis, Atterberg limits, California bearing ratio (CBR), proctor, consolidation, tri axial compression (UU) and plate bearing tests were carried out on the soil to determine ground classification, compression values, bearing capacity and the cut-off thickness.

The geotechnical characteristic of the ground was determined as CH (high plasticity clay), CL (low plasticity clay), SP-SM (poorly graded silt with sand). As an average of the units, 9% of gravel, 28% of sand, 63% of clay-silt; 9% of water content value were determined. The

Liquid Limit value in the clayey levels was 53%, the Plastic Limit value was identified as 25% and the Plasticity Index was identified as 28%.

Based on this assessment, the thickness of the concrete, the protective layer and the base layer thickness were determined.

3. DESIGN OF THE ROAD SECTION

The section of the road was prepared according to the ground survey results, climatic conditions and the traffic loads. The design life of this road was chosen 30 years.

3.1 Joints

Concrete joint depth was selected 1/3 of the pavement thickness. (T=30/3=10 cm)

3.2 Dowel bars

38 mm Dowel bar diameter, 400 mm length and 300 mm interspace between each bar was selected.

3.3 Tie bars

16 mm Tie bar diameter, 800 mm length and 750 mm interspace between each bar was selected.

3.4 Lean concrete

The thickness of lean concrete was selected 15 cm to prevent the pumping effect of the ground water and the loss of the sub-base ground. Lean concrete also supply to bear up the thickness of the concrete.

3.5 Joint spacing

The distance of between the joints was calculated 4.3m in order to minimize the deflection and distortion effects.

3.6 Geotextile

Geo textile was used between top pavement and lean concrete.

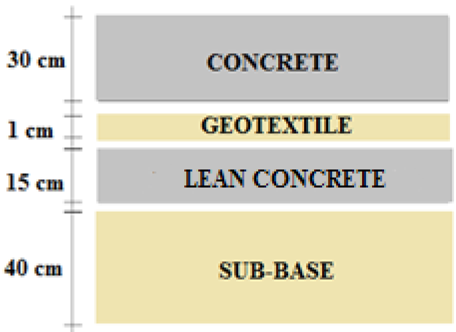


Figure 2: Section of the pavement

4. DRAINAGE SYSTEM

In this project, drainage channels, collecting pools were made to improve the ground and to equip for manufacturing in the area existing ground water and surface water. The drainage pipes were placed both to the left and to the right side of the road.

Picture 1: The drainage system

5. GROUND IMPROVEMENT AND SUB-BASE LAYING

In some areas, cutting and filling works were carried out to improve the ground.

Picture 2: Ground improvement and Nuclear Troxler test

The most suitable filling material was selected after testing the samples in the laboratory. Then the compressibility tests of the material were made by modified proctor test method. Mechanical material was laid in 30 cm layers and compressed. Nuclear Troxler and sand cone tests were carried out on the pressed ground. In the area, soil compaction was controlled by an electronic device per 250 m^2 and sand cone test per 500 m^2. According to the result of compaction, other layers were enabled.

Picture 3: Sand cone test

Picture 4: Protecting the sub-base from rain

MC30 material, a kind of bitumen, was laid to protect the compressed ground from the rain.

6. CONCRETE DESIGN

Portland cement, CEM I 42.5 R conformed to EN 197-1 was obtained from Çimsa Eskişehir Cement Factory. A fly ash conformed to EN 450-1 was used to control heat of hydration and and to refine durability.

Table 1: Physical Properties of Portland Cement

Property	
Specific Gravity	3,11
Specific Surface Area (cm^2/gr)	3210
Time of Setting (min)	
Initial	210
Final	240
Water (%)	30,1
90 micron (%)	0,6
45 micron (%)	6,3
Compressive Strength (MPa)	
2 days	26,7
7 days	40,0
28 days	52,0

Table 2: Chemical Composition of Portland Cement

Oxides	**%**	**Oxides**	**%**
CaO	61,00	SO_3	3,40
SiO_2	19,18	K_2O	0,89
Al_2O_3	5,32	Na_2O	0,33
Fe_2O_3	2,63	LOI	3,24
MgO	2,33	Insoluble residue	0,50

Table 3: Chemical Composition of Mineral Admixtures

Oxides	Fly Ash (%)
CaO	3,42
SiO_2	58,96
Al_2O_3	22,67
Fe_2O_3	6,42
MgO	1,69
SO_3	0,21
K_2O	1,38
Na_2O	0,71
TiO_2	1,33
Mn_2O_3	0,08
LOI	2,23

Table 4: Physical Properties of Mineral Admixtures

Property	Fly Ash
Specific Gravity	2,25
Specific Surface Area (cm^2/gr)	4510
Activity Index	%75 CEM I 42,5 R, %25 Fly Ash
7 days (%)	73,1
28 days (%)	77,4

Table 5: Concrete Mix Designs

Material	Mix Design-1 (kg/m^3)	Mix Design-2 (kg/m^3)
Cement	350	325
Fly Ash	75	100
Water	172	180
0-5mm Crushed Sand	166 (%10)	785 (%46)
0-7mm C. Sand	590 (%34)	0
7-15mm Crushed Stone	405 (%24)	370 (%22)
15-22mm C. Stone	549 (%32)	0
15-38mm C. Stone	0	523 (%32)
Air-Entraining Admixture	0,18%	0,19%
Slump Value	2-4cm	2-4cm

7. EXPERIENCES OF THE TEST ROAD

Previously, trial production was carried out on 50 m length of the road to observe the adequacy of the rate of the production and the ready mix concrete plant industrially.

Picture 5: Trial production

The most important fresh concrete property was consistency and air content of the concrete in this project. The strength of the concrete was tested per 50-100 m^3 production. The average compressive strength of concrete was 39.6 MPa and the standard deviation was 1.9. The average flexural strength of the concrete was 4.2 MPa.

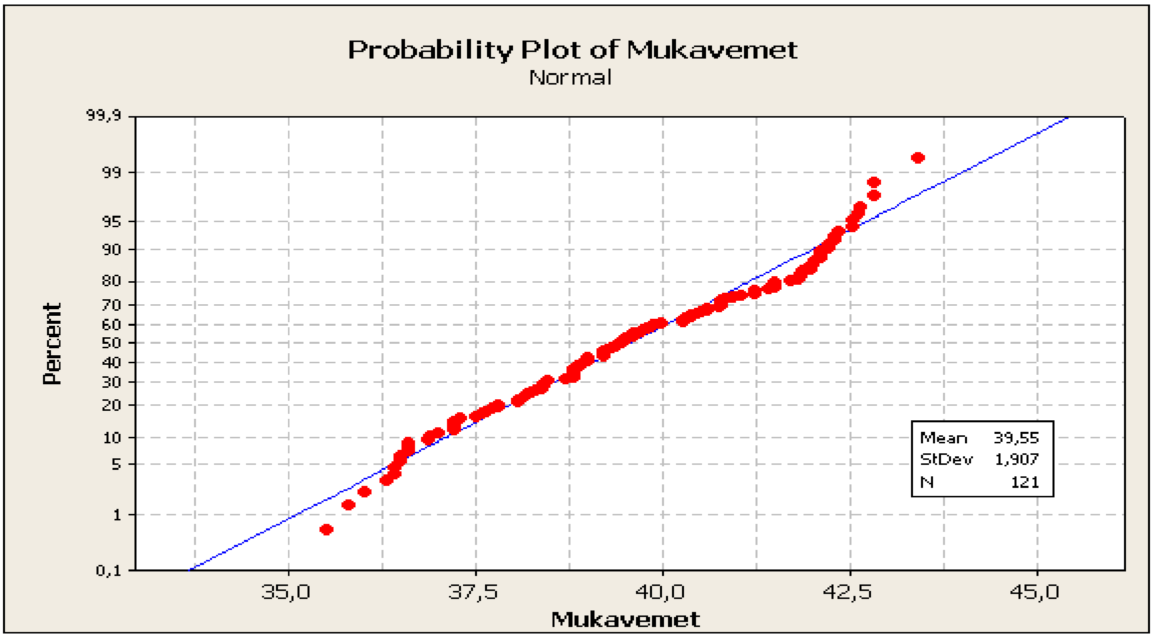
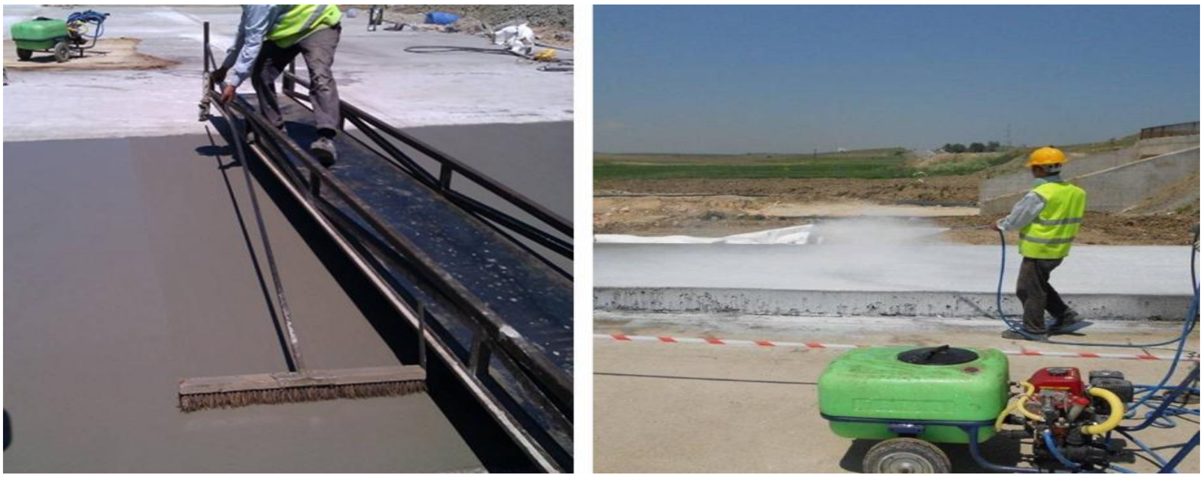


Figure 3: Distribution of compressive strength of the concrete

Picture 6: Spreading the geotextile and placing the Dowel bars

Picture 7: Fixing the Dowel bars and spreading the main concrete layer

Picture 8: The application of finisher



Picture 9: The appearance design and the curing of the concrete

Picture 10: Cutting and curing the joints

Picture 11: Curing the concrete and Tie-bar application

DEVELOPMENT OF A POLYMER-MODIFIED LIGHTWEIGHT MORTAR FOR THE REPAIR OF EXPOSED CONCRETE FACADES ON HISTORIC BUILDINGS

Murat Ünal (1), Götz Hüsken (1) and Hans-Carsten Kühne (1)

(1) BAM Federal Institute for Materials Research and Testing, Germany

Abstract

Reinforced concrete buildings are frequently damaged by cracks, spalling and corroded reinforcement bars. The present article focuses on the damage analysis of one of the first buildings in Berlin that was constructed using reinforced lightweight concrete, which makes it historically important. The compressive strength, carbonation depth, location and diameter of reinforcement bars as well as moisture and salt profiles were determined. The carbonation-induced corrosion of the reinforcement bars represented the main damage of the building. Certified repair mortars with lightweight aggregates (LWA) do not exist on the German market. Therefore, an appropriate repair mortar was developed in accordance with DIN EN 1504 and the German guideline for the repair and protection of concrete structures (RL-SIB).

On the one hand, historic preservation requirements, such as: colour of the mortar, possibilities of surface patterning, maximum preservation of the original substance and reversibility of any intervention must be met. On the other hand, technical requirements of the relevant standards must be achieved. The developed mix was designed to resist the identified causes of deterioration. Thermal strains within the repair mortar and between the repair mortar and the substrate influence the crack formation and thus the durability. Therefore, special attention was paid to the bonding and shrinking behaviour of the concrete repair system. By using polymers, both hardened mortar properties and durability were influenced positively.

Keywords: Lightweight concrete, repair mortar, historic building, architectural use, polymers, shrinkage

1. INTRODUCTION

The history of the lightweight concrete began about 2,000 years ago, when the Roman architects built the dome of the Pantheon in Rome in Opus Caementitium. They applied Roman concrete with a density of 1350-1750 kg/m³ [1]. Even today, the lightweight concrete is used in different types on a wide scale. Lightweight concretes can either be lightweight aggregate (LWA) concrete, foamed concrete or autoclaved aerated concrete (AAC). Such lightweight concrete blocks are often used in house construction as they provide an excellent thermal insulation and a healthy indoor climate. The present article focuses on the damage analysis of a historically building that was constructed using structural lightweight concrete. The special feature of this building is the fair-faced concrete façade. It is designed as board formed concrete, which is the name for a process that leaves an imprint of the wood formwork on the final surface of the concrete. Considering the special building situation and existing damages, a repair mortar with adapted mechanical, chemical and optical characteristics (colour, surface pattern) was developed adapting the requirements of the German guideline for the repair and protection of concrete structures (Richtlinie Schutz und Instandsetzung von Betonbauteilen – RL-SIB)

Figure1: Listed Building made of structural lightweight concrete

2. STRUCTURAL INSPECTION

A professional planning forms the basis of a durable repair concept. The determination of both the damage and the causes of damage are required for further steps. Therefore, destructive and non-destructive testing methods were applied for the structural inspection. In listed buildings, non-destructive test methods should have priority. Therefore, after the visual inspection, first the concrete cover, location and diameter of reinforcement bars and capillary water absorption by Karsten were determined. Furthermore, a potential mapping was performed to detect possible corrosion spots. However, the non-destructive testing must be validated and complemented with destructive methods such as determination of compressive strength, density, carbonation depth, identification of concrete composition, as well as moisture and salt profiles. The results of the destructive and non-destructive testing are summarized in Table 1.

Table 1: Results of the building inspection [4]

Test methods	Standard/Guideline	Minimum	Maximum
Compressive strength (cores Ø 50 mm)	DIN EN 12504-1 DIN EN 12390-3	34,3 N/mm²	45,0 N/mm²
Concrete density	DIN EN 12390-7	1,47 kg/dm³	2,19 kg/dm³
Concrete cover (Profometer 5)	-	23 mm	58 mm
Carbonation depth	DIN EN 14630-1	2 mm	40 mm
Surface tensile strength	RL-SIB	1,36 N/mm²	3,07 N/mm²
Chloride content*	Issue 401 DAfStb	0,03% by mass	0,48% by mass
Capillary water absorption by Karsten test tubes	-	0 cm³/min	0,027 cm³/min
Moisture content (drilled concrete dust)	DIN EN ISO 12570	2,49% by mass	10,67% by mass

*chloride content in % by mass, based on 15% by mass cement content in the concrete

The lightweight concrete of the examined building has a maximum aggregate size of d_{max} = 16 mm and the grading curve matches the grading curve B16 according to DIN 1045-2. Expanded clay was used as coarse aggregate. Based on the compressive strength results, the substrate can be classified according to EN 13791 as LC 35/38 and density class D1,6 according to EN 206.

As shown in Figures 2a-c the carbonation-induced corrosion of the reinforcement bars, spalling concrete and rock pockets are the major damages. The most severe damages occur due to inadequate local concrete covering.

Figure 2: a) corroded reinforcement; b) rock pockets; c) cracks, spalling concrete

3. MORTAR DEVELOPMENT

Any repair work must be tailored to the building and the deterioration of the building. In Germany, there is no repair mortar available that uses lightweight aggregates. Therefore, a suitable polymer modified repair mortar (PCC) with LWA (expanded clay) was developed that meets the requirements of DIN EN 1504 [2] and the German guideline for the repair and protection of concrete structures (RL-SIB) [3]. Furthermore, the fresh mortar properties have to be adapted to the needs of the restorers. Considering both requirements defined by the relevant standards and practical needs determined by the architect and the restorer, the following requirements were defined:

- Applicability on vertical surfaces
- High plasticity and good modelling behaviour
- Long workability time
- Good bonding properties

The special feature of the developed mortar is the use of expanded clay which is a porous material with high water absorption. This factor must be considered in the mix design. Depending on the water absorption, the required quantity of absorbed water must be added to the mixing water on site in order to meet the water requirement of the mix design. In the laboratory, the amount of absorbed water (8.5% by dry mass of expanded clay) was added to the expanded clay. After 10 minutes, the remaining ingredients were added to the prewetted LWA and mixed. The consistence was assessed using the flow table test according to Hägermann (DIN EN 1015-3). The range for the aimed consistency was set to a spread of 130 –140 mm.

Besides requirements on the fresh mortar, further requirements on the hardened mortar were defined that are mainly based on requirements given in RL-SIB as well as the material properties of the lightweight concrete of the existing building. The following parameters were defined for the present case:

- Medium mechanical strength that is comparable to the strength of the substrate (compressive strength of about 35 N/mm²)
- Low shrinkage
- High carbonation resistance
- High frost resistance

3.1 Mix design

In the first instance, a basic mix design M1 was developed that meets the required fresh and hardened mortar properties as described before. The mix design of the basic mix M1 is based on the idea of optimized particle packing, as the fresh and hardened mortar properties are affected substantially by the grading of the designed mix. The mix design tool developed by [5] was used to optimize the grading of the developed repair mortar. This mix design tool considers the entire grading of all solid ingredients to find the optimum composition of aggregates, binders, and filler materials. In the next step, the shrinkage of the repair mortar was optimized to prevent large deformations of the repair mortar which will show a negative effect on the adhesion strength and the durability of the repair mortar. The shrinkage of cementitious materials can be influenced by the mix design and the addition of concrete admixtures. For this purpose, the influence as well as the interaction of a shrinkage reducing admixture (SRA) and ethylene-vinyl acetate copolymers (EVA) was investigated in detail. The EVA was used in concentrations of 1.5 and 3.0% based on the weight of cement (mix M2 and M3). The SRA was added in concentrations of 4.0 and 6.0 % based on the weight of cement (mix M4 and M5). The final mix design M6 contains a combination of both concrete admixtures. Both SRA and EVA were added as powder. The remaining ingredients, such as cement, fly ash and limestone powder were kept constant. The water-cement ratio was 0.4 in all mixtures (see Table 2).

The shrinkage measurements were performed according to the specifications given in DIN 52450. Mortar prisms with a dimension of 40 x 40 x 160 mm³ were cast with measuring taps type 1. The cast samples were stored at 95% relative humidity for 24 hours. After 24 hours, the prisms were stripped from the mould and stored at standard climate (23 ± 2) °C, (50 ± 2) % relative humidity. Shrinkage measurements were performed up to 91 days.

Table 2: Mix designs of mortars used for shrinkage measurements

	M1	M2	M3	M4	M5	M6
Cement type	CEM I 42,5 N					
Cement content [kg/m³]	500.0	500.0	500.0	500.0	500.0	500.0
Limestone powder [kg/m³]	40.0	40.0	40.0	40.0	40.0	40.0
Fly ash [kg/m³]	60.0	60.0	60.0	60.0	60.0	60.0
EVA [% bwoc]	-	1.5	3.0	-	-	1.5
LWA [kg/m³]	227.6	227.6	227.6	227.6	227.6	227.6
Sand (0-1) [kg/m³]	798.2	798.2	798.2	798.2	798.2	798.2
SRA [% bwoc]	-	-	-	4.0	6.0	6.0

bwoc: Based on the weight of cement

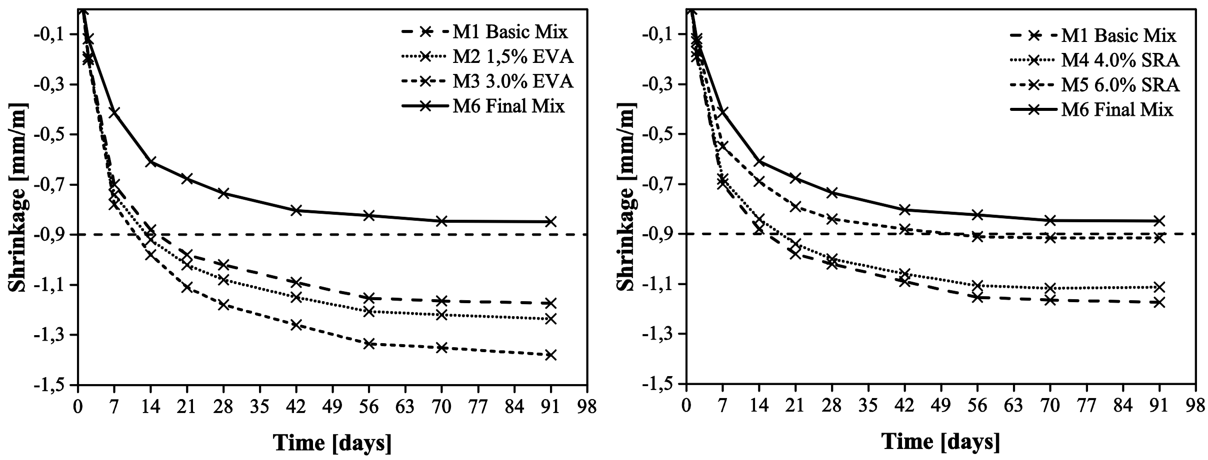


Figure 3: a) measured shrinkage for varying concentrations of the copolymer used;
b) measured shrinkage for varying concentrations of the shrinkage reducing admixture used

Based on the results of the optimized grading of mix M1 and the results of the shrinkage optimization the mix design of the final mix was developed (see Table 3 and Figure 4). The developed PCC lightweight mortar was subjected to tests required by RL-SIB that are part of part of an initial assessment. The results of the corresponding tests are discussed in the following sections.

Table 3: Mix proportioning per m³ mortar of the final mix

Material	Mass [kg]
Cement	485.2
Limestone powder	38.8
Fly ash	58.2
Sand 0.1/0.5	570.5
Sand 0.5/1.0	204.0
LWA 1.0/4.0	220.8
Plasticizer	5.8
EVA	7.3
SRA	29.1
Water	194.1
Absorbed water	18.77
Total	1832.6

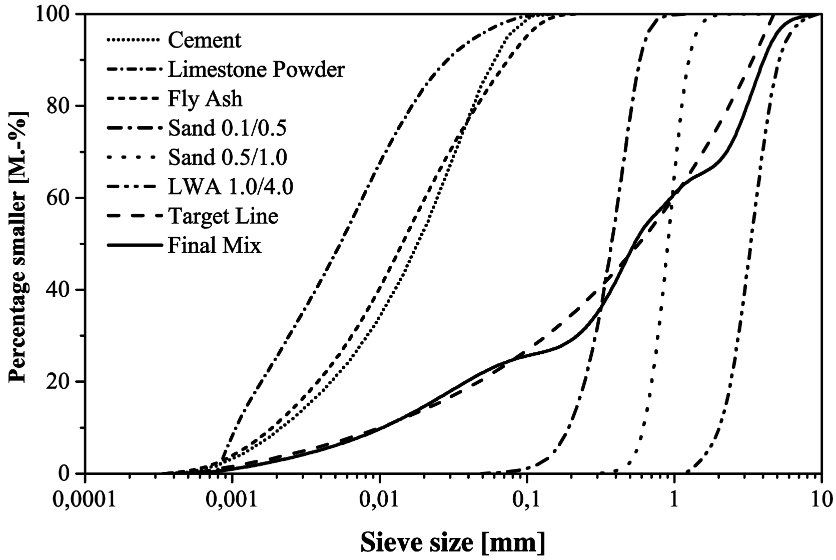


Figure 4: Grading of the materials used, target line and final mix

3.2 Fresh mortar properties

There are no quantitative requirements on the fresh mortar properties described in RL-SIB, as the consistency of the repair mortar and related fresh mortar properties depend mainly on the needs of restorers and the building situation. However, some basic properties of the fresh mortar were determined and are listed in Table 4.

Table 4: Comparative overview of fresh mortar properties of a M2 mortar according to RL-SIB and the developed PCC lightweight mortar

Property	Requirement RL-SIB	Actual value
Consistency (Flow table test)	-[1]	136 mm
Air void content	-[1]	7,9 Vol.-%
Fresh mortar density	-[1]	1,79 kg/dm³

[1:] No quantitive requirements imposed

3.3 Hardened mortar properties

The requirements of the RL-SIB to the hardened concrete are aiming for a durable repair. However, these requirements apply only for the repair of conventional concrete with standard aggregates. In the present case, LWAs were used in the repair mortar in order to meet the defined requirements. Furthermore, a main task of the mortar development was to adapt the strength of the repair mortar to the properties of the substrate concrete in order to avoid shell-like flaking that reduces the durability of the concrete repair. Both compressive strength and modulus of elasticity are lower for lightweight concrete than for conventional structural concrete. Therefore, not all requirements listed in Table 5 and that are described in RL-SIB are fulfilled.

Table 5: Comparative overview of hardened mortar properties of a M2 mortar according to RL SIB and the developed PCC lightweight mortar

Property	Requirement RL-SIB	Actual value
Hardened mortar density	-[1]	1,70 kg/dm³
Flexural tensile and compressive strength (Storage A)	$ß_{FT,90} \geq 0{,}70\ ß_{FT,90}$ (Storage B) $ß_{C,90} \geq 0{,}70\ ß_{C,90}$ (Storage B)	$6{,}7 \geq 0{,}7x6{,}3$ $44{,}3 \geq 0{,}7x39{,}3$
Flexural tensile and compressive strength (Storage B)	$ß_{FT,28} \geq 8$ N/mm² $ß_{C,28} \geq 45$ N/mm²	6,1 N/mm² 37,0 N/mm²
Swelling (Storage A)	$\varepsilon_q \leq 0{,}30‰$ after 28d	0,12‰
Shrinkage (Storage B)	$\varepsilon_s \leq 0{,}90‰$ after 28d	0,74‰
Static modulus of elasticity	-[1]	18134 N/mm²
Dynamic modulus of elasticity	25 kN/mm² $\leq E_{dyn} \leq$ 40 N/mm²	19532 N/mm²
Adhesive strength 90d (Storage A)	$ß_{AS} \geq 2{,}0$ N/mm²	2,81 N/mm²
Adhesive strength 7d (Storage B)	$ß_{AS} \geq 2{,}0$ N/mm²	2,05 N/mm²

[1:] No quantitive requirements imposed
Storage A: 24h humid, than water storage
Storage B: 24h humid, than standard climate (23 ± 2) °C, (50 ± 2) % rel. hum.

4. DISCUSSION

Increasing additions of EVA increased shrinkage of the developed basic mix M1 (Figure 3a). The absolute changes in length are even higher than for the basic mix M1. The requirements of RL-SIB allow a maximum shrinkage of 0.9‰ after 28 days. This value is obtained neither from the basic mixture nor the mixtures M2 and M3 with EVA in different concentrations.

An opposite trend was observed for the samples with SRA. Here, the addition of SRA with a concentration of 4% based on the weight of cement (mix M4) reduced the shrinkage (Figure 3b). But this mortar does also not meet the requirements of the RL-SIB on the allowed shrinkage. The requirement of the RL-SIB was achieved only by the highest addition of 6% SRA (mix M5). The combination of both admixtures, EVA and SRA, has a beneficial effect as the lowest shrinkage deformation was obtained with the addition of 1.5% EVA and 6% SRA (mix M6). Due to the combination of EVA and SRA, the requirement of the RL-SIB, which allows a maximum shrinkage of 0.9‰ after 28 days, was fulfilled and a stable value of the total shrinkage was obtained that does not exceed a value of 0.9‰ after 91 days.

Further durability tests such as freeze thaw resistance (CF test according to CEN/TS 12390-9), adhesive strength after thunder shower simulation (RL-SIB part 4 § 2.7.7), adhesive strength after frost-deicing salt stress (RL-SIB, part 4 § 2.7.4) were performed will be discussed elsewhere.

5. CONCLUSIONS

The damage and the causes for the deterioration of a listed building were investigated. The examined building ismade of structural lightweight concrete and has a unique façade that is designed as board formed concrete. The carbonation-induced corrosion of the reinforcement is the main cause of the damages. As there is no certified repair mortar with lightweight aggregates available on the German market; a polymer modified repair mortar was developed in accordance to DIN EN 1504 and RL-SIB. The existing German standards and guidelines for concrete repair cannot be applied to present case without additional modifications. Therefore, the mechanical properties of the developed repair mortar were adapted to the existing structure. In this paper the influence of admixtures on the shrinkage behaviour was investigated in detail. The results show that the interaction of concrete admixtures has a beneficial effect on the shrinkage.

In summary, it should be noted that the workmanship of repair is an important issue. Therefore, the maintenance planning for listed buildings requires close cooperation and coordination of experts in concrete technology, conservators, art historians and public authorities[6].

ACKNOWLEDGEMENTS

We would like to thank the owner of the building Ms Dr. Locke for her support as well as Müller Simon Architekten for the implementation of the maintenance work.

REFERENCES

[1] Zementmerkblatt B13: `Leichtbeton`, Verein Deutscher Zementwerke e.V. Germany, 06.2014.

[2] DIN EN 1504 - Produkte und Systeme für den Schutz und die Instandsetzung von Betontragwerken - Definitionen, Anforderungen, Güteüberwachung und Beurteilung der Konformität, Teile 1-10. Beuth Verlag, Berlin 2004-2010.

[3] Instandsetzungsrichtlinie Schutz und Instandsetzung von Betonbauteilen, Teile 1-4. Deutscher Ausschuss für Stahlbeton (German Committee for Reinforced Concrete). Beuth-Verlag, Berlin 2001.

[4] BARG Baustofflabor GmbH & Co. KG, Prüfzeugnis Nr.: An/23548-11/12.

[5] Hüsken, G. and Brouwers, H.J.H., A new mix design concept for earth-moist concrete: A theoretical and experimental study', Cement and Concrete Research **38**(10) (2008) 1246–1259.

[6] Ünal, M., Kühne, H.-C., `Repair and Restoration of "Dalle de Verre" Concrete Windows`, in International Conference on Concrete Repair, Rehabilitation and Retrofitting` Cape Town, South Africa, 02-05. September 2012, 607-612.

SESSION THREE

Advances in concrete production and use

STRENGTH DEVELOPMENT, FORMWORK REMOVAL AND TEMPERATURE CONTROLLED CASTING OF CONCRETE IN A HIGH-RISE BUILDING

Ali Elmaskaya (1) and Mehmet Ali Taşdemir (2)

(1) Nida Construction & Turism Co, Inc., Istanbul

(2) Istanbul Technical University, Istanbul

Abstract

The three meters deep foundation of the high-rise building studied, called Palladium Tower, was cast in three layers, one meter at a time. The maximum temperature of the mass concrete foundation at the core and the maximum allowable mean temperature of each foundation layer calculated in terms of core, bottom and edge temperatures were 55ºC and 53ºC, respectively. During hardening, the maximum allowable temperature difference between the internal average temperature and the surface temperature measured at the concrete cover depth was less than 20ºC for each layer. The average temperature differences between the layers were also limited to a maximum value of 20ºC. At early ages, a reliable basis for determining the proper formwork removal time was used to prevent cracking of reinforced concrete members. For this purpose, the curing parameters including the time, temperature, the method of placing concrete in the structure and the test specimens, especially cylinders cured on site, as well as the weather conditions were recorded. When formwork was removed, there was no excessive deflection or distortion and no evidence of cracking or other damage to the structural members were observed. Protection of the steel reinforcement against corrosion, due to chloride ion diffusion and/or carbonation, is the most important factor in achieving long service life of the structure. Thus, the concrete cover on the reinforcement was ensured to have enough thickness and strength. For this purpose, special repair techniques were applied to the locally segregated surfaces against corrosion of reinforcements.

1. INTRODUCTION

In the design and construction of high rise buildings, high strength concrete is used more often because element dimensions of these structures are larger compared to those built using normal strength concrete. However, the heat of hydration of their mass concrete foundations and the resulting temperature rise in the concrete can cause thermal cracking [1]. It is commonly thought that mass concrete principles only apply to large dams, but they apply to any large pours such as massive foundations, bridge piers, thick slabs, nuclear plants, and some large scale structural columns and shear walls. Therefore, ACI 207 defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking" [2].

The cracking of concrete at early ages can be prevented by controlling its volumetric consistency during hardening, the time dependent deformations and their variations due to the temperature differences between the interior and the exterior as well as between the layers, the element sizes and the pouring order [3-8]. When dimensions are greater than 1m, the temperature rise should be considered. Since the depth of foundation in Palladium Tower investigated here is 3m, it meets the definition of mass concrete. The heavy reinforcement in the foundations of this project does not mean that the concrete will not crack and certainly will not prevent generation of heat [1]. Results from the literature suggest that the formation of delayed ettringite is activated when concrete is subjected to elevated temperatures [9-15]. Thus, increased internal temperature due to casting of mass concrete must be considered. For this purpose, a temperature monitoring method for the deep foundation in this project was used to minimize the maximum concrete temperatures and temperature differentials, thus preventing thermal cracking and damage through delayed ettringite formation. In addition to temperature controlled casting of massive members, strength development of concrete, formwork removal methods, and repair techniques in the local areas with surface defects were employed.

2. ABOUT THE PROJECT

Palladium Tower was built in Ataşehir which is a district in the Asian part of Istanbul. Its construction was completed in 2014. The total construction area in the project is 99.784 m^2. The tower is used as an A+ type office building. The tower consists of 4 basements, ground floor, and 42 normal floors. The height of the tower is 180 m. The investor was Tahincioğlu Group and main contractor was Nida Construction Co. In the foundations of the tower, concrete class was C40/50. However, in the construction of columns, core and shear walls, and slabs, concrete class of C50/60 was used.

3. EXPERIMENTAL WORK

The Palladium Tower Project required concrete designed for durability, unlike those used in normal structures. In order not to cause cracks in concrete due to the early age temperature differences between the interior and the exterior as well as between the layers, cements with low heat of hydration, but with high strength were utilized.

Binders and aggregates with low alkali and reactive silica contents were selected to prevent undesired alkali-aggregate reactions which can appear after long years. The aggregates used were continuously tested in batches to ensure that they are clean and have proper shape and size distribution. In the selection of chemical admixtures, it was aimed to achieve the consistency between cement and aggregate as well as the consistency of the fresh concrete within its respective tolerances over time.

The main principle for durability was producing a concrete that was as impermeable and crack free as much as possible. An impermeable concrete was achieved by using low water/cement ratio and by keeping the properties of the binding material under control in order to minimize the capillary pores. As a consequence of the achieved durability targets through minimization of capillary pores, the concrete strength requirement was automatically satisfied.

3.1 Cements

Cements used in this study were as follows; CEM I 42.5 R and CEM IV/B-P 32.5 N. Densities of these cements were 3.16 g/cm^3 and 2.86 g/cm^3, respectively. Initial and final setting times of CEM I 42.5 R were 166 and 210 minutes, respectively. These values were 188 and 230 minutes, respectively, for CEM IV/B-P 32.5 R. Additional tests on these cements verified that their physical, chemical and mechanical properties satisfied the requirements of TS EN 197 standard.

Heat of hydration of cements at 2, 7 and 28 days, determined according to TS EN 196-8 are given in Table 1.

Table 1: Heat of hydrations of cements.

Cement	Heat of hydration, cal./g		
	2 days	7days	28 days
CEM I 42.5 R	71.9	86.1	92.5
CEM IV/B-P 32.5 R	48.0	61.2	66.4

3.2 Aggregates

Particle densities and water absorptions of aggregates used are shown in Table 2. An aggregate sample containing a fraction of 10-14mm sizes taken from both coarse aggregates was subjected to the abrasion test in accordance with TS EN 1097-2. After 500 revolutions, the loss of abrasion was 20.3%, which corresponds to the Los Angeles Abrasion Category of LA_{25}.

Table 2: Particle densities and water absorptions of aggregates

Aggregate	Density, g/cm^3	Water absorption,%
Natural sand	2.58	1.6
Limestone fines (0-4mm)	2.74	1.1
Coarse aggregate No. I (5-12mm)	2.73	0.8
Coarse aggregate No. I (12-22mm)	2.75	0.6

Petrographic analysis of limestone fines and coarse aggregates taken from the same quarry are summarized below.

Rock contained similar proportions of cryptocrystalline/micritic and recrystallized/ microcrystalline calcite grains. Texture of the rock was homogeneous. Micritic calcite formed the matrix of the rock. Grain size of the calcite was generally 0- 0.01 mm. Recrystallized calcite was middle – large sized, generally 0.02 mm, with a homogenous grain size distribution. Trace amount of opaque minerals was observed.

Table 3: Crushed Rock Mineralogical Constituents

Mineralogical Constituents	(%)
Calcite (Primary, micritic)	45-60
Calcite (Secondary, recrystallized)	40-45
Opaque mineral + Iron oxide	1-1,5

From the ASR point of view, ASTM C 1260 states that if the expansion of mortar bar is lower than %0.10 the aggregate is considered innocuous, if the range of expansion is between 0.10% and 0.20%, the aggregate is considered potentially reactive and above %0.20 the aggregate is considered reactive. In natural sand, limestone fines and coarse aggregates from the same quarry and in coarse aggregates, expansions recorded according to ASTM C 1260 were 0.04% and 0.08%, respectively. As a result, these aggregates can be considered innocuous, because the expansion values measured according to ASTM C 1260 are less than 0.1%.

3.3 Main concrete mixtures used

In the composition of Mix 1 used for the foundation (i.e., C40/50), cements were as follows; CEM I 42.5 R: 120 kg/m^3 and CEM IV/B-P 32.5 N: 260 kg/m^3 and water /cement ratio was reduced to 0.40. Densities of these cements were 3,16 and 2,86 g/cm^3, respectively. Thus, the mixture composition was: Total cement: Natural sand: Limestone fines: Coarse aggregate No.I : Coarse aggregate No.II: Chemical admixture (in plant): Chemical admixture (in construction site): Water = 1 : 1.251 : 1.017 : 1.880 : 1.271 : 0.019 : 0.004 : 0.40. For this mixture, the target strength was as follows: $f_{tg} = f_{ck} + 1.48\ \sigma = 40 + 1.48 \times 3.55 = 45.3$ MPa, here the 1.48 value corresponds to 93% confidence level in accordance with EN 206.

For cold weather conditions, in the composition of Mix 2 used for columns, walls and slabs (i.e. C50/60); cements were as follows; CEM I 42.5: 300 kg/m^3 and CEM IV/B-P 32.5 N: 120 kg/m^3 and water /cement was reduced to 0.36. Thus, the mixture composition was: Total cement: Natural sand: Limestone fines: Coarse aggregate No.I : Coarse aggregate No.II: Chemical admixture (in plant): Chemical admixture (in construction site): Water = 1 : 1.040 : 0.793 : 1.990 : 0.531 : 0.022 : 0.004 : 0.40. For this mixture, the target strength was as follows: $f_{tg} = f_{ck} + 1.48\ \sigma = 50 + 1.48 \times 2.51 = 53{,}7$ MPa, here the 1.48 value corresponds to 93% confidence level in accordance with EN 206.

For normal weather conditions, in the composition of Mix 3 used for columns, walls and slabs (i.e. C50/60); cements were as follows; CEM I 42.5 R: 130 kg/m^3 and CEM IV/B-P 32.5 N: 290 kg/m^3 and water /cement was reduced to 0.37. The other constituents were kept the same as in Mix 2.

4. TEMPERATURE CONTROLLED CASTING OF FOUNDATION CONCRETE

A maximum temperature difference between the mean temperature of the element and the temperature in the surface of the element was defined. For this purpose, during hardening, the maximum allowable temperature difference between the internal average temperature and the surface temperature measured at the concrete cover depth was required to be less than 20ºC. The maximum temperature of the mass concrete foundation was 55ºC at the central part of the third layer of the foundation. During the three consecutive concrete castings of the foundation, interlayer differences in temperature were also restrained at 20ºC and the targeted limit values were not exceeded.

As seen in Figure 1, the three meters deep foundation of the high-rise building studied was cast in three layers, one meter at a time. Before casting of the mat foundation, three trial concretes of 1 m^3 volume were prepared. Based on the temperature-time curves and strength developments of these concretes, in accordance with the targeted principles, temperature controlled casting that also comply with the design strength requirement was accomplished.

The average internal temperature (T_{ave}) was calculated as:

$$T_{ave} = (2/3)T_c + (1/6)\,T_b + (1/6)\,T_e \quad (1)$$

where T_c, T_b and T_e are the temperatures at the center (core), bottom and midpoint of layer edge, respectively. T_s in Figure 2 is the temperature at the concrete cover depth (i.e. 2-3cm).

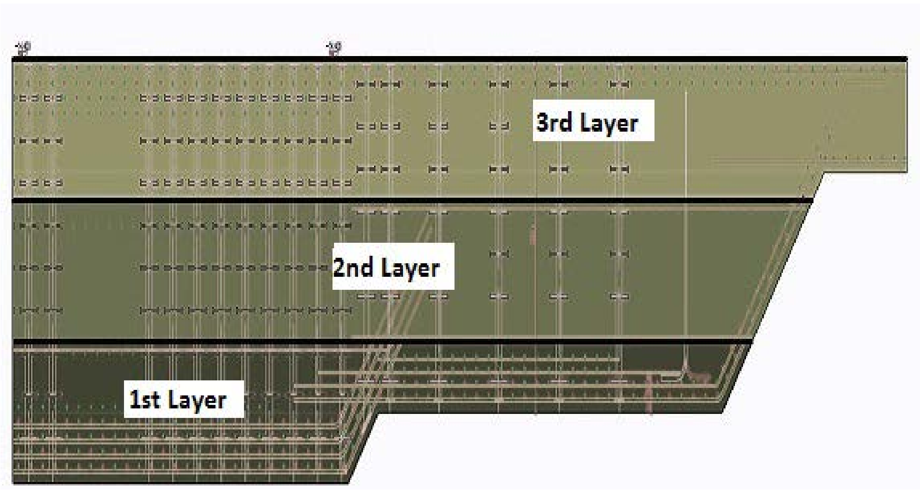


Figure 1: Cross-section of the foundation cast in three layers

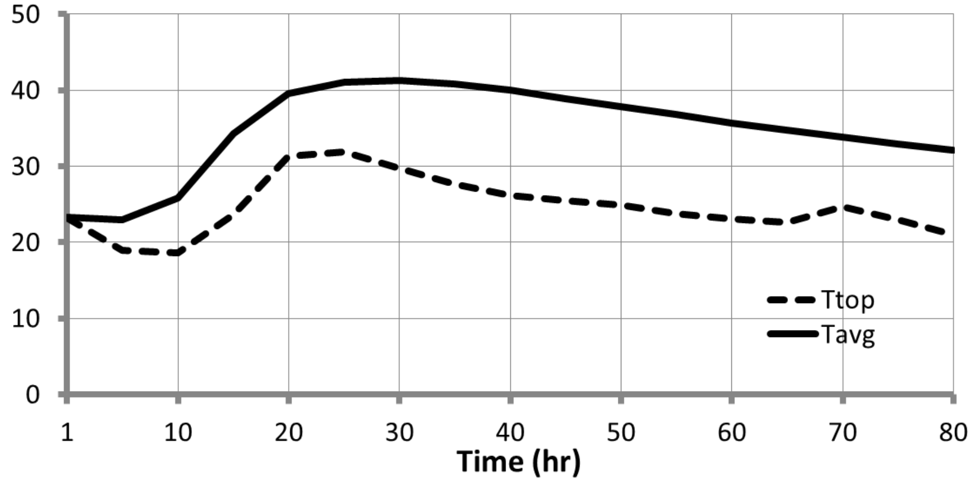


Figure 2: Typical average temperature-time and surface temperature-time curves for the first layer of the foundation.

Thus, in this study, following targeted objectives were realized: i) Maximum temperature in mass concrete elements was controlled and kept below 65ºC, ii) Average internal temperature of concrete was also controlled and kept below 60ºC, iii) The amount and type of cement were carefully selected, pozzolans were employed for lower heat of hydration, iv) For each foundation layer, the maximum allowable temperature difference between the internal average temperature and the surface temperature measured at the concrete cover depth was less than 20ºC, v) Alternating moisture conditions were avoided, vi) Permeability of concrete was kept at a low level, and vii) During placement, the maximum fresh concrete temperature was less than 32ºC.

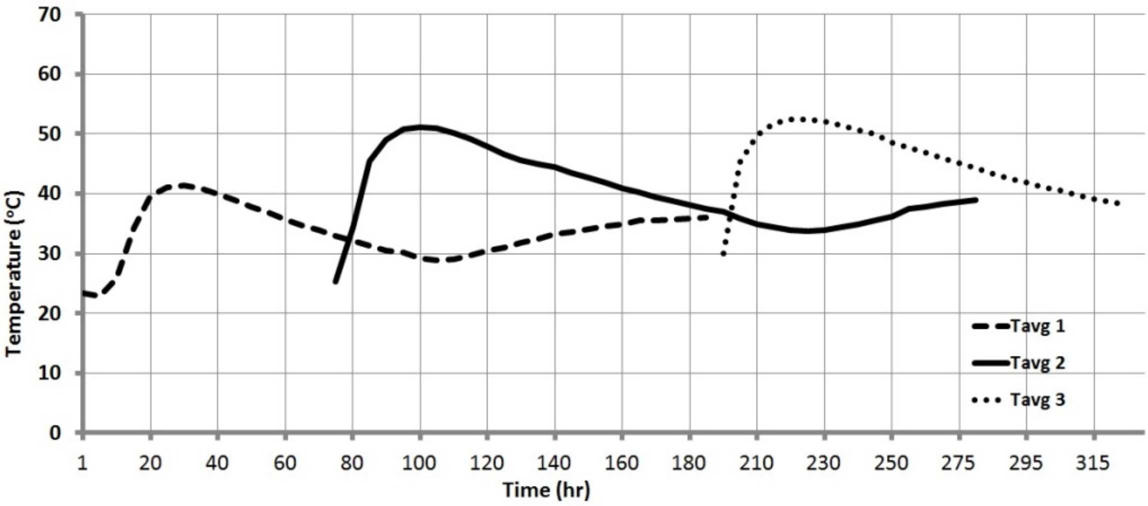


Figure 3: The average temperature versus time curves for the first, second and third layers.

5. STRENGTH DEVELOPMENT OF CONCRETE AND REMOVAL OF FORMWORK

For the vertical R/C elements, there are several factors affecting lateral pressure on the formwork. They can be summarized as follows: the vertical formwork height, rate of placement, materials of formwork, vibrator's power, vibration immersion length, mix design (water/binder, fine aggregate/coarse aggregate, paste volume, binder content, binder type, supplementary cementitious materials and filler, chemical admixtures), rheology of concrete, concrete temperature, amount and location of reinforcement, concrete slump, unit weight of concrete, size and shape of formwork, and maximum aggregate size.

5.1 Strength development

In this study, the strength development of concrete in R/C structures was investigated for timely removal of the formwork. For this purpose, separate test specimens that were cured on site were kept in the environment of the construction site. It is clear that curing conditions of these specimens were different from those cured under standard laboratory conditions to evaluate the 28-day strength of concrete. For determining the suitable formwork removal time, the curing parameters including the time, temperature, the method of placing concrete in the structure and the test specimens, as well as the weather conditions were recorded.

For the strength development of concrete, the relation between age and compressive strength can be written as follows:

$$f_c' = a + b\log(t) \ , \tag{2}$$

where, t is the age of concrete (days) , a and b are coefficients of the equation in dimensions of MPa ve MPa/log(days). In this equation, the value of constant a can be considered as the level of strength at one day of age, and b shows the slope of the line in the strength versus log(t) in the relation.

The relation in Eq. (2) includes the combined effects of weather temperature and the type of cement. It is clear that besides the type of cement used and the ambient temperature, there are several factors affecting the strength of concrete such as mix composition, aggregate type, and curing time.

For cold weather conditions, Mix 2 was used to determine coefficients in Equation 2. In order to find out a and b in Eq. (2), a total of 35 specimen groups were tested at different ages such as 4, 5, 6, 7 and 28 days. Each group consisted of three specimens and the least square method was used to obtain the best fit. As seen in Figure 4, the relation between strength and time was obtained as $f_c' = 10.53 + 30.315\log(t)$.

For normal weather conditions, Mix 3 was used to determine the coefficients in Eq. (2). In this analysis, a total of 27 specimen groups were tested at 5, 7 and 28 days of age. Using the same approach, the relation was obtained as $f_c' = 25.23 + 20.191\log(t)$.

As can be seen in Figure 4 and Figure 5, knowledge of the strength development is helpful for determining the removal time of formwork. Especially, the use of cylinders cured on site was beneficial and high correlations were obtained in the analysis. In addition to the parameters influencing in-situ strength development, the water/cement ratio also affect the rate of strength gain of concrete. Since the mixes 2 and 3 used in this study have low water/cement ratios, they displayed rapid gain of strength. The reason is that in concrete with low water/cement ratio, the cement grains are closer to one another and a more continuous system of gel develops. Thus, in this study, the concrete strength gain with age was monitored by a simple logarithmic equation. The equation has the potential to predict strength for a given age. This approach has helped in making fast decisions to prevent unsuitable concreting at the construction site and hence reducing delays in the duration of construction in such an important project.

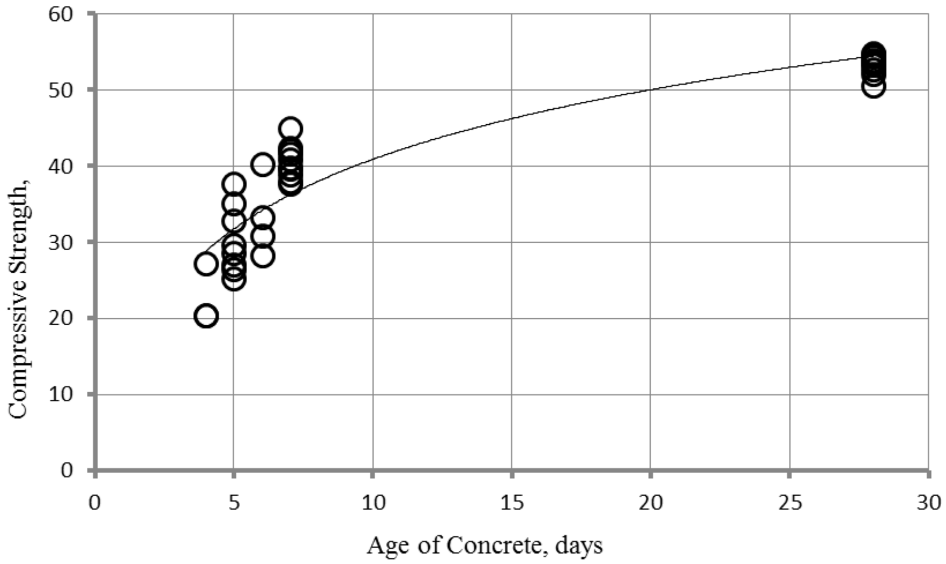


Figure 4: In cold weather conditions, relation between cylinder compressive strength and age of concrete for Mix 2

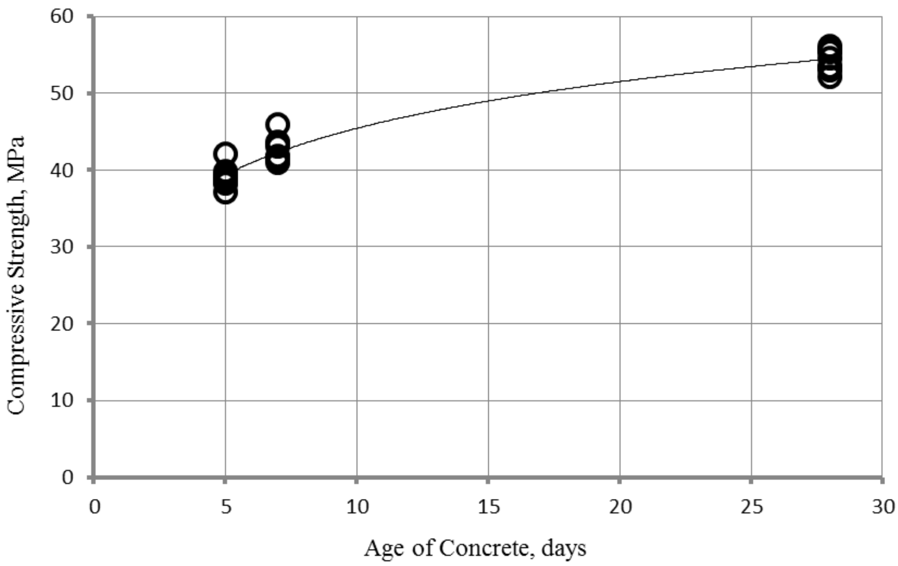


Figure 5: In normal weather conditions, relation between cylinder compressive strength and age of concrete for Mix 3

5.2 Removal of formwork

At early ages, a reliable basis for determining the proper formwork removal time was necessary to prevent the damage to the structure. When formwork was removed, there was no excessive deflection or distortion and no evidence of cracking or other damage to the R/C structure was observed.

Factors affecting the formwork removal time are the position of the forms, the loads coming on the R/C members after stripping, the ambient temperature, and subsequent loads on the member.

The time of formwork removal was based on the strength gain of the cylinder specimens cured on site. For the evaluation of formwork removal, weather conditions were recorded and used in conjunction with the cylinder test results. The formwork removal times were increased if low temperatures were recorded. Early removal of formworks was desirable for completing construction on time and was usually desirable for curing. Formwork of the vertical elements such as shear walls, columns and also beam sides were removed without damage to concrete surfaces and edges. For, the vertical R/C members, the compressive strength of concrete cube specimens kept at construction site was approximately 10 MPa prior to the formwork removal. Thus, forms and supports were removed without any impact and load eccentricity.

The forms of the horizontal R/C members such as beams and slabs were removed carefully when the compressive strength was equal to or greater than a minimum of 75 percent of the specified design compressive strength. Therefore, re-shoring was one of the most critical operations in the construction of slabs in this type of multistory building. Special attention was paid on removal of forms and shores. Necessary re-shoring was used to support combined dead and construction loads, thus sufficient load carrying capacity was provided for each slab. During re-shoring, construction loads were not permitted on the newly constructed slab. As seen in Figure 6, re-shoring elements were almost located in the same position on each successive slab. In order to support the imposed loads without excessive stress or deflections, shoring was provided for a sufficient number of floors such as four floors as indicated in Figure 6. The most severe loads such as construction materials and equipment were not imposed on the structure during construction of each slab.

Figure 6: A typical example showing shores and re-shores for the high rise building studied.

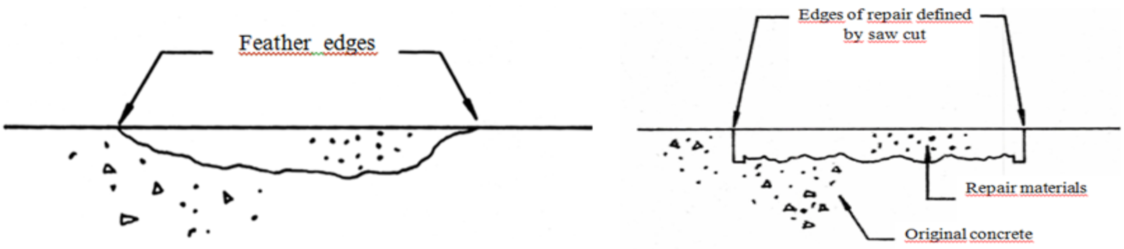
It is known that a minimum curing period of seven days is recommended for all exposure classifications. The vertical R/C elements such as column and shear walls were maintained in sufficiently moist condition using curing compounds. For both vertical elements and slabs, depending on the climatic condition, water sprays, water retaining techniques such as polyethylene sheets or insulating blankets were used.

Since the main cause of cracking in R/C elements is insufficient ambient temperature required for early age strength gain, special measures were taken to maintain and systematically check the weather conditions during the casting and curing of the R/C elements. It was essential to make an evaluation of whether the temperature conditions were sufficient for early strength gain in order to find the correct removal time for the R/C elements, or to modify the temperature conditions of the construction site to comply with the required temperature for early formwork removal.

6. CORRECTION OF DEFECTS IN NEWLY HARDENED CONCRETE

When constructing a structure, the stages that should be implemented can be summarized as follows: 1º) The preparation of the project documents which also include the design that has been made according to the present soil and environmental conditions as well as the earthquake performance calculations, 2º) selection of materials that are suitable for the objectives of the project, 3º) assembly and workmanship, 4º) inspection and auditing, and 5º) repair works. As mentioned here, repair work is an indispensable part of the project, therefore, it should be properly applied to the defective areas.

Steps for repairing the local areas of surface defects can be summarized as follows: i) The boundary of each defective area to be repaired was outlined. ii) In order to remove loose particles, the area was cut and chipped to create a rough surface for proper bonding of the repair cement based mixture. For chipping a small electric chipping hammer was used. iii) The feather edges as indicated in Figure 7 were not allowed. As seen in this figure, the first one (Fig.7a) is incorrect and the second one (Fig. 7b) is correct. iv) The dust and debris in the area to be repaired were removed, since a well-prepared area should allow the new repair material to have a suitable surface for good bonding. v) All surfaces to be repaired were dampened for good bonding, however ponding of the area was not allowed. vi) The repair material was prepared to fill in the area that was chipped. vii) Although it was necessary to follow the manufacturer's instructions, for strong bonding an epoxy with two components was chosen (Figure 8). viii) The repair mortar was placed into or on the patch location, and force was applied to get a repair material without air bubbles. ix) The surface of the repair material was leveled using a trowel, leaving the repaired area slightly higher than the adjacent edges to allow for setting and shrinkage. x) The area was trowelled with a steel finishing trowel when the patch material has become stiff (Figure 9). xi) After another hour or two allowed for setting and initiation of hardening, final finish was applied through trowelling. During these stages, it was necessary to splash a little water on the surface to retard the drying, also to make the finishing process somewhat easier. The repaired area was kept wet for a few days, it was protected from direct sunlight. Protection was also necessary in cold and hot weather conditions. xii) The repaired surface was ground slightly after one week, until uniform color was obtained (Figure 10).



(a) Incorrect (b) Correct

Figure 7: Edges to repairs, the saw cut should be used to avoid feather edges.

Figure 8: Application of epoxy using a brush (For strong bonding, an epoxy with two components is suggested)

Figure 9: Repair mortar was applied to locally defective concrete surface

Figure 10: Slight grinding after one week, nearly uniform color was achieved

Protection of the steel reinforcement against corrosion, due to chloride ion diffusion and/or carbonation, is the most important factor in achieving long service life of the structure. Thus, the concrete cover on the reinforcement bar was ensured to have enough thickness and strength. Eventually, in order to ensure sufficient strength gain at early ages, sufficient curing was applied and the hardening period was closely followed.

As typically observed, there were some voids like “bird eyes” on the surfaces of the R/C elements. These voids, also called bug holes, are small regular or irregular cavities, usually not exceeding 10 mm in size, resulting from entrapment of air bubbles on the surface of cast concrete during placement and consolidation. The number of such voids in a concrete surface of 1 m^2 was less than 50, this limitation was also used in several specifications. In this study, the following key factors were also considered to minimize these voids and to improve the surface quality of concrete: i) mold release agent, ii) free water in concrete, iii) air, iv) condition of formwork, v) vibration, vi) maximum aggregate size, combined aggregate grading and coarse aggregate shape, vii) consistency of concrete, and vii) rate of concrete casting.

7. QUALITY CONTROL OF CONCRETE

Quality control of concrete started with the inspection of aggregate, cement, water and additives. It continued with the measurement of properties in the fresh and hardened state of concrete formed by these constituents. Continuous inspection of concrete from a production plant was performed through standard cylinder compressive strength tests on samples taken at the plant and at the job site where acceptance criteria were applied for each concrete batch. The results obtained by the inspection company were evaluated through standard requirements. Therefore, the quality control of concrete consisted of the following processes: i) Mix design (calculation of the amounts of materials that constitute concrete), ii) Production, iii) Sampling, storing and curing, iv) statistical evaluation and application of acceptance criteria.

In this project, high quality concrete required in the project specification was obtained through proper production, placing, and curing. For these purposes, implemented steps were as follows: selection of the right materials for the purpose, use of high quality materials, proper composition and mixture of materials including water, proper placing and compaction with minimum bleeding, proper concrete curing for desired maturity, and prevention of high temperatures and temperature gradients within concrete in the initial hardening period.

For C50/60 concrete cylinder compression test results that follow normal (Gaussian) distribution, if the mean compressive strength is f_{cm}, the characteristic strength (f_{ck}) can be calculated as follows: $f_{ck} = f_{cm} - 1.48\ \sigma \geq f_{cd}$. In the expression for the characteristic strength, the 1.48 value corresponds to 93% confidence level in accordance with EN 206.

Control cards were used to inspect the compressive strength of concrete. Horizontal axis of these cards may be time, production unit or the unit of the inspected property. Vertical axis shows the arithmetic average of the production unit, warning and action limits. In this project, the design concrete strength was 50 MPa and 95% confidence level was specified. For this concrete class, since the standard deviation was 2.51 MPa, the target strength (f_{ct}) can be calculated as follows: $f_{ct} = f_{cd} + z_{0.05}.\sigma = 50 + 1.64x2.51 = 54.1$ MPa, where $z_{0.05}$ is the value of the standard normal variable that corresponds to 95% confidence level. The average strength of 286 groups was 54.14 MPa, which is identical to the target strength (54.14 MPa). The target strength for the 93% confidence level, however, was calculated as $f_{ct} = 50 + 1.48x2.51 = 53.7$MPa. In this case, the average strength is greater than the target strength calculated according to the 93% confidence level.

Therefore, the top and bottom limits corresponding to 95% confidence level were 56.4 MPa and 51.7 MPa, respectively. These values are the action (production termination) limits. In addition, the top and bottom limits corresponding to 90% confidence level for the strength of batches were calculated as 52.2 and 56.0 MPa, respectively. These are, however, warning limits for the production of concrete.

The production chart obtained in this study has shown that the course of production was good. The results mostly did not fall outside the action (production termination) limits and they have shown an approximately balanced distribution around the mean. Similar results were also obtained for the foundation concrete (i.e., C40/50).

8. CONCLUSIONS

Based on the results obtained in this study, the following conclusions can be drawn:

1) The three meters deep concrete foundation of the tower was cast in three equal layers. In the first, second and third layers, the maximum temperatures recorded were 45ºC, 51ºC and 55ºC, respectively. For these layers, however, the maximum average internal temperatures were 41ºC, 51ºC, and 53ºC, respectively. The internal average temperature differences between the layers were below or equal to 20ºC. For the first, second and third layers, the temperature differences between the internal average temperature and the surface temperature at the concrete cover depth were 14ºC, 17ºC and 18ºC, respectively. Thus, during casting of the concrete foundations, maximum temperatures and the temperature differences between interior-exterior and between-layers were restrained and the specified limit values were not exceeded.
2) For determining a suitable formwork removal time for the slabs, separate test specimens were cast and cured on-site, exposed to the environment of the construction

site. These specimens were different compared to those cured under standard laboratory conditions to evaluate the 28-day strength of concrete. Since the early removal of formwok was desired, the use of a logarithmic type of relation for the strength development was beneficial. In addition, the curing parameters including the time, temperature, the method of placing concrete in the structure and the test specimens, and the weather conditions were recorded.

3) The forms of beams and slabs were removed carefully when the compressive strength was equal to or greater than a minimum of 75 percent of the design compressive strength. Necessary re-shoring was used to support combined dead and construction loads. Shoring was provided for a sufficient number of floors such as three or four floors without any early age cracking due to excessive deflections. During re-shoring, construction loads were not permitted on the newly constructed slab. During formwork removal, there was no excessive deflection or distortion and no evidence of cracking or other damage was observed to the structural member.
4) Formwork of the vertical elements such as shear walls, columns and also beam sides were removed without damage to concrete surfaces and edges. At the construction climatic conditions, about 10MPa cube compressive strength prior to the formwork removal was sufficient for the vertical elements mentioned. Thus, forms and supports were removed without any impact and load eccentricity.
5) Since the thickness and strength of the concrete cover play important roles in the long term performance of an R/C element, the repair work was applied to the defective surface areas as an indispensable part of the construction. Especially, for the locally segregated surface areas, a special patching technique was employed as a protective measure against future corrosion of reinforcements.
6) Mean concrete compressive strength was greater than the target strengths calculated according to both 93% and 95% confidence levels for each concrete class (i.e. C40 and C50). The production charts used in this study showed that the results mostly did not fall outside the action (production termination) limits and they have shown an approximately balanced distribution around the target strengths.

ACKNOWLEDGEMENT

The authors acknowledge Mr. Mustafa Adnan Öğüt - Emir Engineering Ltd., for his encouragements to write this paper, who is the structural designer of the Palladium Tower Project.

REFERENCES

[1] Whittier, S., Olyniec, J., and McGlohn, R., "Minimizing Temperature Differentials in Mass Concrete", Concrete International, Dec., 2014, pp. 43-45.

[2] ACI Committee 207, "Mass Concrete – 207. 1R-96" ACI, 1996.

[3] "Temperature control of mass concrete", in *EM 1110-2-2200, US Army Corps of Engineers Publications*, 1995.

[4] Elmaskaya, A. ve Taşdemir, M.A., "Temperature Controlled Casting and Strength

Development of the Deep Foundation Concrete of Palladium Tower Project", 21-23 February 2013, Proceedings of RMCO of Turkey 2013, pp. 366 – 381(in Turkish with English summary).
[5] Yetiş S., Kaya A. ve Taşdemir, M. A., "Temperature Controlled Concrete Casting in Foundations of Porta Batumi Tower, Strength Development, Removal Times of Formwork and Surface Properties of Concrete", Proceedings of 9th National Concrete Congress, Antalya, 16-18 April 2015, pp.503-516 (in Turkish).
[6] Taşdemir, M.A., Akkaya, Y., Erdoğdu, S., and Ozturk, M., "Quality Assurance for the Deepest Immersed Tube Tunnel: Marmaray Project", Advances in Concrete : An Asian Perspective, ACECON 2010, Eds. R. Gettu et al., 2010, pp. 252-267.
[7] Taşdemir M.A. and Akkaya, Y., "Turkey - Concrete Construction Industry - Cement Based Materials and Civil Infrasturucture - (CBM-CI)", CBM-CI International Workshop, Ed. S. Ahmad, Karachi, 2007, pp. 185-200.
[8] Akkaya, Y. and Taşdemir, M.A., "Turkish Experience in Concrete Construction Industry and Infrasturucture", UNESCO-IPRED-ITU Workshop, 6-7 July 2009, pp. 159-164.
[9] Stark, J., Bollmann, K., "Delayed Ettringite Formation in Concrete", Nordic Concrete Research, (23)2, pp. 1-25, 1999.
[10] Collepardi, M., "A State-of-the-Art Review on Delayed Ettringite Attack on Concrete", Cement and Concrete Composites, 25, pp. 401-407, 2003.
[11] Akman, M. S., "Gecikmiş Etrenjit Oluşumu (DEF)", 5. Ulusal Beton Kongresi, 1-3 Ekim, İstanbul, s.9-15, 2003.
[12] Divet, J., and Randriambololona, "Delayed Ettringite Formation: the Effect of Temperature and Basicity on the Interaction of Sulphate and C-S-H Phase", Cement and Concrete Research, 25, No. 3, pp. 357-363, 1998.
[13] Heinz, D., and Ludwig, U., "Mechanisms of Secondary Ettringite Formation in Mortars and Concretes Subjected to Heat Treatment", Katharine and Bryant Mather International Conference, J.M. Scanlons, ed., ACI, Detroit, 1987.
[14] Lawrence, C.D., "Mortar Expansions Due to Delayed Ettringite Formation: Effects of Curing Period and Temperature", Cement and Concrete Research, 25, No. 4, pp. 903-914, 1995.
[15] Zhaozhou, Z., Olek, J., and Diamond, S., "Studies on Delayed Ettringite Formation in Heat-Cured Mortars II. Characteristics of Cement that maybe Susceptible to DEF", Cement and Concrete Research, 32, pp. 1737-1742, 2002.

USE OF CONTROL CHARTS IN PRODUCTION AND CONFORMITY CONTROL – CASE STUDY

Ceyda Sülün

Assistant for standardization and environmental affairs of the German Ready Mixed Concrete Association (BTB)

Abstract

The revised EN 206 offers a new method of conformity control for compressive strength. This is the use of control charts as an alternative to the ´traditional´ conformity assessment of compressive strength.

In Germany there is no experience with the procedure of using control charts. Therefore a case study was carried out by the BTB in parallel with the current conformity control of concrete, and practical experiences with using the CUSUM control charts were collected.

For the case study small, medium and large production ready-mixed plants were selected and analyses carried out on individual concretes and concrete families. The study compares the method of control charts with the current methods and shows the advantages and limits for the practical implementation. The results of the case study shows that the CUSUM control charts improve the production and conformity control.

Keywords: Conformity testing, control charts, factory production control, compressive strength

1. INTRODUCTION

The CUSUM method for production and conformity control introduced with the new EN 206 concrete standard offers a number of advantages provided certain conditions are met. In a BTB pilot project, the advantages and limitations of the method have been assessed.

The introduction of the new EN 206 standard means that another method of conformity control for mean strength can now be used in a ready-mixed concrete plant in addition to the standard ones. Method B, the individual and mean value criteria can still be used for assessing the conformity of the concrete compressive strength. However, besides the usual proofs for the mean value criterion of individual concrete types and and families, concrete manufacturers can now also use the so-called control chart method.

2. GRAPHICAL EVALUATION

The control charts which can be used in accordance with Annex H of the EN 206 are the Shewhart method or the CUSUM method. In contrast to the standard methods, these are based on a graphical evaluation, with the Shewhart method showing the evaluation of the mean value criterion in graphical form. The CUSUM procedure, which is also evaluated graphically, refers to the so-called cumulative sums, or CUSUM in short. What is special about this method is that it initially serves as an actual control element of production control and additionally allows the conformity of the concrete compressive strength to be proven without special effort.

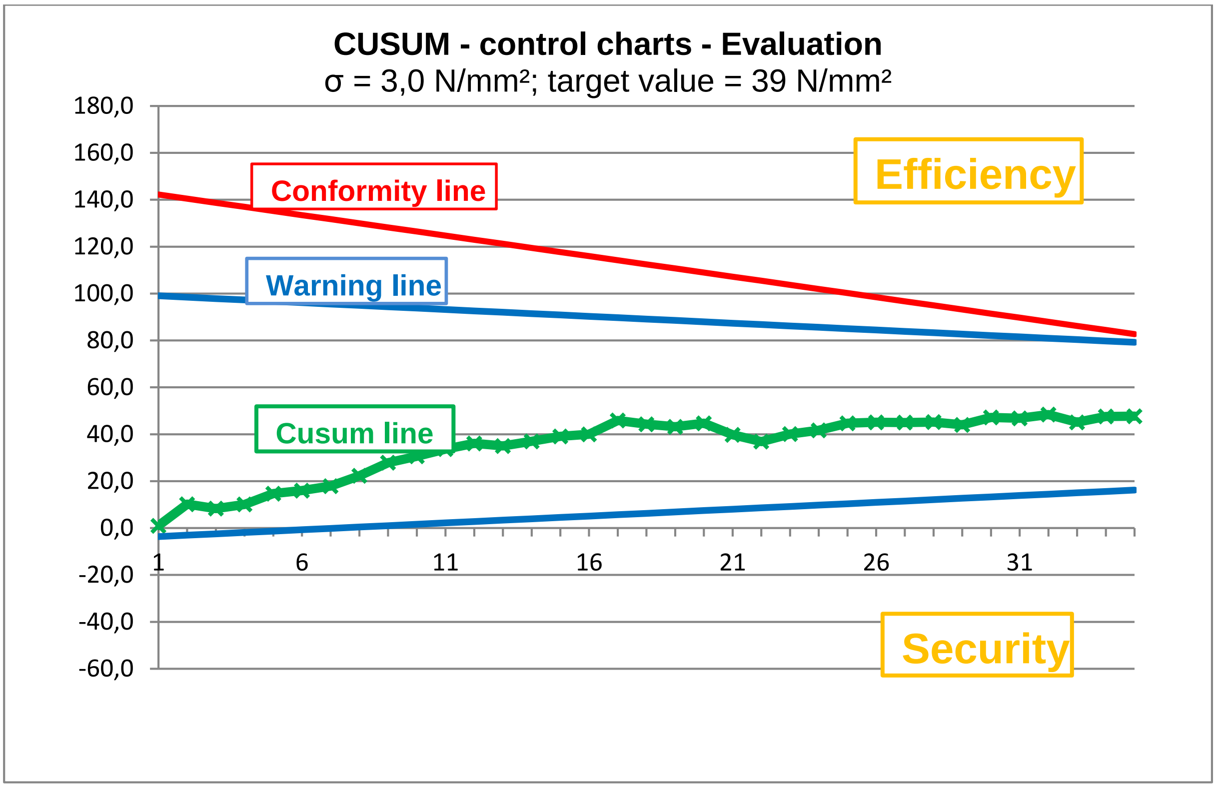


Figure 1: CUSUM control chart evaluation

3. STUDY ASCERTAINS SUITABILITY

As up until now there has been very little experience with the control chart system in Germany, the BTB commissioned a study to initially theoretically compare the "old" and "new" methods. The study noted that this method is well suited to improving production quality and also rendering it more economical.

Subsequently, the CUSUM method was investigated within a BTB pilot project for its practical pros and cons. The field test was carried out in six small, medium and large ready-mixed concrete plants. The results of the test show that the CUSUM method is advantageous for plants with continuous production. This is because, in addition to "formal" conformity control, it also allows more precise production control in terms of efficiency and security.

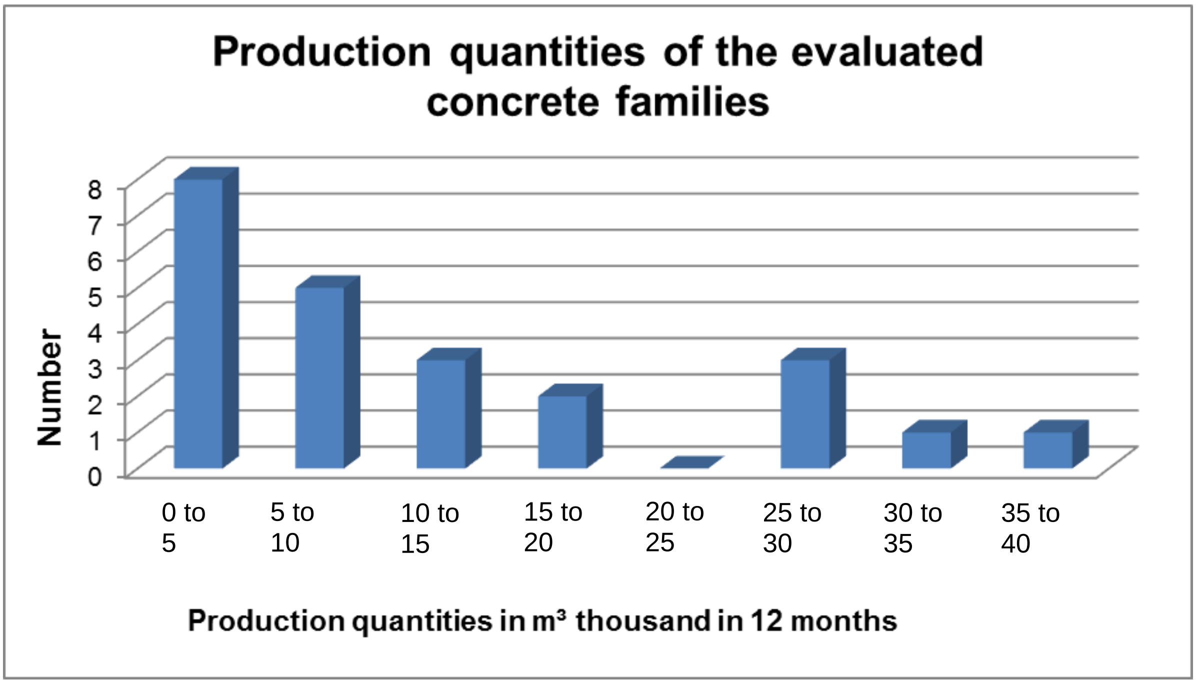


Figure 2: Production quantities of the evaluated concrete families

4. SYSTEMS WITH 15 TEST RESULTS AS A REFERENCE PERIOD

In the standard EN 206 it is stated that the reference period can deviate from 35 test results. The parameters for the use, for example, of 15 test results as a reference period are described in the notes in /1/.

It has also been studied whether the recording and evaluation of 15 test results is of equal value to the system with 35 test results, and therefore plants with continuous production and a lower testing frequency can also use CUSUM as proof of conformity. Concrete families with a production volume below 8000 m^3/year achieve at least 15 test results in three months.

5. CUSUM, METHOD C, COMPARED WITH EXISTING METHOD B FOR CONTINUOUS PRODUCTION

The majority of the concrete families evaluated at the same time in the pilot project were always conforming, and this was also reflected in the CUSUM evaluation. Often with the conforming concrete families the warning lines were crossed by the CUSUM lines (the cumulative totals), thereby indicating that a response was required i.e. some action needed to be taken in the quality system. These warnings were not discernible in the evaluations of the concrete families using method B. In order to compare methods C and B, a concrete family was used that according to the current evaluation was conforming. The reference period was 35 test results, as is now commonly used. The family was evaluated using CUSUM. According to the CUSUM evaluation, the family is compliant. The result was the same for the concretes with method B.

6. RESULTS OF THE STUDY

CUSUM means a restart for conformity control. A rethink is not required because it is a graphical analysis. Targeted monitoring of production is possible with CUSUM . CUSUM is a sound tool for production control, regardless of whether it is used for conformity control. Production trends can be recognised from the CUSUM curve. A sharp rise in the line indicates a target strength not corresponding to reality. A CUSUM line fluctuating around zero is ideal and reflects very uniform production. Continuously sloping CUSUM lines indicate that non-conformity is to be expected, as production is then taking place at a level significantly lower than the desired target strength.

The V-masks of the warning lines often provide an early indication that countermeasures need to be initiated. This in turn allows for a quicker response to deviations in production. An overlapping evaluation after each test result for production control is used, with the V-masks from the latest available test result being produced.

For clarity reasons, the representation of the overlapping evaluation with a reference period of 15 test results is broken down into steps of 5 test results. Concrete families and individual concretes can be evaluated using the CUSUM method (Method C) in a plant with continuous production, and a CUSUM evaluation then issued. This means that production plants with a low testing frequency, including and in particular plants with lower annual production volumes, can use CUSUM as a compliance system.

CUSUM allows the manufacturer to set their own reference period for each family or individual concrete that is produced continuously. The amount of work involved should be considered the same in each case, as use is always made of computer-aided evaluation.

Monitoring the standard deviation and the mean target value is an important parameter, otherwise evaluation with the CUSUM system as proof of conformity is impossible, possibly resulting in incorrect conformity statements. If these values alter after evaluation of the reference period, they should be amended and a new reference period started. A new reference period is reached at the evaluated plants no later than after 3 months. Continuous monitoring of production with CUSUM enables production to be both uniform and efficient.

REFERENCES

[1] Caspeele, R.; Taerwe, L.; Combined production and conformity control of concrete with acceptance cusum control charts. In: Proceedings of the 7th International Probabilistic Workshop 2009, Gent, S. 73 - 86

ASSESSMENT OF CONCRETE COMPRESSIVE STRENGTH IN STRUCTURES

T A Harrison

Professor, Chairman of the ERMCO Environmental and Technical Committee

Abstract

EN 13791: *Assessment of in-situ compressive strength in structures and precast concrete components* is being revised by CEN and will be probably released in 2016. The changes to this standard are significant and it now covers two main topics: the determination of the characteristic in-situ strength for structural assessment purposes using EN 1990, Annex D; and where there are issues over the quality of concrete supplied, whether the concrete conformed to its specified compressive strength class. A standard is not a textbook and consequently the 'rules' are not fully explained. This paper describes the background to a number of the key issues addressed in EN 13791. In particular the background to the procedures for combining core and indirect test data to determine the characteristic in-situ strength and the in-situ strength at specific locations. The reasoning behind the different approach used to assess whether the concrete conformed to its specification is also explained.

Keywords: In-situ strength, assessment, concrete, cores, rebound number, pulse velocity, conformity, statistics.

1. INTRODUCTION

EN 13791: *Assessment of in-situ compressive strength in structures and precast concrete components,* was published in 2006 and at its 5-year review it was decided to revise this standard. The 2006 version of the standard determined the EN 206-1 concrete compressive **strength class** based on in-situ testing, but for the new version of the standard, it was agreed that one main target would be to determine not a class, but the **characteristic in-situ compressive strength** in terms of cores with a 2:1 length to diameter ratio as this is the input needed for designing in accordance with EN 1990, Annex D [1]. The objectives of the standard are two: While to obtain data for structural analysis and re-design is one prime purpose of EN 13791, there is another equally important role which is to provide procedures and criteria to determine by in-situ testing whether the supplied concrete conformed to its specified compressive strength class. The Committee decided to keep the two objectives within a single standard.

The 2006 version of EN 13791 did not provide any detail on what are common assessment procedures and so the scope of the standard has been significantly increased. New topics include:

- smaller diameter cores;
- check if the concrete belongs to two different populations;
- test for outliers;
- characteristic in-situ compressive strength from core test data only;
- characteristic in-situ compressive strength from combined core and indirect test data;
- screening test using rebound number to determine if the concrete conformed to the specified compressive strength class;
- relative testing where conforming structural elements are compared with similar elements where the quality of the supplied concrete is under investigation;
- much more guidance on investigations.

The changes from the 2006 version of the standard are so significant, it is better to regard the revised standard as being a new standard as only Annex B of the 2006 version has been retained unchanged. All references to the revised EN 13791 refer to the February 2015 draft of the standard (draft 14).

The Task Group revising EN 13791 has requested CEN permission to produce a CEN Technical Report explaining the background to the revision and including worked examples to guide users. It is anticipated that this request will be approved.

2. SMALLER DIAMETER CORES

When coring a structure, it is best to avoid cutting rebar and this is not always possible with the 100mm diameter cores. While EN 13791:2006 permitted cores with diameter down to 50mm, it did not provide any guidance on cores other than 100mm diameter cores; this task was left to national provisions. Technically the strength of smaller diameter cores is more variable, particularly if they have a 2:1 length to diameter ratio.

Experience has shown that in practice either 2:1 or 1:1 cores are used. As design to the concrete Eurocode is in terms of 2:1 cylinder strength, it was agreed that the in-situ compressive strength would always be expressed in terms of a 2:1 core irrespective of the diameter and any 1:1 core would be transposed to the equivalent 2:1 core by using a constant factor of 0.82, which is the average of the ratio between cylinder and cube strength in the compressive strength classes in

EN 206. For practical reasons a small tolerance on the length after capping is permitted, but the target length of the capped specimens should be either twice or the same length as the diameter. The diameter is the diameter of the core and not the hole. There are limits on core diameter related to the maximum aggregate size.

While 2:1 cores require no conversion factor, 1:1 cores are often more practical. To deal with the issue of higher variability with smaller diameter cores, cores less than 80mm in diameter are required to have a 1:1 length to diameter ratio and a number of cores taken at one location are averaged to give a single test result. At 80mm diameter or more, both 2:1 and 1:1 cores are permitted and a test result may be based on a single core.

The current version of EN 13791 required a period of drying in laboratory air prior to testing and this will enhance the strength of the core. The consensus view of TG11 is the moisture content of cores should be that found in-situ, as it is the in-situ strength that is being determined, so after coring the surface is dried with a paper towel and then the core is labelled and placed in a close fitting sealed container, e.g. a polythene bag.

3. CHECK IF THE CONCRETE BELONGS TO TWO POPULATIONS

While careful selection of test regions will minimise the risk of including two strength classes in a single population, it does not entirely exclude the possibility that the test region contains more than one compressive strength class. There is a requirement to check that this is not the case. Exactly how this is to be undertaken is not specified as it is often based on an inspection of the data from different elements. If the data looks as if it came from two populations, the data should be split and a statistical test applied to determine if this is true. An example of how this is done will be given in the Guide to EN 13791.

4. CHECK FOR OUTLIERS

The revised EN 13791 includes the use of a test to check for statistical outliers. Outliers are results that are about 3 standard deviations from the mean value and values that in a Gaussian distribution have a one in a thousand chance of occuring. The test may be applied twice to a set of data provided certain conditions are satisfied. If more than two test results are outliers, this may be an indication that the concrete comprises two populations. Guidance is provided on handling outliers and there is no presumption that they should be excluded from the data analysis. In all cases an outlier needs special consideration of its cause and how it should be handled. For example, if it represents a weak area of concrete that will be removed, it should not be included in the determination of characteristic in-situ strength. On the other hand a high outlier i.e. a too high strength, in air-entrained concrete may be an indication of insufficient entrained air. The standard simply provides the tool to determine outliers and the organisation or person appointed to review the data decides what to do about them.

5. CHARACTERISTIC IN-SITU COMPRESSIVE STRENGTH FROM CORE TEST DATA

Where using core test data only to determine the characteristic in-situ strength, the core locations are selected to represent the average quality of the in-situ concrete. Guidance is provided on selecting core locations.

The approach to determining the characteristic in-situ strength has been changed to align with that in EN 1990:Annex D: *Design assisted by testing,* except that the revised EN 13791 does not use the log-normal version of the equation. The equation is now based on the t-statistic at 95% probability and is:

$$f_{c,is,ck} = f_{c,m(n)is} - t_{0.05} s_n \sqrt{(1 + (1/n))}$$

As the number of core test data is often low, there is a risk that the sample will yield an unrealistically low standard deviation (s_n) and so a minimum value of it, 3.0 N/mm^2 , is specified and this is independent of the mean strength.

6. CHARACTERISTIC COMPRESSIVE STRENGTH BASED ON COMBINED CORE AND INDIRECT TEST DATA

The revision of EN 13791 includes three procedures for using a combination of indirect tests and core tests depending upon the number of pairs of data. A pair of data is where there is both an indirect test measurement, e.g. ultra-sonic pulse velocity, and a core test result from the same location. For 12 or more pairs of data a correlation between the 2 sets of results is determined, for 11 to 6 pairs of data the mean values are used to shift a standard curve given in EN 13791and for five to three pairs of data a non-statistical procedure is used. EN 13791:2006 provided some standard curves but these have been revised based on comprehensive test data supplied by a Swiss testing materials equipment company.

While EN 13791:2006 provided means for converting indirect test data, e.g. ultra-sonic pulse velocity and rebound number, into in-situ compressive strength, there was no guidance on how to convert the estimated in-situ strength values into an in-situ characteristic strength. Handling such data is not as simple as it may first appear. This is best illustrated with an example from practice. A structure was surveyed using a rebound hammer and then selected cores were taken to determine the correlation between the rebound number and the in-situ strength. The characteristic in-situ strength determined using the core test data only was 14.0 N/mm^2 but when determined using both the core data and the in-situ strengths estimated from rebound tests at locations where there was no core test data it was 18.9 N/mm^2. Neither of these values give the correct characteristic in-situ strength for the following reasons.

When determining the correlation, there is a requirement to take cores over the whole range of indirect test values, if safe to do so. This is to ensure the best correlation between the results from the two procedures, and there is a limit of 5 N/mm^2 on extrapolation of the correlation. Figure 1 shows a typical set of data. Unsurprisingly, more cores have been taken where the indirect test indicated low strengths. There are data over the whole range, but the core data set does not represent the population of results (it is under-representative of the average concrete quality) and this data set gives a higher standard deviation than the population as a whole. When combined with the $\sqrt{(1 + (1/n))}$ term, this results in an unrealistically low characteristic in-situ strength.

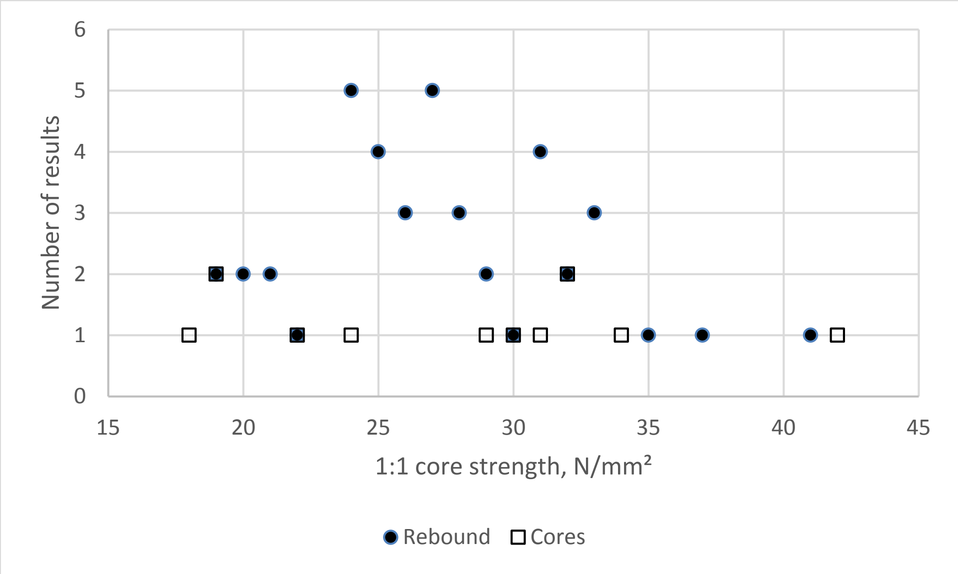


Figure 1: Distribution of strength in an element under investigation

On the other hand, simply using the correlation curve to detremine the in-situ strength at locations where there is no core result and calculating the mean and stanadrd deviation of these values leads to an unrealistically high characteristic in-situ strength for the following reason. Figure 2 shows that the actual pairs of results are spread around the correlation. For every location where there is only an indirect test value, this test value is converted to an estimated in-situ strength using the correlation curve equation and these values will lie exactly on the correlation curve. In reality these results would be spread around the correlation curve and consequently by using the correlation curve the spread of results is under-estimated and the characteristic in-situ compressive strength is over-estimated.

The equations provided in the revised EN 13791 take account of this under-estimation of the variability in the transposed indirect test results when calculating the in-situ characteristic compressive strength. When this is done the characteristic strength falls between these two values given earlier and for the same example is a value of 16.5 N/mm^2 (greater than 14.0 N/mm^2 but less than 18.9 N/mm^2).

The correlation between the indirect test value and the in-situ compressive strength is a mean to mean relationship meaning there is a 50% probability that the actual strength is lower than the estimated strength. Therefore the use of such an estimated value of in-situ strength is not safe when assessing the performance of the structure at a specific location. With correlations based on relatively low numbers of pairs of results the use of the 90% confidence limit is not sufficiently safe [2] and so EN 13791 requires the use of what is known by statisticians as the prediction limit to determine the in-situ strength at a specific location, see Figure 2.

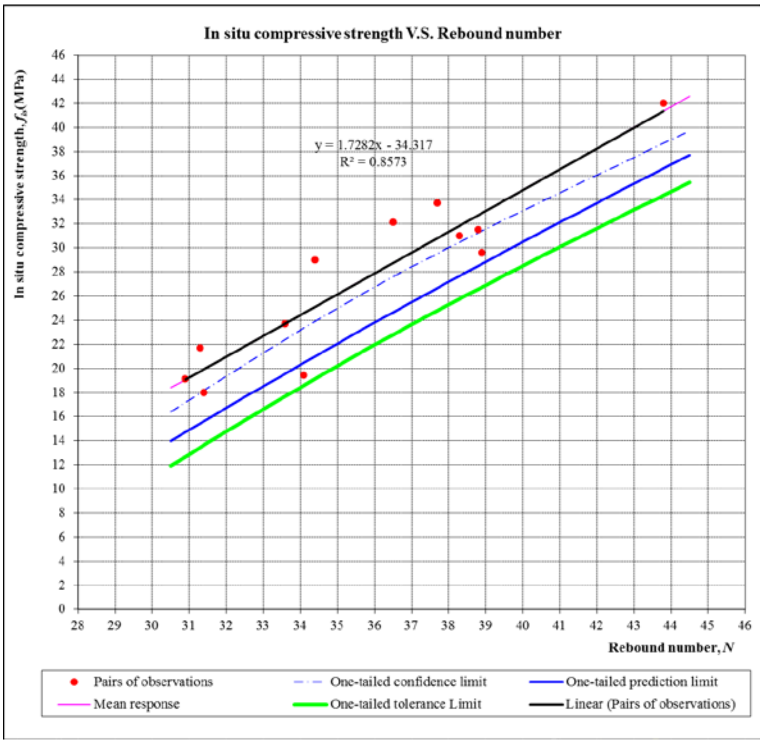


Figure 2: Illustration showing why the 90% confidence limit is not safe
(Figure by courtesy of INEC)

When there are insufficient pairs of data to determine a reliable relationship, the given standard curves may be used. For the pairs of data, the mean in-situ strength and the mean indirect test value is determined. The standard curve is then shifted vertically to pass through this point. This shifted curve is then used to determine the in-situ strength for all the test locations where there is only an indirect measurement. The way in which the data are used to determine the characteristic in-situ strength is the same as for a correlation.

Where the strength at a specific location is required, the prediction limit is needed but there are not sufficient data to determine reliably the prediction limit. Consequently the strength at a specific location is based on empirical rules given in EN 13791.

When there are five to three pairs of data, a simple non-statistical approach is applied. After the indirect test survey, the average quality areas are identified and cored. The mean of the core values, provided the spread is not more than 15%, is taken as the characteristic in-situ strength. This rule does not apply where there are issues over the quality of concrete supplied. When detremining the characteristic in-situ strength or the compressive strength at a specific location within the structure, there is no assumption about the value and the procedures lead to safe design values. When checking whether the concrete conformed to its specified compressive strength class the assumption is that it conformed and for a small volume of concrete, e.g. concrete where only a few cores are taken, if its mean strength is within the accepted tail of the strength distribution, the concrete is accepted as conforming to its specified strength class.

7. ISSUES OVER THE QUALITY OF CONCRETE SUPPLIED

Clause 9 of EN 13791 is for the situation where there are issues over the quality of concrete supplied. If the concrete producer has declared non-conformity, clause 9 does not apply and the procedures given in clause 8 to determine the characteristic in-situ compressive strength and, if necessary, the in-situ compressive strength at specific locations may be used as input for a structural assessment of the impact of the non-conformity. Where there are differences, for example, between the producer's conformity control and identity testing, the null hypothesis is that the concrete conformed to the specification, and the tests are used to determine whether this hypothesis is correct. The null hypothesis is the hypothesis that there is no significant difference between specified populations, any observed difference being due to sampling or experimental error.

The concrete under investigation is treated as a 'lot' and the average quality of the 'lot' may be determined using core testing or combined indirect and core testing to determine the mean strength of the lot. Multiplying the average strength by a factor 1.18 (=1/0.85) to convert the in-situ strength to the equivalent strength of test specimens, this value is compared with the limit for the specified compressive strength class. The factor of 0.85 is the part of the partial safety factor for concrete that is attributed to differences between the strength of test specimens and the in-situ compressive strength. For small volumes, e.g. 25m^3, and for compressive strength classes ≥C20/25, the strength of the lot is compared with the lowest acceptable strength, i.e.:

$$1{,}18\ f_{c,m(n)is} \geq (f_{ck,spec} - 4)$$

Where the compressive strength class is less than C20/25, the margin of 4 is reduced. This criterion is accepting concrete that is in the tail of acceptable strengths. For large volumes of concrete, it is reasonable to expect that at least the characteristic strength is achieved and thus the criterion becomes:

$$1{,}18\ f_{c,m(n)is} \geq f_{ck,spec}$$

8. ALTERNATIVE APPROACHES

While cores give the most reliable measure of in-situ strength, it is often more convenient, quicker and less expensive to use one of the other procedures provided in EN 13791. EN 13791 provides two options that do not involve core testing: a) a screening test using the rebound hammer and b) relative testing.

9. SCREENING TEST USING THE REBOUND NUMBER

EN 13791 has adopted a screening test using the rebound number that has been widely used in Germany. The screening test is a safe relationship between rebound numbers taken in-situ and the compressive strength class of the supplied concrete. If the concrete in the structure satisfies the given criteria, it may be assumed that the concrete conformed to its specified compressive strength class; however, failure to meet these criteria is insufficient proof that the concrete was not conforming and one of the other assessment procedures has to be used. The basis for these criteria are given in Figure 3.

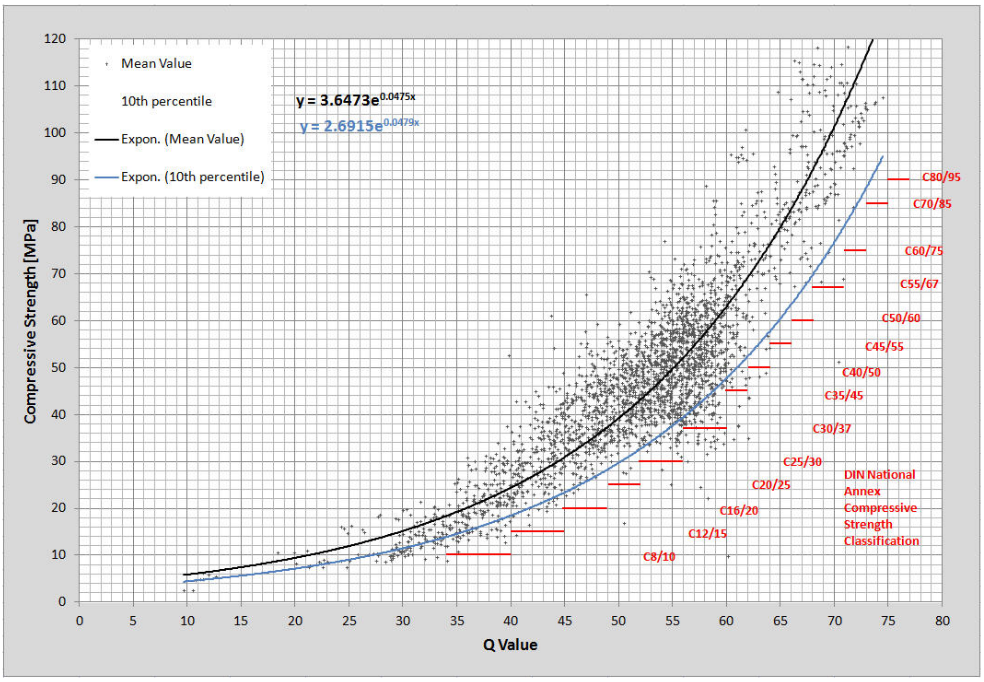


Figure 3: Relationship between cube strength and rebound number (Type Q)
(Figure by courtesy of Proceq)

10. RELATIVE TESTING

A procedure that has been used successfully and now adopted within EN 13791 is the use of relative testing. This is where the element under investigation is compared with a similar element of accepted quality. The use of the rebound number or pulse-velocity are ideal for use in this procedure.

As the null hypothesis is that the two elements have the same concrete quality, the fewer the data the less is the chance of proving a difference. Consequently EN 13791 recommends that at least 20 pairs of data are obtained. The criteria for a comparison of twenty pairs of data are provided in the standard but any (higher) number may be compared using standard statistical tests for difference in mean value.

11. MORE GUIDANCE

Whilst drafting the revision of EN 13791, a need for more guidance was identified. A clear difference between the procedures necessary to obtain a test result, characteristic in-situ strength, strength at a specific location or to identify an outlier (the normative text) and guidance on what to do with these data (in informative annexes) is made in EN 13791.

One of the new annexes describes the differences between test specimens and the concrete in the structure. We are hoping that CEN TC250/SC2, the European Concrete Design Committee will confirm what is included in the partial safety factor for concrete, as the factors in the concrete Eurocode were based on calibration with existing design methods and not on a fundamental analysis of the factors involved. This is more than an academic issue as there is a rare possibility that the concrete in a structure may be assessed as conforming to its specification, but from a structural analysis viewpoint prove to be inadequate. By working

together on these issues we may be able to find a solution, we should be able to produce better standards with a clear understanding of what is included in each of the factors.
The Task Group that prepared the draft of EN 13791 have plans to produce a CEN Technical Report setting out the background to the revision and examples of the calculations.

12. CONCLUSIONS

The revised draft of EN 13791 if positively voted upon will provide a more comprehensive and useful standard for the assessment of concrete compressive strength in structures and precast concrete components.

REFERENCES

[1] CEN, *Eurocode — Basis of design*, EN 1990, 2002 + A.1: 2005.

[2] DE GRYZE S, LANGHANS I and VANDEBROEK M, *Using the correct intervals for prediction: A tutorial on tolerance intervals for ordinary least-square regression,* Science Direct, Chemometrics and intelligent laboratory systems, 87 (2007) p147-154.

REALTIME MONITORING OF HYDRATION TEMPERATURE AND COMPRESSIVE-STRENGTH-DEVELOPMENT OF CONCRETE

Alexander Reinisch (1), Wilko van der Meer (2) and Toine van Casteren (2)

(1) Doka GmbH, Austria

(2) BAS Research & Technology, Netherlands

Abstract

One of the future trends in building is to making processes more and more efficient to save money and to achieve a high level quality result.

For optimizing a process it is essential to measure the relevant parameters. Real-time concrete hydration temperature and strength monitoring is an efficient instrument for optimizing the building process. The compressive strength development of concrete is one of the most relevant parameters in the construction process. Also the monitoring of the temperature development is useful to minimize the risk of early age thermal cracking.

The innovative real-time concrete monitoring system is a combination of the state-of-the-art maturity method by de Vree and modern mobile communications technology.

This tool is also a great opportunity for the ready-mix-concrete industry to increase the sustainability of concrete by using performance based concrete.

Keywords: Building and construction process, process and concrete optimization, real-time monitoring, quality management, compressive strength, increasing quality and sustainability, integral building, performance based concrete;

1. INTRODUCTION

1.1 Trends in the building industry and construction process

For several years the construction industry has been in a state of structural change. Cost and time-pressure are increasing constantly, both for construction companies and their suppliers. The requirements for quality, defined by the contractor of the building, remain on the same level. In some aspects of construction work, the requirements for quality are increasing.

The main trends for the future of the construction industry are going in the direction of increasing speed and same or lower prices of construction work and materials by increasing quality and sustainability.

In some cases it will be a big dilemma to solve this problem of low costs and high quality.

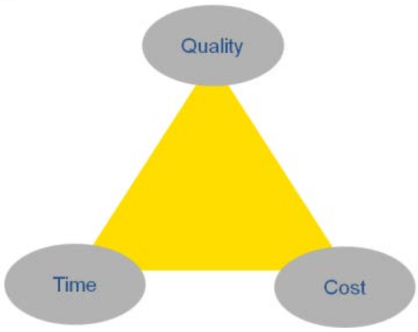


Figure1: Cost quality trade-off triangle

The classical construction process is nearly like a prototyping process, because every building is different in its architectural configuration, space on site, weather conditions during construction, construction design and timetable. There is almost no high-level quality management system established, as in other industries with a high level of industrialization, like high level of standardization, and with parameters changing less frequently.

So there is a big opportunity, for the construction company and for the contractors, to optimize the construction process and save money.

1.2 A new innovative solution for the construction process

To optimize the process, you have to increase the efficiency of the construction process. Parameters or measured values are required to make the efficiency of a process visible and to be able to control it. One of the most important parameters in this instance is the compressive strength development of concrete in its construction state. Striking times, curing times, time of pre-stressing etc. are derived from the early concrete strength development. The real-time concrete monitoring system Concremote® enables the user, online anywhere in the world, to make decisions based on values measured direct in the structure and not small in cubes and cylinders.

2. TECHNOLOGY

2.1 Function and use of concrete monitoring

The concrete monitoring system Concremote® is a service for the non-destructive real-time measurement of the concrete strengths present in a structural member (floor, wall, beam....) etc. on the construction site. The service consists of two components

- Measuring sensors (slap sensor, wall sensor + cable) (Figure2)
- Services (data management, data processing (strength calculation), data output (web portal))
- The sensors are either attached to the formwork prior to pouring the concrete or are placed on the structural member immediately after concreting of the freshly poured concrete.

Figure2: Reusable slap sensor on the upper face of the slap and cable sensor in the wall area

- The sensors continuously measure the temperature of the concrete, which is mainly influenced by the hydration heat of the cement and the ambient temperature. The faster the generation of the heat, the faster is the strength development of the concrete.
- In a further step, the measured data (temperature measurements) of the slap or wall are transmitted hourly, in the form of a data packet, via the mobile telephone network to a data center, where in accordance with the maturity method it is analysed automatically by using the calibration measurement.
- A separate calibration measurement is necessary for each type of concrete that is measured on the construction site. This calibration measurement can be performed either by the customer himself, the concrete supplier or an appointed testing institute, with a specially developed calibration box. In this process six cubes are stored under defined semi-adiabatic conditions. The cubes are tested at different times, depending on the customer's target value (N/mm² for striking, curing....). A compressive strength value with a relevant temperature value is obtained from this. Based this calibration measurement, the relationship between strength and maturity is determined for the concrete in question.
- The measured and analyzed data and strengths are made available continuously and in real-time to the customers by way of a secure web portal. This way the user can perform live monitoring of the strength development in the structural member.
- If the target value, defined by the customer (e.g. N/mm² for striking, curing, etc...) here been reached, he can make his decision based on the real-time data.

Figure 3: Process of concrete monitoring system

2.2 The maturity method by de Vree

The method for determining the concrete strength on the basis of the maturity of the concrete was technically established over 30 years ago. The best known methods include the determination of maturity according to De Vree, which is also used by Concremote®. Calculation of the weighted maturity is performed as follows:

$$\boldsymbol{Rg = 10 * \frac{[C^{(0,1T-1,245)} - C^{(-2,245)}]}{\ln C}}$$

Rg= weighted maturity per hour [C°h]
T= mean hardening temperature of the concrete in one hour
C= reactivation parameter of the cementing agent

To determine the concrete maturity, the weighted maturities are totaled every hour. [1] A compressive strength value is assigned to each maturity on the basis of the strengths derived from the calibration measurement.

The method for determination of the concrete strength by way of the maturity method is dealt with in the following technical documents and standards:

- DBV-Merkblatt, Betonschalungen und Ausschalfristen, 2006 [2]
- DIN 1045-3,Tragwerke aus Beton, Stahlbeton und Spannbeton - Part 3, 2008 [3]
- NEN 5970, Determination of strength of fresh concrete with the method of weighted maturity, 2001 [4]
- TS 13508, Estimation concrete strength by the Maturity Method, 2012 [5]

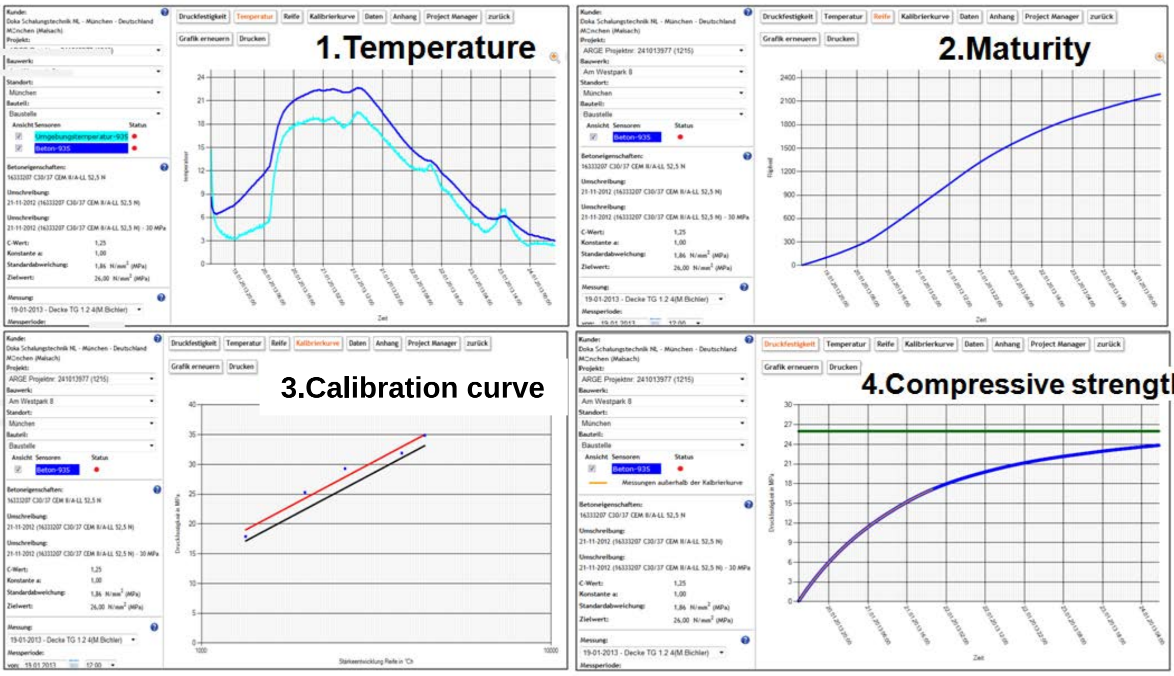


Figure 4: the maturity method

2.3 Application and technical and economic benefits

By using the real-time concrete monitoring it is possible, on the basis of the measured strength data, to render the construction process safe and to optimize or accelerate it by means of the corresponding measures. Concrete Monitoring Concremote® can be used for the following applications:

- Targeted determination of the time of striking optimization of the cycle times – possibly reduction of the cycle times
- Process dependability - decisions based on measured values and not on estimated values
- Determination of the curing time via the measured strength development
- Safety when using climbing formwork
- Measurement of the hydration heat development in massive structural members to prevent cracks
- The possibility of season related adaptation and optimization of the concrete mix design through continuous measurement of the compressive strength development (e.g. with slow strength development in the winter - change over to more rapid strength development for concrete)

3. CONCLUSION

Real-time concrete monitoring is an innovative process that allows making decisions based on proof measurements of the compressive strength in the building component. This technique allows the site manager to use real measurements instead of estimations.

Concrete monitoring is another step forward to 'Integral construction philosophy', including the effect of safeguarding and optimization of the process.

Especially for optimizing the concrete mixture, that is for using the needed mass of cement (clinker) – not more as needed – this method includes a high benefit for the construction company and the contractor. Real-time concrete monitoring is an efficient quality management tool and an opportunity to go in the direction of using ´Performance Based Concrete mixture´.

Concremote also makes the in-situ concreting method of construction, and therefore ready mixed concrete, more attractive for the user due to the process optimization possibilities created by it.

REFERENCES

[1] de Vree, R.; Tegelaar, R.: Gewichtete Reife des Betons, Beton 48 (1998), Issue 11

[2] DBV-Merkblatt, Betonschalungen und Ausschalfristen, 2013

[3] DIN 1045-3, Tragwerke aus Beton, Stahlbeton und Spannbeton – Teil3, 2008

[4] NEN 5970, Determination of strength of fresh concrete with the method of weighted maturity, 2001

[5] TS 13508, Estimation concrete strength by the Maturity Method, 2012 [5]

[6] Reinisch, A.: Zerstörungsfreie, onlinebasierende Betonfestigkeitsmessung auf der Baustelle, Tagungsband Baukongress 2014, ÖBV, Wien 04.2014

[7] Reinisch, A.; van der Meer, W.: Realtime Monitoring of Compressive-Strength-Development of Concrete, 10th CCC Congress Liberec 2014

*the content of this paper was first published at CCC Liberec [7]

CONCRETE RHEOLOGY CHARACTERIZATION: AN EASY WAY

S.Moro (1), R.Magarotto (1), F.Moratti (1) and G.Aykan (2)

(1) BASF Construction Chemicals Italia Spa, Italy

(2) BASF Türk Kimya Sanayi ve Ticaret Ltd. Sti., Turkey

Abstract

Fresh rheoplastic concrete usually is characterized on the job site with standardized methods such as Slump, Slump Flow or VEBE Test. The obtained values are used to describe both the initial fluidity of the concrete and the workability retention, but these methods are not able to characterize the rheological properties of concrete which may have an important impact on its production.

The rheology improvement of rheoplastic concrete thanks to more advanced superplasticizers allows the solution of difficult situations where the higher concrete stickiness impedes easy placing and affects the quality of the surface.

From the mixing time to the surface finishing, through the pumping and placing, the concrete rheology plays a key role on the construction life, but its quantification in each steps is not an easy task.

In this paper, the rheology of concretes cast with different superplasticizers is quantified by new test methods able to simulate and differentiate the placing step: the laboratory trials validate the test methods and the best performing admixtures.

Keywords: Concrete, superplasticizer, rheology, placing, surface, test method

1. INTRODUCTION

Concrete technology has already achieved almost the limit in terms of “traditional” performances: it is possible to produce members with extreme mechanical and flexural properties, to pour concrete that retains workability for hours under the most severe conditions and to assure very high performances in terms of durability and aesthetics.
But pushing concrete towards such extreme implies working with a material that increasingly becomes difficult to handle, to move, to pump, to pour and to finish.
New performances, other than water reduction and workability retention, are required today to meet the customer requests and to enter a new part of the market (labor) where the contribution of superplasticizer is not fully exploited.

Rheology is, by definition, the study of the flow of matter, primarily in the liquid state, but also as “soft solids” or solids under conditions in which they respond with plastic flow rather than deforming elastically in response to an applied force. As a colloidal system, rheology of concrete can be expressed by the Bingham model. A Bingham fluid is a viscoplastic material that behaves as a rigid body at low stresses but flows as a viscous fluid at high stress. The yield stress (τ_0) determines the value when concrete begins to flow under its own mass. The plastic viscosity (μ) determines the flow time or speed of concrete during pouring or pumping. This value indicates how easily the concrete can be placed or filled into forms.
The traditional methods of measuring slump or slump flow are not capable of characterizing the fundamental rheological properties of concrete during the processes of mixing, transporting, and placement.

Rheometers for concrete are designed to characterize the static yield stress, the dynamic yield stress and plastic viscosity of the concrete. A high static yield stress is desirable because it reduces formwork pressure and increases the resistance to segregation. But for ease of pumping, placement, and self-consolidation, a low dynamic yield stress is necessary. The dynamic viscosity provides cohesiveness and contributes to reducing segregation when concrete is flowing.

Till now, the tests performed in lab and job site were focused on evaluating the workability of fresh cementitious materials by measuring a change in slump or slump flow. These industrial tests are, in most circumstances, directly correlated to yield stress. Although cementitious materials are not only yield stress fluids, this pragmatic approach was justified by the fact that yield stress is often the most relevant parameter to describe the ability of a material to fill, under its own weight, a formwork or more generally a mold.

Recent trends in modifying mix design (reducing W/C ratio, adding harsh aggregates, etc.) have shown, however, dramatic consequences on the workability of the material and workers at the building site often complain about these “sticky” concretes that they are unable to vibrate and surface finish. This “stickiness” and, more specifically, the additional stress needed to place the material, is not only related to yield stress but also to plastic viscosity.
Therefore, a reduction of both yield stress (τ_0) and plastic viscosity (μ) contributes to improving rheology of concrete.

The practical evaluation of stickiness and rheology, in general, is a tough task, being rather difficult to attribute a single number to the “perceived feeling” of rheology that customers have.

Usually, rheometer is used to measure yield stress and plastic viscosity values but both these numbers are not so directly and easily correlated to concrete properties measured on the job site and not all job sites are typically equipped with this instrument. Some test methods at laboratory scale have been developed [1] [2], but new tools have to be developed with the aim to simulate some aspects of the work at the job site, like placing, vibration and finishing. Moreover, they have been thought to be portable.

"Placeability" is used herein as a measure of the ease of moving concrete after pouring. In this paper a new method for characterizing placeability is reported: a special formwork which is equipped with a probe was developed, where probe shape, size, depth in concrete and displacement rate can be modified in order to achieve the best simulation of the placing in the real formwork. The use of different admixture could strongly impact the behavior of the concrete in this respect [3].

2. TEST METHODS

2.1 Rheometer

Generally the rheological characterization of concrete is carried out with rheometers. Different examples are available in the market: the instrument used for this study is the RHM-3000 ICAR Rheometer (Germann Instruments) (see figure 1).

Figure 1: RHM 3000 ICAR Rheometer

The yield stress and the plastic viscosity are measured by a vane able to rotate in a specific and proper designed container:

- The first parameter is proportional to the maximum torque detected when the vane constantly rotates at 0.025 rev/s
- The second parameter derives from the slope determination (according to specific calculations) of the interpolation line of six torques measured at six decreasing rotational speeds from 0.6 to 0.1 rev/s.

2.2 Portable placing simulator

The aim of this device is to assign a number to the feeling of the skilled operator who can detect differences between concrete mixture by moving them with a trowel but is nor able to quantify them.

The developed portable tool quantifies the torque needed to move an embedded paddle in the concrete through a pulley system (the figure n°2 explains the entire equipment).

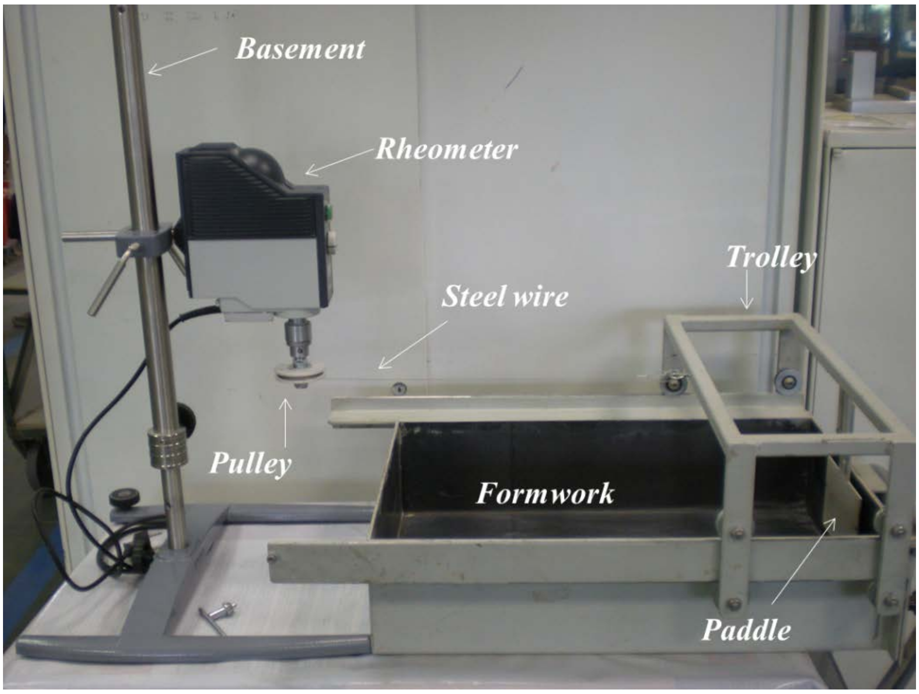


Figure 2: Placing Simulation Tool

The formwork is filled with 30 liters of concrete: the embedded paddle connected to a wheel trolley moves via a steel wire while the pulley constantly rotates through the rheometer (RZR 2102-Control by Heidolph). The torque evolution is recorded by a laptop connection and relative acquisition software: figure n°3 shows an example

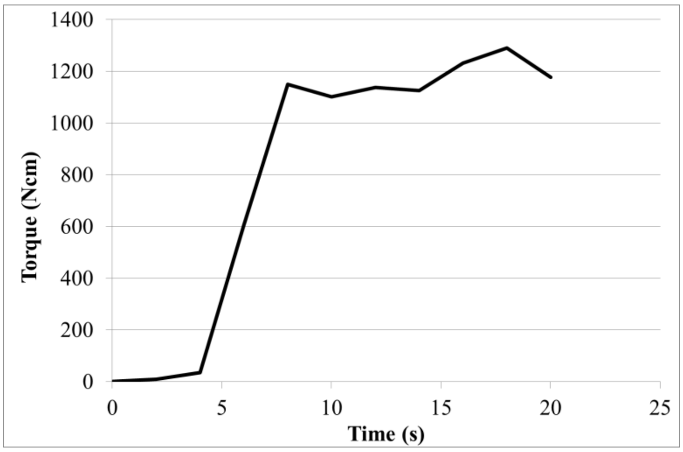


Figure 3: Example of torque evolution recorded

The reproducibility of the test method has been fully assessed: after approximately hundred different concrete repetitions the standard deviation among the tests is 8%.

The limit of the test method can be identified in the minimum consistency class necessary in order to have reliable measurement: below 10-12cm slump the measured torque exceeds the maximum value of the rheometer.

3. MATERIALS AND MIX DESIGN

The tests have been performed comparing two superplasticizers having different chemistry:

- Polycarboxilate ether (PCE); 30% solid content solution; density 1.040gr/cm^3
- Polyaryl ether (PAE); 30% solid content solution; density 1.080gr/cm^3

PAE chemistry is a technology for which BASF filed patent applications [4] which proved to be very effective in improving rheology of concrete.
Limestone Filler, dolomite crushed Sand 0/4, Coarse Aggregates 8/12 and 12/19 have been used in this investigation: Table 1 explains the characteristics of aggregates

Table 1: Aggregates Properties

	Limestone Filler	Sand /04	Coarse Agg. 8/12	Coarse Agg. 12/19
Density (kg/m^3)	2710	2741	2754	2753
Water Absorption (%)	NA	0.9	0.8	0.8
		Cumulative Sieve Retention (%)		
16mm				66
12.5mm			87	98
8mm			98	100
4mm		8	100	100
2mm		29	100	100
1mm		46	100	100
0.5mm		56	100	100
0.25mm		72	100	100
0.125mm		89	100	100
0.063mm	100	98	100	100
Fineness Modulus	NM	3.00	6.87	7.66

The experiments have been carried out with 3 different Italian cements:

- CEM II/A-LL 42.5 R Monselice
- CEM II/A-LL 42.5 R Colacem Gubbio
- CEM IV/A-P 42.5 R Colacem Gubbio

All cements reflect the limits according the EN 197-1: the average 28dd compressive strength is between 42.5 and 52.5 MPa.

The higher concrete stickiness is generally observed when higher fines (cements, binders, sands) and low water/cement ratio compose the cementitious matrix, so the materials have been combined according to the mix described in the table 2, where 400kg/m^3 of cement and 50kg/m^3 of limestone filler have been blended in w/c = 0.48 concrete and the admixtures have been dosed in order to achieve 220±10 mm slump. The same table also reports the concrete fresh properties measured for each combinations.

Table 2: Mix design & concrete fresh properties

Sand 0/4	1050kg/m^3					
C.Agg. 8/12	364kg/m^3					
C.Agg 12/19	405kg/m^3					
L.Filler	50kg/m^3					
W/C	0.48					
Cement	400kg/m^3					
Cement Type	CEM II/A-LL 42.5 R Monselice		CEM II/A-LL 42.5 R Colacem Gubbio		CEM IV/A-V 42.5 R Colacem Gubbio	
PCE (% b.w.c.)	0.70		0.80		0.85	
PAE (% b.w.c.)		0.75		0.80		0.85
Slump (mm)	225	220	215	225	220	225
Air (%)	1.8	1.9	2.1	2.0	2.1	2.2

Apparently all concretes seem similar, because the measured slump and air content do not different so much.

However the rheological evaluation shows how the different admixture chemistry has an impact on concrete stickiness and above all on the feeling of the operators.

Both the ICAR Rheometer and the Placing Simulator assessments have been carried out for each concrete: the yield stress and the plastic viscosity have been compared towards the maximum torque detected by the new equipment.

Figure 4 show the different behavior of the torques measured in the CEM II/A-LL 42.5 R Monselice serie.

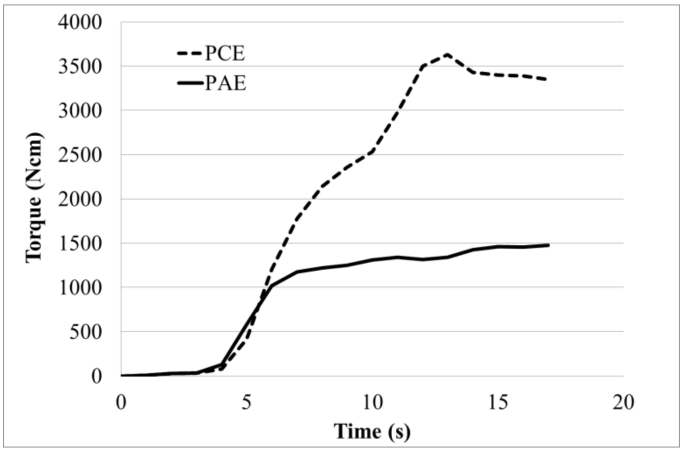


Figure 4: Placing Simulator (CEM II/A-LL 42.5 R Monselice)

PAE based admixture is able to reduce the max torque while the PCE reaches a torque value 3 times higher than the new molecule.

The Table 3 summarizes all values measured and demonstrates as the Placing Simulator is in line with the results and differences obtained by the ICAR Rheometer.

Table 3: Rheological parameters

Cement Type		CEM II/A-LL 42.5 R Monselice		CEM II/A-LL 42.5 R Colacem Gubbio		CEM IV/A-V 42.5 R Colacem Gubbio	
Admixture		PCE	PAE	PCE	PAE	PCE	PAE
ICAR Rheometer	τ_0 (Pa)	1242	917	1144	1058	980	1241
	μ (Pa*s)	175	90	220	112	250	115
Placing Simulator	Torque (Ncm)	3510	1500	4221	1740	4310	1650
Operators' Feeling (Stickiness detected by trowel)		The Higher	The Lowest	The Higher	The Lowest	The Higher	The Lowest

In all cases the Placing Simulator results demonstrate that PAE technology reduces the maximum torque and this trend is in line with the differences of plastic viscosity (μ). On the other hand not substantial differences can be observed from the yield stress (τ_0): the behavior can be easily explained by the fact that this parameter is more correlated to the slump, and because all concretes have similar workability, the yield stress values reflect it.
The availability of new chemistry and its capability on rheology improvement allows some considerations:

- Better Rheology approach – Concrete stickiness reduction
- Durability Improvement approach - The water content can be reduced improving both mechanical properties and durability
- Cost Optimization approach - The cement content can be reduced maintaining the same water-cement reduction and the mechanical properties by consequence

The following example (Table n°4) explains the last approach and the results measured with both Rheometer and Placing Simulator confirm the possibility to optimize the cost maintaining the same properties of daily concrete:

Table 4: Cost Optimization approach example

CEM II/A-LL 42,5 R Monselice		400kg/m^3	370kg/m^3
Admixture		PCE (2,8 l/m3)	PAE (3,0 l/m^3)
W/C		0,48	0,48
Slump (mm)		225	220
Air Content (%)		1,8	2,0
ICAR Rheometer	τ_0 (Pa)	1242	1145
	μ (Pa*s)	175	186
Placing Simulator	Torque (Ncm)	3510	3600
Operators' Feeling (Stickiness detected by trowel)		Similar Stickiness	

30kg/m^3 of cement can be reduced maintaining the same water-cement ratio (mechanical properties) and rheology.

4. CONCLUSIONS

New performances, other than water reduction and workability retention, are required today to meet the customer requests and to enter a new part of the market (labor) where the contribution of superplasticizer is not fully exploited and rheology improvement is one such new performance.

Its characterization can be obtained with rheometers, but in most of the cases the values (yield stress and plastic rheology) cannot be understood by all operators who work in the concrete production field.

The development of an easy, comprehensive and portable tool that can correlate the science of rheology (rheometer) with the concrete life (operators' feeling) is a must: the Placing Simulator is able to bridge these two realities.

The quantification of the stickiness reduction due to the new PAE technology can be easily measured by the Placing Simulator supported by the Rheometer avoidance.

The rheology improvement achieved by PAE polymer allows one to enhance the concrete performances in terms both of durability improvement (lower water content) and cost optimization (lower cement content).

REFERENCES

[1] Ferrari, C.F., de Larrad, F., 'Modified slump test to measure rheological parameters of fresh concrete', Cement, Concrete, and Aggregates, CCAGDP, Vol. 20, No. 2, Dec. 1998, pp 241-247

[2] Magarotto, R., Moratti, F., Moro, S., Vierle, M., Zeminian, N., 'Practical test methods for concrete rheology evaluation', International Congress on the Chemistry of Cement (ICCC), Montreal, Canada (2013)

[3] Kraus, A., Mazanec, O., Dengler, J., Hillesheim, N., Bokern, J., 'Influence of PAE, SMD and PCE superplasticizers on the rheological properties of mortars and concretes', International Conference "Application of Superabsorbent Polymers and other new Admixtures in Concrete Construction", Dresden, September (2014)

[4] International patent application "WO 2010/040612 A1, 15th April 2010

INTRODUCING A NEW CLASS OF SUPERPLASTICIZERS FOR HIGHLY VISCOUS CONCRETE MIXES

Jan Kluegge (1) and Gulnihal Aykan (2)

(1) BASF Construction Solutions, Germany

(2) BASF Turk Kimya Sanayi ve Ticaret Ltd. Sti., Turkey

Abstract

Since comb polymers based on PolyCarboxylicEthers (PCEs) have been introduced, they shifted the limits of achievable concrete performance dramatically. Today we are able to build higher, stronger and more durable than ever before. As a consequence of the outstanding water reduction and early strength performance of PCEs the industry was enabled to lower W/C ratios, reduce cement contents and utilize more SCM in the concrete mixes. However, these optimized concrete mixes are prone to increased concrete viscosity and stickiness of the concrete.

We have developed a new superplasticizer technology based on comb polymers featuring aromatic backbones: PolyArylEthers (PAE). Compared to conventional PCE, these new admixtures are able to reduce the thixotropy of concretes. In addition, a good early strength development and outstanding water reduction is maintained. More than 25 years after introducing the first PCE into the industry, PAE provides a breakthrough of rheology performance improvement for concrete.

High viscosity of fresh concrete has a big impact on its pump-ability, spray-ability, place-ability and surface finishing. Using PAE based admixtures equip concrete with appropriate rheological properties that help save placing time and labor costs and are therefore highly relevant for the construction industry. The workability of fresh concrete is usually characterized by its slump and slump flow values. However, workers often notice significant differences when handling concrete prepared according to different recipes, even if they exhibit very similar workability. Especially concretes with high SCM content that are treated with PAE are less sticky and can be placed with low effort. This behavior is quantified by comparing yield values, plastic viscosities and thixotropy and is characterized by using V-funnel and rheometer tests. The advantages of this technology are exemplified on mixes optimized for their sustainability and compared with conventional mix designs based on a life cycle assessment.

Keywords: Sustainable construction, high range water reducers, life-cycle analysis, admixture innovation, carbon footprint, concrete performance

1. INTRODUCTION

Global warming as a consequence of the increasing emission of carbon dioxide is considered to be a major challenge for mankind. One significant source of human CO_2 emission is the production of Portland cement [1]. For every ton of Portland cement produced, approximately 900 kg carbon dioxide is released into the atmosphere [2]. In 2010, around 3.3 billion tons of cement was produced, which corresponds to more than 3 billion tons of CO_2 and the demand for cement is expected to continue to grow with the global population and the development of urban centers. It is therefore not surprising that construction industry is experiencing increasing pressure to reduce their CO_2 emissions in order to become more sustainable. Consequently, modern concrete technology aims at minimizing the need for Portland cement without sacrificing the strength and the durability of the hardened concrete.

Superplasticizers allow maintaining the workability of fresh concrete with less water and hence help significantly increase the strength after hardening. Stronger concrete allows slimming down structures and thus decreasing the Portland cement consumption. A second important lever to reduce the content of Portland cement in a concrete is the use of supplementary cementitious materials (SCM). However, (partially) replacing Portland cement by SCM often leads to a loss of (initial) strength that is usually compensated by lowering the water cement ratio. The resulting workability drop can again be compensated by the utilization of superplasticizers. However, with high loads of fillers and SCM in the concrete mix design, state of the art PCE based superplasticizers are reaching technical limits: Although PCE based superplasticizers manage to establish the desired slump and water reduction, the concrete becomes very “sticky” and has a tenacious appearance. The concrete appears viscous and seems to have a higher cohesion, which negatively influences its pump ability, spray ability, place ability and surface finishing. Interestingly, not all superplasticizers have the same negative impact on rheology. Thus, it is very important to choose the right superplasticizer type in order to avoid undesirable stickiness.

This work presents a novel class of superplasticizers, developed by BASF that provides excellent rheological properties even in demanding mix designs. At the same time it maintains known advantages of PCE based superplasticizers like high water reduction capability, high early strength and tune-able slump retention properties. These new dispersants differ from conventional superplasticizers in the chemical nature of their polymer backbones. Like conventional polycarboxylate ethers (PCE), the polymers exhibit a comb structure with polyether side chains; however the acrylate-free backbone has an aromatic rather than aliphatic character and is considered to be more rigid. Moreover, they feature phosphate moieties as anchoring groups. The presence of aromatic units in combination with a very high density of negative charges is believed to increase the affinity to the cement surface, whereas the presence of side chains adds a steric component to the electrostatic component of the inter-particle repulsion forces. These aromatic comb polymers will be called polyaryl ethers (PAE) in order to distinguish them from conventional PCE. They combine some of the beneficial properties of poly(ß-naphthalene sulfonates) (BNS) with the high performance of PCE (Figure 1). The aromatic character of the backbone is believed to induce a much better carbon black dispersion capability, which improves the surface appearance of e.g. fly ash containing concretes.

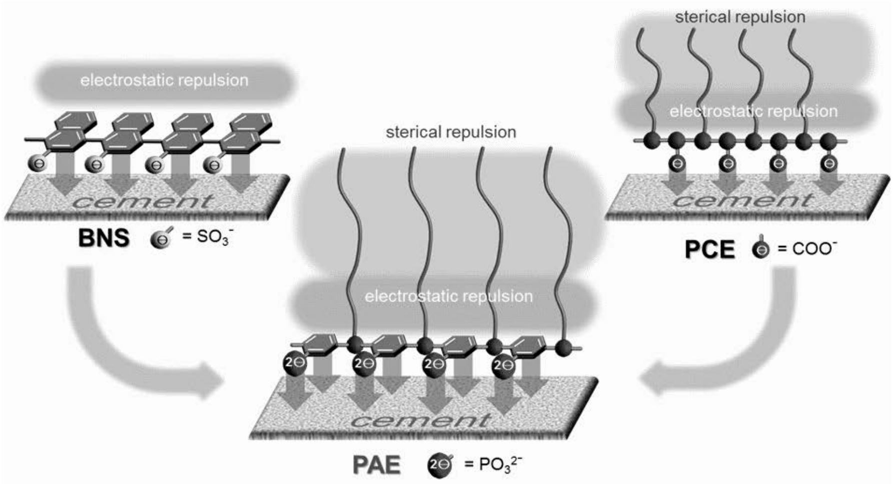


Figure 1: Schematic sketch of the structure of aromatic comb polymers (PAE)

The influence of PAE on the rheology of fresh concrete and mortar is compared with commercially available conventional PCE as well as with laboratory-made small molecule dispersant (SMD). The SMD-type dispersant consists of a single polyether chain that is comparable to those used as side chains in PCE. However, this single polyether chain is not attached to a polymer backbone. Instead, it is functionalized with negatively charged functional groups that serve as anchoring group and thus provide adsorption to concrete (Figure 2)


Figure 2: Schematic sketch of the structure of a small molecule dispersant SMD

2. THEORETICAL BACKGROUND

A Bingham type relation is often used to describe the flow behavior of concrete at low and intermediate shear rates. It describes a linear dependency of shear stress τ in relation to the applied shear rate $\dot{\gamma}$:

$$\tau = \tau_0 + \mu \cdot \dot{\gamma}$$

In this model, the yield stress τ_0 determines the value when concrete begins to flow under its own mass. This value can be easily measured by the flow test [3]. The flow of concrete or mortar correlates with yield stress τ_0. On the other hand, the plastic viscosity μ determines the flow time or speed of concrete during molding or pumping. This value indicates how easily the concrete can be placed or filled into forms (knead ability, place ability, spread ability, spray ability, pump ability or flow ability). Furthermore, the flow properties of concrete are also determined by its thixotropy, a shear thinning effect. Due to its thixotropic nature, concrete appears thick (viscous) under static conditions and will start to flow (become thin, less viscous) over time when subjected to mechanical stress (e.g. agitation, shaking, stirring). Upon relieve of the mechanical stress, it takes a certain time for the concrete to return into its previous, more viscous state. Thus, mechanical stress helps to improve the flow ability of fresh concrete.

The flow behavior of concrete is influenced by the water content, type, shape, size and amounts of the solids, viscosity modifying agents (VMA), air entraining agents (AEA) and superplasticizers. *Wallevik et al.* have nicely summarized these influencing factors in form of a rheograph [4]. According to them, dispersants improve the flow ability of concrete mainly by lowering its yield stress. Most superplasticizers have only a minor impact on the plastic viscosity. For some dispersant types a rise of the plastic viscosity can be observed and for others a decrease.

The main application of dispersants, however, is not to improve the flow ability of concrete at a given W/C. More typically, they are used to reduce the water content with the target to achieve a higher compressive strength. The dispersant takes care of maintaining the same flow ability of the concrete by lowering the yield stress compared to the same concrete mix without dispersant.

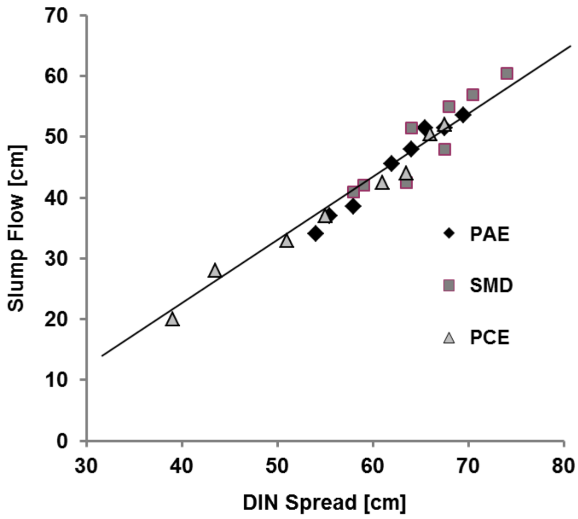


Figure 3: Correlation slump flow and spread (according to DIN EN 12350-5)

The most widely used industrial application test method to characterize the fluidity of concrete is the slump test. The obtained values are based on the flow of concrete under its own weight. However, the concrete is subjected to only minor shear forces during the test. Even the DIN Spread test, a variation of the slump test in which the flow diameter is determined on a shocking table, does not involve substantial shear forces. Thus, the obtained values for slump (remaining height of the concrete pile compared to the test cone), slump flow (flow diameter without shocking) and DIN spread (flow diameter on a shocking table) are in principle equivalent. If slump values are plotted against slump flow or DIN-spread values, the data show a very good correlation and always fit to the same curve - independent of the concrete mixture, the type of superplasticizer and the age of the fresh concrete. All these values correlate very well with the yield stress. However, plastic viscosity, the other characteristic indicator for the rheology, cannot be assessed by using these methods.

The effect of increasing plastic viscosity becomes obvious in concrete mixture proportions that exhibit a low water to cement ratio and increasing solid volume fractions. These concretes have a sticky appearance to the workers despite exhibiting a normal behavior in slump tests [5]. The increase of the plastic viscosity is related to contact interactions between the fine particles.

They are a result of repulsive electrostatic forces, caused by the electric double layers on the surfaces, attractive van der Waals forces and by the steric repulsion caused by the layer of adsorbed superplasticizer. The latter is the dominating force between the fine particles. Due to the steric repulsion caused by the presence of a superplasticizer, agglomerates are getting fractured into smaller particles, which can now be more densely packed compared to the larger agglomerates. This means that the maximum possible solid volume fraction increases. Water that had been stored in interparticle pores becomes displaced and forms a water film around the particles and helps to separate the contact points between the particles. Thus, the mobility of the particles increases and the system becomes more fluid.

The apparent viscosity of a mortar very much depends on the shear rate that is applied. At low shear rates the apparent viscosity is dominated by colloidal interactions and very sensitive to the superplasticizer dosage, because it decreases the stability of the colloidal network. With increasing shear rates (in the range of 3 - 30 s^{-1}), the stability of the colloidal network loses significance and viscous interactions start to take over. Finally, at very high shear rates, the liquid interparticle phase loses influence and the system starts to behave like densely packed solid spheres and the apparent viscosity rises again. The best-suited shear rate window for measuring the apparent viscosity is therefore the region around the minimum of the apparent viscosity curves. Artelt et al. have correlated the range of shear forces that typically impact the fresh concrete when applying typical industrial test methods or processing steps [6]. According to them, segregation occurs at virtually non-existent shear rates. At shear rates below around 0.1 s^{-1}, concrete stops to flow visibly. For practically relevant concrete processing steps like placement, finishing and handling, the applied shear forces are estimated to be in the range of 10 - 30 s^{-1}, which is similar to the shear forces in V-funnel or L-box tests. However, concrete also exhibits a pronounced thixotropic behavior. External shear forces degrade the colloidal network formed in fresh concrete, resulting in a decrease of the viscosity. After a certain time, the network has degraded completely and the viscosity reaches its minimum. However, this thixotropic behavior is reversible. As soon as the concrete is put to a rest, the intercolloidal network starts to reestablish itself. Theoretically, the viscosity should rise until its initial value has been reached. In practice, the final viscosity will be higher than its initial value, due to the ongoing hydrates nucleation [7]. In addition these hydration products increase the surface roughness of the particles, which leads to stronger contact interactions. Consequently, it is only possible to compare values for the plastic viscosities of different concretes if they all have been determined at the same stage of the thixotropic effect, ideally in a stage were all colloidal networks are disrupted.

In a previous work [8] it was reported that PAE and SMD are able to considerably decelerate the colloidal network reformation when compared to PCE. This was proven by applying an experimental setup, in which the colloidal network formation was subsequently completely destroyed by applying high shear stress followed by a period of rest. After destroying the colloidal network, the sample was always treated with a very low shear rate of 0.5 s^{-1} in order to minimize the plastic viscosity contribution. The shear stress was therefore a more or less pure static yield stress response. The thixotropy can be determined as the first derivative of the yield stress as a function of time. As can be seen in figure 4, the yield stress derived from the shear stress recordings increases over time. However, the slope decreases when comparing the first period of rest (0-30 s) with the second period of rest (30 - 120 s). Moreover, it becomes obvious that PAE exhibits the lowest slope and therefore the lowest thixotropy (Figure 4).

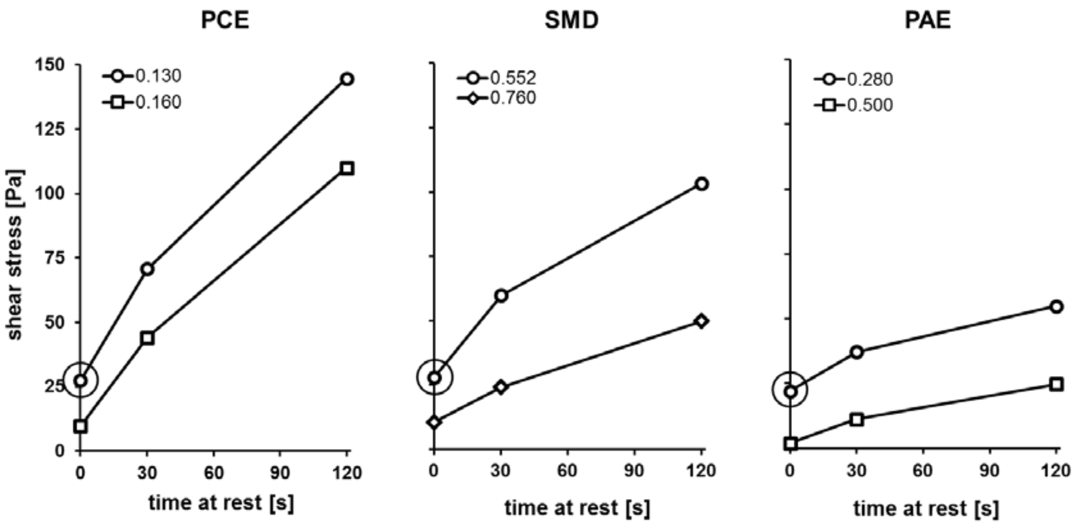


Figure 4: Correlation between shear stress and time at rest for different superplasticizer types, applied at two different dosages

In other words, stiffening of the mortar (increase of the yield stress) over time is much less pronounced if PAE and SMD are used as plasticizers. Both SMD and PAE are able to significantly reduce the thixotropy of mortar and concrete compared to PCE. However, in terms of practical use PAE offers two important benefits: PAE is significantly more dose efficient and, even more important, it is much less retarding compared to SMD. PAE delivers superior early compressive strength: While the 24 h strength could not be tested in case of the SMD-containing concrete (mixture 4), the PAE-containing concrete (mixture 5) already exhibited a compressive strength of 9.9 MPa.

In this work we investigate if the results found for mortar are also valid for concrete. We compare the rheology of concretes prepared with PAE and SMD with PCE by utilizing V-funnel or L-box tests, which give a rather rough qualitative assessment of the plastic viscosity, as well as by employing a concrete rheometer and tribometer. The latter are much better suited for the measurement of the plastic viscosity of concrete, as they allow applying defined shear forces and viscosity measurements without significant time lapse.

3. RESEARCH SIGNIFICANCE

The introduction polycarboxylate ethers (PCE) in the mid 1980ies, was the last major technological breakthrough in the field of admixtures and established new “superplasticizer” performance levels for water reduction and workability retention. About 30 years later BASF introduces an entirely new superplasticizer technology: Phosphate-functionalized Polyaryl ethers (PAE) offer state of the art water reduction and outperform most PCE in reducing the stickiness and tenacity of modern SCM based concretes. This work aims toward comparing the influence PAE, SMD and PCE superplasticizers on the rheology of concrete by using industrial V-funnel and L-box as well as a concrete rheometer and a tribometer.

4. EXPERIMENTAL INVESTIGATION

4.1 Materials and mixture proportions

All concrete mixture proportions used in this work are summarized in table 1. Ordinary Portland cements were used for the concretes. Aggregate grading was in between grading curve A and B according to DIN EN 1045-2. The maximum grain size D_{max} was 16 mm.

The superplasticizers PCE and PAE were characterized by having relatively short side chains. However, the side charge density was lower in the case of PAE. For comparison a small molecule dispersant (SMD) was also tested. It consisted of a long polyether chain with one chain end being functionalized with four anionic moieties. The admixtures dosages were adjusted to achieve a comparable initial slump flow. All dosages are based on solid content of admixture by weight of cement. The air content has been kept constant in a range of 2 to 2.5%.

Concrete slump flow tests were carried out according to DIN EN 12350-5 and V-Funnel tests according to the EFNARC_SCC Guidelines, May 2005 47-56 [9].

Table 1: Overview of the concrete mixture proportions used in this work:

Mix No.	Cement (Type)	Cement (kg/m^3)	Fly ash[1] (kg/m^3)	Slag[2] (kg/m^3)	Limest. Powder[3] (kg/m^3)	Sand/ Aggregates	Water (kg/m^3)	W/C	Admixture (type)	Admixture dosage (%)[4]
1	CEM I 52.5 N	400	-	-	-	nat.rounded	152	0.38	PAE	0.68
2	CEM I 52.5 N	400	-	-	-	nat.rounded	152	0.38	SMD	0.99
3	CEM I 52.5 N	400	-	-	-	nat.rounded	152	0.38	PCE	0.48
4	CEM I 42.5 R	300	75	120	40	crushed	186	0.62	PAE	0.84
5	CEM I 42.5 R	300	75	120	40	crushed	186	0.62	SMD	1.1

[1]Siliceous Fly ash: Blaine 4050 cm^2/g, density 2.27 kg/l; [2]Slag: Blaine = 4250 cm^2/g, density = 2.91 kg/l; [3]Limestone powder: Blaine = 5750 cm^2/g, density = 2.71 kg/l. [4]Admixture dosages are given in % solid content by weight of cement.

For the rheological measurements a simple mortar consisting of 50 wt.% CEM I 52.5 N (see table 1) and 50 wt.% fine quartz sand with a maximum particle size of 0.5 mm was used. The W/C was 0.35.

4.2 Experimental results

Starting point of this work was the observation that concrete prepared with PAE and SMD appeared to be much less sticky and tenacious compared to concrete prepared with a conventional PCE, despite showing the same flow behavior in conventional slump tests. Figure 5 shows a typical concrete test result. They turned out to be quite different for the three superplasticizer types. PCE had the lowest dosage requirement of 0.48% by weight of cement; PAE required 0.68% and the SMD had the highest dosage 0.99%. For the first 10 min, all three admixtures were able to retain the workability at the initial level. After 10 min, the concrete made with PCE started to lose its flow ability, whereas PAE and SMD did not show a drop of workability.

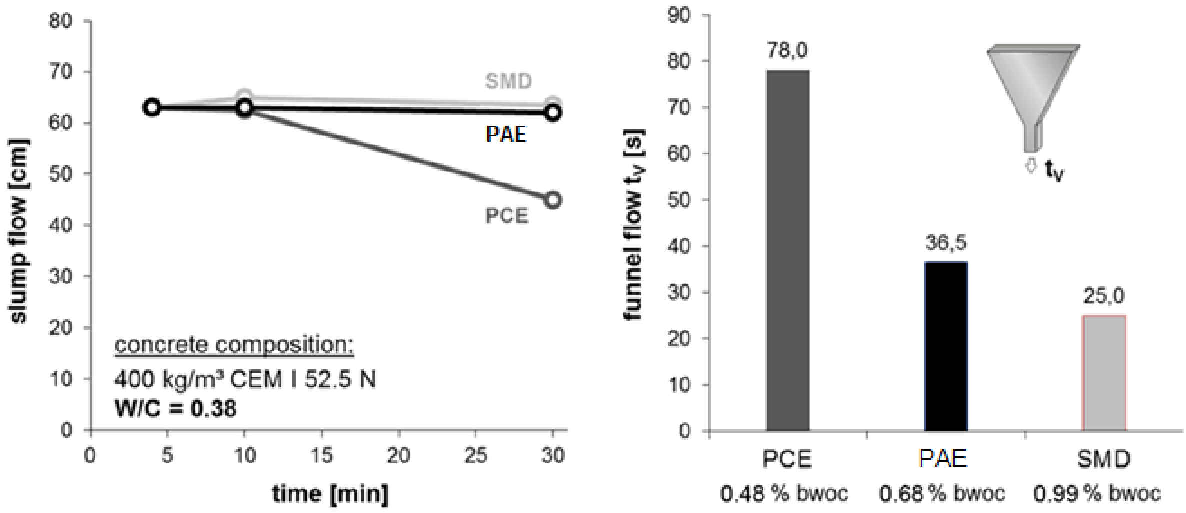


Figure 5: Slump flow behavior and V-funnel times for concrete prepared with PCE, PAE and SMD (concrete mixtures 1, 2, 3)

The lower dose efficiency of PAE compared to PCE can most likely be attributed to its lower charge density, which leads to a slower adsorption. Thus, PAE is able to cover freshly formed surfaces as they appear in the course of the hydration process. In the case of SMD, the adsorption is not sterically hindered since the anionic part is freely accessible, yet the affinity of the SMD to the cement surface is lower compared to PCE and PAE. We believe this can be explained by the fact that PCE and PAE have many more anchoring units per molecule compared to SMD, which leads to a synergistic effect on the strength of adsorption. As long as a polymer is adsorbed by a part of its backbone, it is unable to move away from the surface. The probability of losing the complete surface contact is therefore much lower for a polyelectrolyte (e.g. PCE, PAE) compared to a SMD.

Consequently, SMD require a higher dosage in order to shift the equilibrium between adsorbed and desorbed SMD towards a higher degree of adsorption. As with PAE, the initial surplus of desorbed SMD helps to maintain the flow ability over time.

A first series of experiments was carried out in mortar. The mortar was mixed in an ordinary laboratory mixer. The dry constituents were homogenized for 60 s after which the water and superplasticizer were added and mixed for 90 s. Following a break of 90 s, the mortar was mixed for 60 s. The slump flow was determined immediately after mixing, followed by the rheological measurement. A rotational rheometer Rheotest RN 4 with vane geometry was used to evaluate the rheological properties. A shear rate profile with linearly increasing shear rate up to 100 s^{-1} over a time of 200 s and afterwards linearly decreasing shear rate over another 200 s was applied.

The yield stress was calculated trough the Bingham model using a typical shear stress versus shear rate plot (Figure 6). The apparent viscosity was calculated at a selected shear rate of 10 s^{-1} (Figure 4). Upon increase of the polymer dosage yield stress and apparent viscosity decreases.

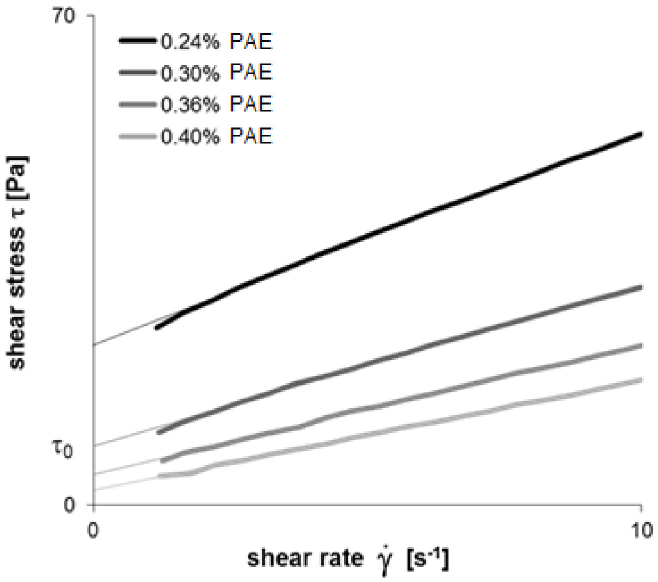


Figure 6: Shear stress of a mortar with different dosages of **PAE** depending on the shear rate.

Like with the concrete tests, the dosages of the admixtures were adjusted to achieve a comparable initial slump flow with the flow-tube (d/h = 3/5 cm) of 13 ± 0.5 cm immediately after mixing. As shown in Figure 7, the mortar prepared with PCE exhibited a higher apparent viscosity than the ones prepared with PAE and SMD. However, it must be noted that there is a time gap of approximately 400 s between slump flow and apparent viscosity measurements.

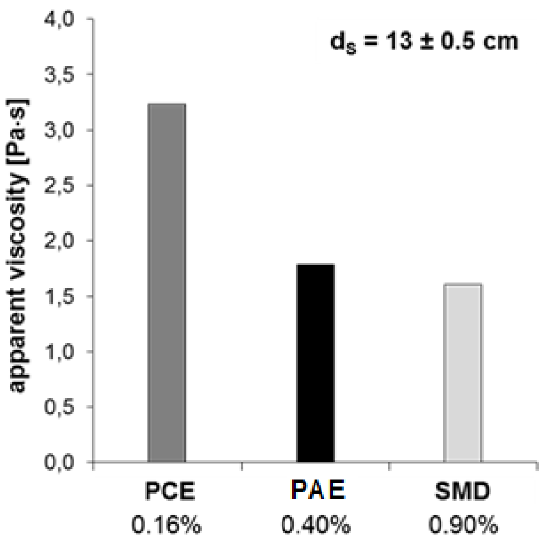


Figure 7: Apparent viscosity at constant slump flow

In order to eliminate the above mentioned time gap, the mortars were compared at the same yield stress (Figure 8, right). Surprisingly, in this experiment all three mortars exhibited virtually the same apparent viscosity at the selected shear rate of 10 s^{-1} (Figure 8, left).

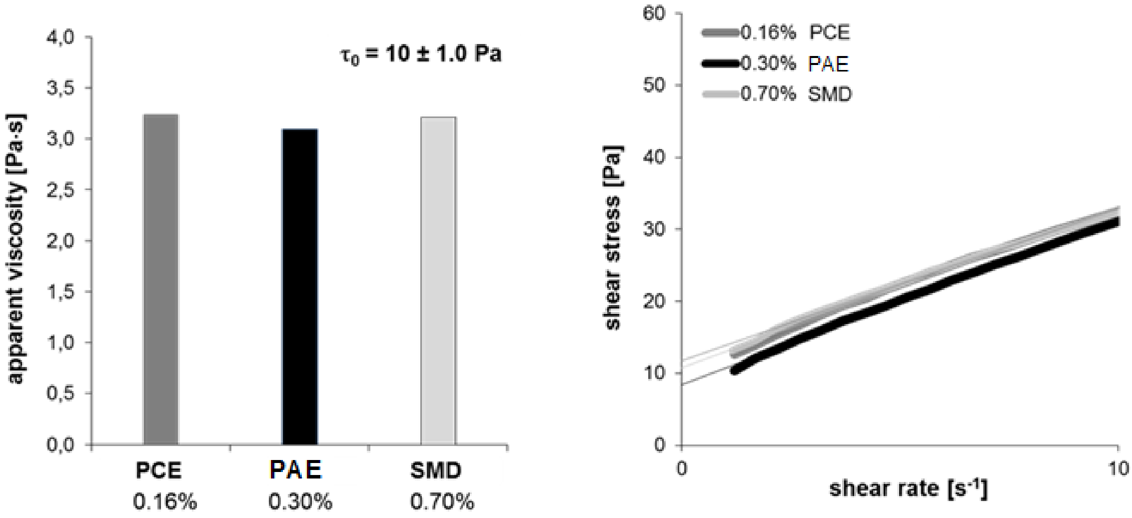


Figure 8: Apparent viscosity at constant yield stress

Now, the question had to be answered why PCE should develop a higher yield stress in comparison to a PAE or SMD within 400 s after showing a constant slump flow. This behavior can be related to two effects that are a consequence of the inevitable time lapse between mixing and testing. In this time gap, only low shear forces occur. The first effect is the partial reestablishment of the colloidal networks that had been deteriorated by mixing (thixotropy of cement paste [2]). A second effect is the consumption of water due to hydration reactions. Both effects lead to a higher yield stress and apparent viscosity.

For proving the first hypothesis of thixotropy and observing the formation of a network in time at rest, a second shear rate program was used (Figure 9).

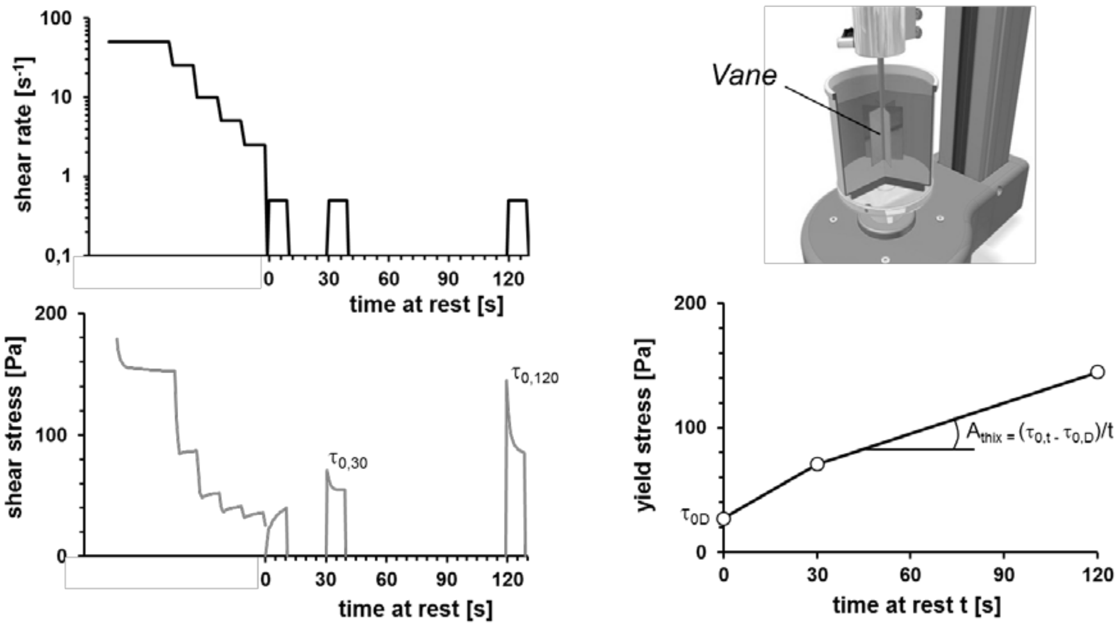


Figure 9: Shear rate profile for detection of shear stress depending on time at rest.

In the first step, the network formation was completely destroyed by applying high shear stress. Then, the sample was treated with a very low shear rate of 0.5 s^{-1} in order to determine the shear stress at the starting point (t = 0 s). Owing to the low shear rate, the measured shear stress is mainly due to the static yield stress of the mortar. The sample was then left at rest for 30 s and the shear stress was again recorded for 10 s. After that, the sample was again allowed to rest until 120 s and again the shear stress was recorded. The first derivative of the yield stress as a function of time is a measure of the thixotropy A_{thix}.

As can be seen in the lower right plot of figure 10, the yield stress derived from the shear stress recordings increases over time and exhibits a change in slope from 0 to 30 s compared to 30 - 120 s, due to above mentioned effects. Above described test regime allows now to distinguish the behavior of the three different superplasticizer types (Figure 10).

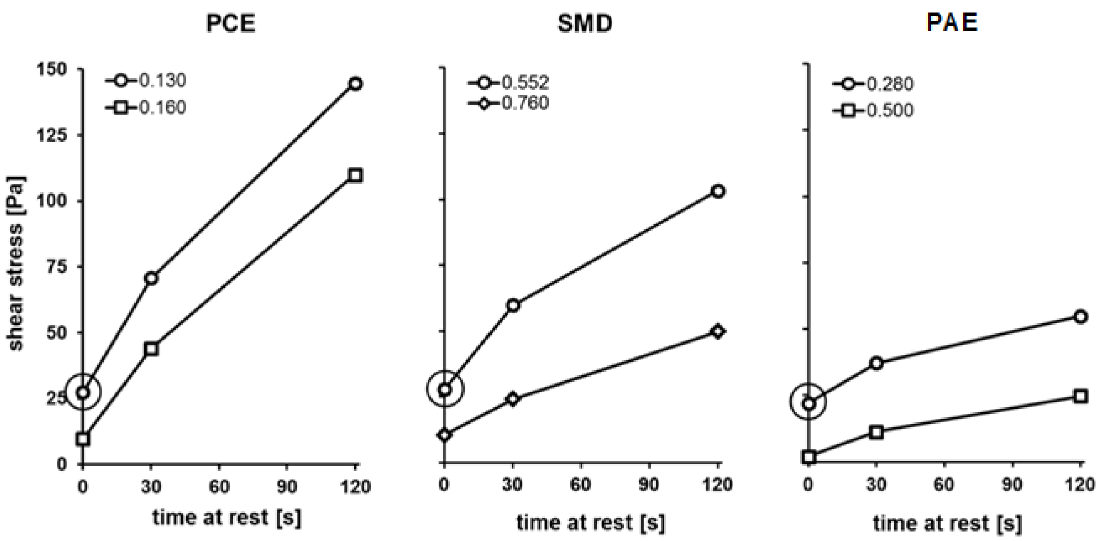


Figure 10: Correlation between shear stress and time at rest for different superplasticizer types

Starting at a shear stress of 25 Pa for all mortar slurries (encircled reading point), the gradient of the slope can now be compared for the three different superplasticizer types. It can be clearly seen that the slope of the curves for different PCE dosages are significantly steeper compared to the curves for SMD and PAE. In other words, stiffening of the mortar (increase of the yield stress) over time is much less pronounced if PAE and SMD are used as plasticizers. Both SMD and PAE are able to significantly reduce the thixotropy of mortar and concrete. However, in terms of practical use PAE offers two important benefits: PAE is significantly more dose efficient and, even more important, it is much less retarding compared to SMD. PAE thus delivers superior early compressive strength: While the 24 h strength was not testable in case of the SMD-containing concrete (mixture 4), the PAE-containing concrete (mixture 5) already exhibited a compressive strength of 9.9 MPa.

5. CONCLUSIONS

PAE, a novel class of superplasticizers, has been developed by BASF. PAE features a comb polymer structure like PCE, but exhibits an aromatic backbone. PAEs can be tailored to different customer needs. They can either act as water reducer or workability retainer, depending on what side chain length and charge density have been realized in the comb polymer structure. What really differentiates PAE from conventional PCE is the capability of PAE to significantly reduce the “stickiness” of concrete. Especially concretes with high SCM content, used to improve the carbon footprint of concrete, often appear sticky and tenacious. In this work we elaborated that the loss off stickiness in case of use of PAE can be traced back to a reduction of the plastic viscosity. This is remarkable, as conventional superplasticizers are only able to reduce the yield stress of concrete. Plastic viscosity of concrete is not properly detectable by using slump flow tests. A full differentiation of yield stress and plastic viscosity is only possible by means of a concrete rheometer.

However, qualitative measurement of rheological properties by tests like by the V-funnel, L-Box and T50 tests clealy support relevance of the rheological measurement for practical applications: Concretes prepared with PAE are easier to place, easier to pump and provide easier surface finishing. For ready mix concrete application these properties can be maintained over extendet times: Workability retention AND rheology retention are achieved in combination with excellent early strength development.

REFERENCES

[1] Hargreaves, D.: “International Cement Review”, The Global Cement Report 10th Edition (2013) 8

[2] Mahasenan, N.; Smith, S.; Humphreys, K.; Kaya, Y.: “The Cement Industry and Global Climate Change: Current and Potential Future Cement Industry CO_2 Emissions”, Greenhouse Gas Control Technologies – 6th International Conference; Oxford, Pergamon (2003) 995-1000

[3] Roussel, N.; Stefani, C.; Leroy, R.: “From mini-cone test to Abrams cone test: measurement of cement-based materials yield stress using slump tests”, Cement and Concrete Research, 35, 5, 2005, 817-822

[4] Wallevik, O.H., Wallevik, J.E.: “Rheology as a tool in concrete science: The use of rheographs and workability boxes”, Cement and Concrete Research, 41 (12) (2011) 1279-1288

[5] Hot, J.; Roussel, N.: “Adsorbing polymers and macroscopic viscosity of concentrated cement pastes”, Proceedings of the 7th RILEM International Conference of Self-Compacting Concrete and of the 1st RILEM International Conference on Rheology and Processing of Construction Materials, 2013, p. 197-204

[6] Artelt, C., Garcia, E.: “Impact of superplasticizer concentration and of ultrafine particles on the rheological behavior of dense mortar suspensions”, Cement and Concrete Research, 38 (2008) 633-642

[7] Roussel, N.; Ovarlez, G.; S Garrault, Brumaud, C.: “The origins of thixotropy of fresh cement pastes”, Cement and Concrete Research, 42 (1) (2012) 148-157

[8] Kraus, A.; Mazanec, O; Dengler, J.; Hillesheim, N.; Bokern, J. “Influence of PAE, SMD and PCE superplasticizers on the rheological properties of mortars and concrete” to be published in RILEM Proceedings (PRO95) International RILEM conference “ Application of superabsorbent polymers and other new admixtures in concrete construction”, 14-17 September 2014, Dresden, Germany

[9] The European Guidelines for Self-Compacting Concrete, 2005, 47-56; http://www.efnarc.org

SYNERGY OF CEMENT TYPE CEM I 42.5 R AND PLASTICIZING ADMIXTURES

Murat Kabadayı (1), Seda Eren (1), Y. Oğuz Aydinç (1), Burhan Manzak (2) and Erbil Öztekin (3)

(1) Traçim Çimento A.Ş, Turkey

(2) Traçim Beton A.Ş, Turkey

(3) Prof. Dr.

Abstract

Cement is the raw material of concrete as a binding agent. Beside this, plasticizing admixtures (water reducer) is used to improve attributions and performance of the concrete. In this context, the interaction between the cement and plasticizing admixture is crucial for the technical and economic performance of the concrete.

This research was done by Traçim Çimento A.Ş to meet the customer satisfaction and to give them better quality service. Traçim Çimento A.Ş investigated the interaction of different raw materials of concrete admixtures with mostly sold cement type CEM I 42.5 R. One reference sample concrete without any admixture was prepared in order see the synergy of cement type CEM I 42.5 R with traditional concrete admixtures which has raw materials like lignosulphonate; gluconate; molasses; naphthalene sulphonate, melamin sulphonate.

Key Words: Plasticizing, concrete, admixture, compressive strength, slump loss, water reducer

1. INTRODUCTION

Binding raw material of concrete is cement that reacts with water; it has a rigid structure and strength day by day. The basic factor affecting compressive strength of concrete is water/cement ratio. When this ratio decreases, the space of the concrete contexture decreases and strength increases. Water demand for the workable fresh concrete can be reduced in the case of usage plasticizing admixtures.

According to the standard [1], there are two group of admixtures that can reduce water demand according to the reference concrete which has same slump and no admixture: one is; “water reducer/plasticizing admixtures” (5% reduce) and the other one is; “high range water reducers/ superplasticizers” (12% reduce).

In the current market, 5 groups of plasticizer admixtures can be seen. Normal plasticizers which have raw materials like molasses, vinas, lignosulphonate and gluconate; superplasticizers which have raw material like naphthalene sulphonate, melamin sulphonate; mid-range plasticizer admixtures that have mix raw materials (performance of them are between normal and superplasticizers); mini superplasticizers; polycarboxylate based hyperplasticizers or new gen superplasticizers.

Traçim Çimento A.Ş is located in Trakya region and it produces mainly CEM I 42.5 R type cement which is highly used in this region. The most common produced concrete types are C20/25 – C35/45 in this region. In these concretes, traditional plasticizers admixtures, that have classic raw material, are used instead of the polycarboxylate based new gen superplasticizers.

2. COMPARATIVE BEHAVIOR OF ADMIXTURES’ RAW MATERIALS

In an internal report Chryso France [2], performance level of principal raw materials of plasticizers (molasses, vinas, gluconate, lignosulphonate, naphthalensulphonate, melaminsulphonate and polycarboxilate-PCP) was plotted (Figure1) to see the effect of increase in 28 day compressive strength versus plasticizer effect. Dose and application ratio is 0.3% for the molasses, vinas, gluconate and lignosulphonate; for the naphthalenesulphonate, melaminsulphonate and polycarboxilate is 0.5-0.7% (in dry basis). Figure 1 is a simple summary of common facts: in the usage of low dose molasses, vinas, gluconate and lignosulphonate, that are traditional raw materials of normal plasticizers admixtures increase plasticity and strength in some period; when the requirement is more slump and strength, high dose naphthalene and melaminsulphonate, for high performance, polycarboxylates should be used.

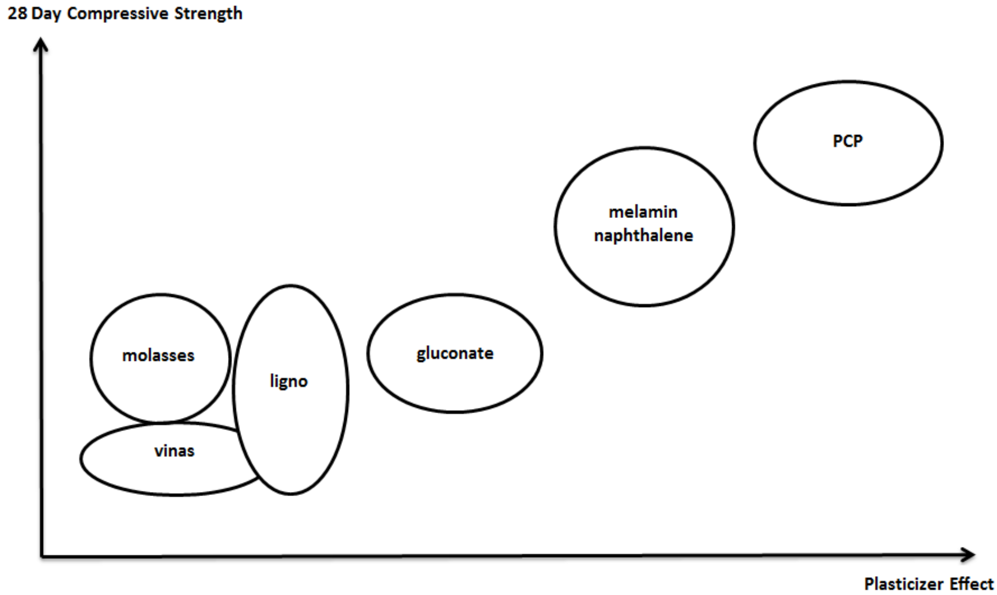


Figure 1: Raw material effects on 28 day compressive strength [2].

In Figure 2 the representation of the side effects of raw materials can be seen: set retardation and air entrainment. As it seen from the Figure 2 melaminsulphonate placed at the bottom left as it has the minimum effect of air entrainment and set retardation. After that, set retardation effect increases for naphthalenesulfonate, lignosulphonate, vinas, molasses and gluconate. Naphthalenesulphonate has max. air entrainment effect and this decreases for vinas, molasses and gluconate.

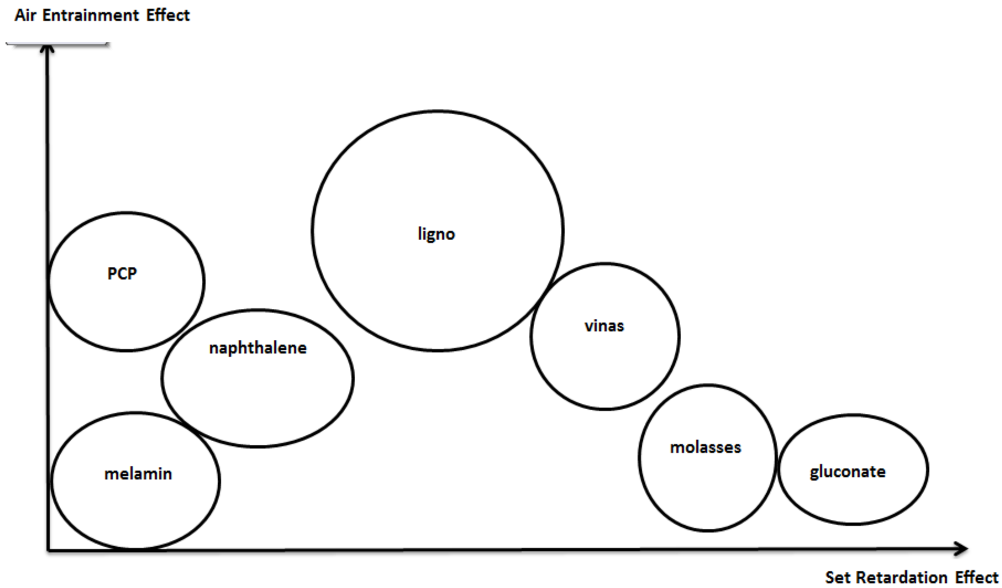


Figure 2. Raw material effects on set time [2].

3. EXPERIMENTAL STUDY

Experimental study was carried out in Traçim Çimento A.Ş Concete Laboratory, concretes in the experiments consist of 300 kg/m^3 CEM I 42.5R type cement, 185 kg/m^3 water, 660 kg/m^3 (12-22) mm crushed stone, 330 kg/m^3 (5-12) mm crushed stone, 700 kg/m^3 (0-5) mm crushed sand and 200 kg/m^3 (0-2) mm natural sand. All experiments were done at 20±^{o}C. Crushed stone and sand is resourced from dolomite based limestone. The chemical and physical properties of cement were given in Table 1.

Table 1: Chemical and physical properties of cement type CEM I 42.5 R in Traçim Çimento.

SiO_2 (%)	**Al_2O_3 (%)**	**Fe_2O_3 (%)**	**CaO (%)**	**MgO (%)**	**Na_2O (%)**	**K_2O (%)**	**SO_3 (%)**	**I.Res (%)**
18,1	4,55	3,51	64,63	0,92	0,25	0,78	3,19	0,23
L.O.I (%)	**F.CaO (%)**	**C_3S (%)**	**C_2S (%)**	**C_3A (%)**	**C_4AF (%)**	**Exp. (mm)**	**Water Demand (%)**	**Sp. Weight (gr/cm^3)**
2,76	2,06	60,70	15,77	7,43	10,86	1,0	28,65	3,10
32 (µ)	**63 (µ)**	**Sp.Surf. Area (cm^2/gr)**	**I. Set (min)**	**F. Set (min)**	**1 day (N/mm^2)**	**2 day (N/mm^2)**	**7 day (N/mm^2)**	**28 day (N/mm^2)**
9,6	1,0	3843,5	160	240	18,3	29,4	46,9	58,1

40 % solid based raw materials were used as a plasticizer admixture. . Admixture / cement ratio 0 % (reference sample); 0,6 % and 1 % with lignosulphonate, gluconate and molasses; 1% and 1,8% for naphthalenesulphonate and melaminsulphonate. All concretes were produced with same slump 140±10mm and then water reduce, slump loss after 60 minutes, 1 - 7 and 28 day compressive strength were determined.

Raw materials of admixtures are molasses, calcium based lignosulphonate, gluconate, naphthalenesulphonate and melaminsulphonate. The results of the experiments were presented in Table 2.

4. RESULTS AND EVALUATION

Table 2 shows us, strength development rate of cement was high as reference sample (without admixture) 1 day/28 day compressive strength ratio was 0.33; 7 day/28 day compressive strength ratio was 0.84.

Table 2: Results of experiments.

Admixture	Adding Ratio (%)	Water Reduce (%)	Slump (mm)	Slump after 60' (mm)	1 day Comp.St. (Mpa)	7 day Comp.St. (Mpa)	28 day Comp.St. (Mpa)
Reference Sample	0	0	140	115	11,2	28,3	33,7
Molasses	0,60	3,6	140	110	7,4	36,4	40,4
Gluconate		9,3	145	115	9,1	39,7	42,7
Lignosulp.		7,2	140	120	9,8	34,8	40,3
Molasses	1,00	8,1	145	115	0	42,1	46,3
Gluconate		13,5	150	135	2,5	44,5	49,5
Lignosulp.		13,5	140	105	11,8	39	45,6
Nap.sulp.		13,2	140	110	14,7	35,3	40,2
Mel.sulp.		11,5	140	120	13,2	37,6	40,9
Nap.sulp.	1,80	20,3	140	115	17,2	39,8	46,1
Mel.sulp.		15,2	140	110	13,6	38,3	43,8

Except 1 day compressive strength, gluconate admixture raw material had the highest performance. For higher water reduce ratio than 13.5 % naphthalenesulfonate or melaminsulphonate should be used in some high ratios. This admixture also had the least slump loss after 1 hour (15mm). 28 day compressive strength has the highest value in gluconate admixture that means 47% increase from reference sample. This performance is better than the 1.8% ratio usage of melaminsulphonates and naphthalenesulphonates. But, in the lack of early day strength, application of gluconate admixture alone and in this ratio becomes crucial. When ratio decrease to 0.6%, 1 day compressive strength increased but 7 and 28 day strength decreased.

Performance of molasses except early strength is significant, especially from economic point of view. Although, unstable quality, Cl content and supply problem of raw material in every season, makes this raw material insecure and unsteady.

Lignosulphonate has the second good performance after gluconate. Even 28 day compressive strength 3-4 MPa lower, it has nearly same 1 day compressive strength with the reference sample. In building site applications it has good performance for setting time. Beside these, it has price advantage comparing to the other raw materials.

Naphthalenesulphonate and melaminsulphonate has surprising bad performance with this cement except early strength.

In this context, it seems that the most suitable admixture raw material is lignosulphonate as it has not set or mold problem and also it has not technical and economical difficulty. For the 28 day strength addition of gluconate and for the early strength addition of naphthalensulphonate seems possible. In Table 3 results were shown of the experiments made for this concept.

Table 3: Additional experiments made with mix raw materials.

Admixture	Reference Samp.	Gluco	Ligno.	Lignosulp. %90 Gluconate %10	Lignosulp. %70 Gluconate %30	Lignosulp. %60 Gluconate %20 Naphth. %20	Lignosulp. %90 Gluconate %10
Adding Ratio (%)	0	1,0	1,0	1,0	1,0	1,0	1,4
Water Reduce (%)	0	13,5	13,5	16,1	17,9	16,1	16,7
Slump (mm)	140	150	140	150	150	140	140
Slump After 60' (mm)	115	135	105	130	135	125	115
1 day Comp.St. (Mpa)	11,2	2,5	11,8	10,2	6,8	8,7	8,5
7 day Comp.St. (Mpa)	28,3	44,5	39,0	40,6	41,5	40,1	44,9
28 day Comp.St. (Mpa)	33,7	49,5	45,6	46,7	48,9	47,2	51,8

Addition of 10% gluconate to lignosulphonate make good effect in water reduces, slump and 28 day compressive strength, but it has no big effect in 1 day compressive strengths. Results were better in 70% lignosulphonate + 30 % gluconate than 1st trial except 1 day compressive strength. But this result is not really bad for building site applications.

With the increase in cost for 20% naphthalenesulphonate addition, a small decrease occurred in 28 day strengths and 1 day strengths are better than before.

90% lignosulphonate + 10% gluconate admixture with ratio of 1.4% makes good effect in 1 day and 28 day compressive strengths. With the application of 300 kg/m^3 cement and %1,2 mid-range admixture seems producible S3 C35/45 type concrete.

5. CONCLUSIONS

The synergy of admixtures for a specific cement has noteworthy deviation such as for the application with same ratio, 28 day compressive strengths changes 10 MPa (~20%).Technical and economic changes in raw materials of admixtures thereby admixtures effects highly the performance of concrete in economical and quality point of view. Therefore, it will be a mutual benefit for the cement producers and their customers if cement producers determine the suitable admixture for their cement. For the cement that is used in this study, 10-20% gluconate + 80-90% lignosulphonate based admixtures seems appropriate in terms of technical and economical point in production of C20/25 - C35/45.

REFERENCES

[1] TS EN 934-2/March 2002 “Admixtures for concrete, mortar and grout - Part 2: Concrete admixtures; Definitions, requirements, conformity, marking and labelling “Turkish Standards Institution, Bakanlıklar, Ankara.

[2] Micollier, S.”Raw Matterials”, *internal report*, Chyrso France, May 2008.

THE EFFECTS OF CEMENT PRODUCED WITH PAAF COMPOUNDS TO PHYSICAL AND MECHANICAL PROPERTIES OF CONCRETE

Hakan Gülseren (1) and Uğur Ersen Şenbil (1)

(1)Idea Construction Chemicals Co.,Turkey

Abstract

Physical and mechanical properties of concrete are affected directly by physical and chemical properties of cement which is used for concrete production. In the recent times, using new generation grinding aids (PAAF Compound) provides saving electric energy in the level of 30-40 %. Also it brought with many advantages to concrete technology such as cement's strength is developed by modification of hydration kinetics and cement's particle size distribution (PSD), water demand is decreased and released hydration heat is remained in optimum level in the first period.

It is observed that concretes which are obtained by cements produced with PAAF compound provide higher consistency in constant water/cement ratio, have higher preservation properties of consistency because of lower released hydration heat in first 10 hours period, have more stable structure based on decreasing concrete's porosity by courtesy of optimal cement particle size distribution (PSD) and developing strength are affected positively especially at older ages due to changing in hydration products.

Keywords: PAAF, cement, concrete, PSD

1. INTRODUCTION

Beside improving quality of raw material and clinker and using chemical additive, producing cement more fine and improve its blaine value are the ways which are chosen by producers for high-strength cement.

In concrete production process create some chemical and physical problems such as increasing the amount water for wetting whole cement surface may decrease strength of concrete by increasing water/cement ratio. Decreased strength may be increased by using high performance additives and this causes incremental costs.

High-strength cement is obtained with PAAF compounds which are developed by Idea Construction Chemicals Co. without fine grinding as conventional additives. These cements were used in producing of concretes and concretes are obtained with equal water/cement ratio, more fine particles and more workable. In other words, cements which decrease water demand of concrete are obtained. Besides that, more productive hydration process, lower porosity and more C-S-H gels in inner structure images were formed in compares with conventional concretes (Senbil et. al., 2014).

Hydration heats at the end of 10 h and 24 h of cements which were produced with PAAF compound and conventional additive are shown in Table 1:

Table 1: Hydration heats

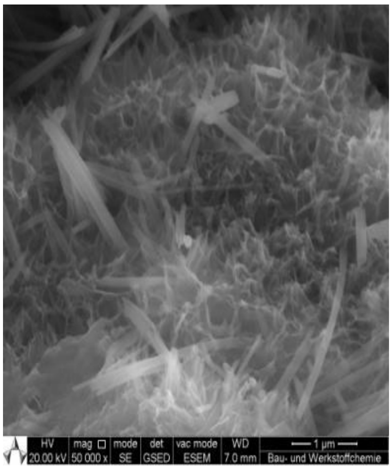
	10 h (g/joule)	24 h (g/joule)
PAAF Compound	112	324
Conventional Additive	110	288

Porosity of prismas which were produced with cements produced with PAAF compound and conventional cement additive are shown in Table 2

Table 2: Total porosity

	Total Porosity (µm)
PAAF Compound	13,5
Conventional Additive	14,6

Environmental scanning electron microscope images of cements which were produced with PAAF compound and conventional cement additive are shown in Image 1 and Image 2 comparatively:



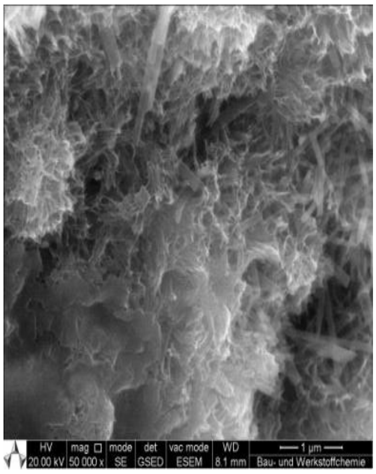


Image 1: Conventional cement additive

Image 2: PAAF cement additive

In this study, effects of cements produced with PAAF compounds to concrete was investigated.

2. MATERIAL AND METHOD

2.1. Cement

K-1 CEM I 42,5 R cement which was produced with conventional cement additive in X factory and P-1 CEM I 42,5 R and P-2 CEM I 42,5 R cements which were produced at two fineness value with PAAF 280 were used in the experiments.

Fineness and blaine values of cements which were used, are shown in Table 3.

Table 3: Fineness and blaine values of K-1, P-1 ve P-2 cements

	40 m residue on sieve (%)	Blaine (N/mm2)
K-1	10,2	3180
P-1	10,5	3140
P-2	11,8	2930

Particle size distribution of K-1, P-1 and P-2 cements are shown in Table 4 and particle size graphics are given in Figure 1 and Figure 2:

Table 4: Particle size distribution of K-1, P-1 and P-2 cements

	D 10	D 50	D 90
K-1	2,78	17,16	57,84
P-1	2,84	17,81	59,56
P-2	2,79	18,83	62,62

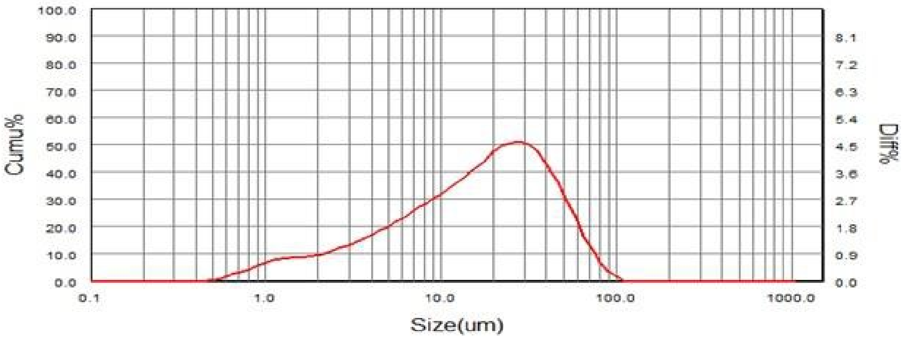


Figure 1: Particle size graphic of K-1 cement

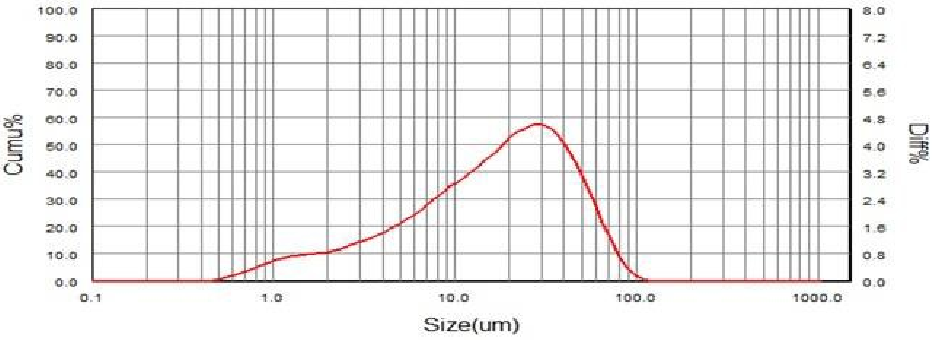


Figure 2: Particle size graphic of P-2 cement

2.2 Aggr egates

Fine aggregate was provided from one source and methylene blue value, water imbibition value, specific surface weight and fineness results dependent on sieve analysis are shown in Table 5:

Table 5: Physical and chemical properties of fine aggregate (crushed sand)

	Fine aggregate (Natural Sand)	Fine aggregate (Crushed Sand)
Methylene blue	0,74	1,48
Specific weight	2,61	2,70
Water imbibition (%)	2,0	2,0
Fine material sieved from 0,063mm (%)	0,9	9,3
Sieve sizes	Sieved (%)	Sieved (%)
4 mm	93	94
2 mm	82	58
1 mm	68	30
0,5 mm	39	14
0,25 mm	10	7
0,125 mm	1	4
Fineness Module	4,1	4,9

Cyclopean aggregate, I no and II no were provided from one source and their physical properties are shown in Table 6:

Table 6: Physical and chemical properties of cyclopean aggregates

	Cyclopean Aggregate (I No)	Cyclopean Aggregate (II No)
Specific weight	2,71	2,72
Water imbibition (%)	0,7	0,9
Fine material sieved from 0,063mm (%)	1,7	1,1
Sieve sizes	Sieved (%)	Sieved (%)
16 mm	-	73
11,2 mm	95	10
8 mm	68	2
5,6 mm	38	-

2.3 Chemical additive

Nitro 168 is used as an additive which is in high level of water reducer/ super plasticizer group according to standards of TS EN 934-2. Chemical analysis of the chemical additive was concluded according to standards of TS EN 934-1 and results are shown in Table 7:

Table 7: Physical and chemical properties of chemical additive

	Density (g/cm3)	pH	Solid Content (%)	Cl Content (%)	Total Alkaline Content
Nitro 168	1,12	9,22	25,79	0,07	1,29

2.4 Experimental study

2 group study was conducted with 3 types of cements which were obtained from industrial experiments. In first group study, no chemical additive was used and concretes were produced with constant w/c ratio. In second group, 1.2% chemical additive was used and w/c ratio was remained constant. Concretes which are in C 25/30 class were produced. Fresh concrete tests and hardened concrete tests of produced concretes were conducted. Properties of concretes which were produced with PAAF compound were compared with properties of conventional additive ones. In the same time, adaptation of concrete chemical additive was investigated in PAAF concretes.

Mixture design of first group study is shown in Table 8:

Table 8: Mixture Design of 1st group study

No	Mixture	Dosage of Cement (kg/m3)	W/C	Chemical Additive (%)	Natural Sand (%)	Crushed Sand (%)	I No (%)	II No (%)
1	K-1 Concrete	250	0,72	0	24	22	26	28
2	P-1 Concrete	250	0,72	0	24	22	26	28
3	P-2 Concrete	250	0,72	0	24	22	26	28

Mixture design of second group study is shown in Table 9:

Table 9: Mixture Design of 2nd group study

No	Mixture	Dosage of Cement (kg/m3)	W/C	Chemical Additive (%)	Natural Sand (%)	Crushed Sand (%)	I No (%)	II No (%)
1	K-1 Concrete	250	0,61	1,0	24	22	26	28
2	P-1 Concrete	250	0,61	1,0	24	22	26	28
3	P-2 Concrete	250	0,61	1,0	24	22	26	28

3. RESULTS AND DISCUSSION

3.1 Results

Results of 1st group study are shown in Table 10:

Table 10: Results of 1st group study

		Temperature		Slump		Air Content	Unit Weight	Strength		
No	Mixture	Lab. (Co)	Concrete (Co)	0' (cm)	30' (cm)	30' (%)	Real (kg/m^3)	1 day (MPa)	7 day (MPa)	28 day (MPa)
1	K-1 Concrete	20,6	23,5	14,0	10,8	2,3	2320	11,8	22,3	29,2
2	P-1 Concrete	20,4	23,6	15,5	11,0	2,0	2328	12,2	24,9	31,4
3	P-2 Concrete	20,5	23,5	16,0	11,0	2,1	2325	11,9	23,4	29,6

Results of 2nd group study are shown in Table 11:

Table 11: Results of 2nd group study

		Temperature		Slump		Air Content	Unit Weight	Strength		
No	Mixture	Lab. (Co)	Concrete (Co)	0' (cm)	30' (cm)	30' (%)	Real (kg/m^3)	1 day (MPa)	7 day (MPa)	28 day (MPa)
1	K-1 Concrete	20,2	22,4	19,5	18,5	1,6	2330	14,1	28,5	34,6
2	P-1 Concrete	20,4	22,3	20,8	18,0	1,2	2338	14,9	28,9	35,7
3	P-2 Concrete	20,6	22,1	21,5	18,8	1,0	2343	14,6	29,5	36,1

3.1 Discussion

When we consider the results of first study, it is observed that in concretes which were produced with constant water, P-1 and P-2 concretes have higher slump values in compares with K-1 concrete. Also it is seen that concretes produced with PAAF cements are more workable than concretes produced with conventional cements.

With respect to results of first study it is stated that concretes produced with PAAF have less air content than conventional concretes and therefore they have higher unit weights. Lower air content is result of having less porosity of concretes produced with PAAF compounds.

When we consider final strengths in first study, strength of PAAF concretes are not lower than conventional concretes, even strength of P-1 concrete is above 2,2M Pa. Being high in slump values of PAAF concretes and having high strength in contrast with producing with constant water are based on more C-S-H gels which are formed and grown due to PAAF compounds' positive effect on hydration reactions.

Results were obtained in second study in which was chemical additive used in parallel with first study. Workability of PAAF concretes is higher, air contents of them are lower and in parallel with unit weights are higher. They were performed in final strength better than conventional concretes. Concrete chemical additive did not perform any adverse reaction in PAAF cements and it did not affect process of concretes development negatively.

It shows that, concretes which were produced with PAAF compounds instead of conventional cement additives, have superior fresh concrete properties and mechanical behaviors.

UK PROCEDURES FOR THE USE OF ADDITIONS AS PART OF CEMENT IN CONCRETE

Chris A Clear

Technical Director of the British Ready-Mixed Concrete Association (BRMCA)

Abstract

The UK uses an Equivalent Performance of Combinations Concept (EPCC) where an addition added at the concrete mixer is considered to perform in the same way as would the same material incorporated into concrete as a constituent of cement to EN 197-1 or EN 14216. This mixer-blend of addition and cement is a 'combination' where BS 8500 (the British Complementary Standard to EN 206) defines this as: "restricted range of Portland cements and additions which, having been combined in the concrete mixer, count fully towards the cement content and water/cement ratio in concrete". The UK procedure for using additions as combinations is called the "Conformity Procedure for Combinations". It only applies for combinations of a CEM I cement of standard strength class 42,5 or greater with either: fly ash to EN 450-1: category A or B; ggbs to EN 15167-1; or limestone fines to BS 7979. On the basis of the established equivalence with respect to chemical, soundness and strength it is accepted that the combination has a durability performance equivalent to the EN 197-1/EN 14216 cement of the same nominal proportions.

Keywords: Additions, cement, combinations, durability, performance

1. INTRODUCTION

The UK ready-mixed concrete industry has played a pivotal role in the adoption of concrete mixer blended combinations of CEM I with additions such as ground granulated blastfurnace slag (ggbs) and fly-ash. Table 1 is a summary of the most significant standards and technical guidance that have played a part in the development of the UK procedures for the use of additions as part of cement in concrete. With hindsight it is possible to categorize these developments into those that could be considered as 'Equivalent Concrete Performance Concept, ECPC' or 'Equivalent Performance of Combinations Concept, EPCC' as defined in the current European Concrete Standard, EN 206. The current UK procedures for the use of additions as part of cement is described, and clearly falls under EPCC. In reality it is the enhanced performance of concretes containing ggbs or fly ash exposed to particular deterioration mechanisms that meant the EPCC was enthusiastically embraced. For this reason the pre-EPCC background is summarized.

2. 1950 TO 1980 - EQUIVALENT CONCRETE PERFORMANCE CONCEPT(ECPC)

Higgins[1] summarised the use of ggbs as an addition in concrete starting with the Trief process, a process whereby the use of wet ground material is added as a slurry at the concrete mixer. Due to the difficulties in storing and transporting wet slurry this process was limited to a small number of projects but included Cluanie and Loyne dams and tunnels constructed in the early 1950s. Notably the use of ggbs in this manner saved 20,000 tonnes of Portland cement and perhaps this inspired the trade name 'CEMSAVE' as used in the early 1960s by Frodingham Cement Ltd the manufacturer of the dry material[2]. The initial technical marketing was based on the performance of the material in concrete, showcasing projects in which the material was successfully used. One of these projects was at a steel works where it could be argued there was a vested interest in the use of the material.

The early marketing of fly ash in the UK, or pulverized-fuel ash (pfa) as it was then called, was mainly based on its performance in concrete supported by a British Standard covering its use in concrete, BS 3892. At this time although pfa was included as an ingredient for use in concrete by the current code for structural concrete CP110, and regular use in dam construction and electric power plant infrastructure[3], its use was not widespread. Essentially it was being left to the Engineer to consider on a case by case basis if the use of fly ash in concrete was technically and economically justified. Most notably pfa was used in construction of the Thames Barrier at Woolwich, where the lower heat generation helped reduce the heat of hydration to reduce the risk of early age thermal cracking making it a performance criterion.

In 1981 the Building Research Establish issued guidance, BRE Digest 250, that also included recommendations about using combinations of Portland cement with either ggbs or pfa for concrete exposed to sulfate bearing soils and groundwaters. At around the same time it became apparent that there was a risk of developing alkali-silica reaction, ASR, where high alkali cement was used with particular sources of aggregate. In 1983 the report of a Working Party Chaired by Hawkins included broad recommendations for the use of pfa and ggbs as effective means of minimizing the risk of damaging ASR. This was a significant driver that increased the use of additions in concrete.

Essentially the late 1970s to early 1980s saw the publication of substantial evidence that combinations of Portland cement and additions are beneficial for particular applications. However, the use of these combinations were not readily accepted by the construction industry for normal building or housing projects, and they were not included as options within the materials and construction standards, an omission that needed to be addressed.

Table 1: UK procedures for combinations – associated standards and technical guidance

First Published	Developments in standards or technical guidance	Equivalent European Standards, cement/addition types or explanatory notes
1904	BS 12 for Portland Cement	EN 197-1 CEM I
1923	BS 146 Portland Blastfurnace Cement	EN 197-1 CEM III/A
1965	BS 3892 Pulverized-fuel ash (pfa) for use in concrete	EN 450 Fly ash
1966	BS 4027 Sulfate-resisting Portland Cement	EN 197-1 CEM I-SR0 & CEM I -SR3
1968	BS 4246 Low heat Portland Blastfurnace Cement	EN 197-1 CEM III/B & C
1972	CP 110 Code of Practice for The Structural use of concrete. Part 1 Design, materials and workmanship.	EN 206 EN 1992, EN 13670
1975	Agrément Certificate No. 75/283. Pozzolan – a selected fly ash for use as a cementitious component in structural concrete.	Covers the use of fine fly ash for use as part of the cement in structural concrete
1981	BRE Digest 250 Concrete in sulphate bearing soils and groundwaters	Recommends the use of ggbs and fly-ash to resist damage from sulfate attack
1982	BS 3892-1 Pfa as a cementitious component in structural concrete	Fine ash, EN 450-1 Category S
	Agrément Certificate Certificate No. 82/1023. Cemsave Ground Granulated Blastfurnace Slag.	Test results comparison with equivalent cements
1983	Hawkins Report. Alkali aggregate reaction - Minimizing the Risk of Alkali-Silica Reaction (ASR) Guidance Notes	Recommends ggbs and fly ash to resist ASR
1984	Quality Scheme for Ready-Mixed Concrete, Technical requirements.	Reference to ‘combinations equivalent to cement…

Table 1(cont.):UK procedures for combinations – associated standards and technical guidance

First Published	Developments in standards or technical guidance	Equivalent European Standards, cement/addition types or explanatory notes
1985	BS 5328: 1981 Specifying concrete including ready-mixed concrete, as amended 1985	Reference to 'combinations equivalent to cement…
	BACMI and BRMCA Combination procedures for CEM I and ggbs	BACMI & BRMCA are Ready-Mixed Concrete Associations
	BS 6588 Portland pfa cements	CEM II/A & B-V
	BS 6610 Portland pozzolanic cement	CEM IV/A & B (V)
1986	BRMCA Combination procedures for Portland Cement and ggbs or pfa	
1986	BS 6699 Ggbs for use with Portland cement	EN 15167 Ggbs
1987	Concrete Society Alkali Silica reaction – minimizing the damage to concrete – Guidance Notes and model specification clauses	Recommends ggbs and fly ash to resist ASR
1988	BRE Digest 330 Alkali aggregate reactions in concrete	Recommends ggbs and fly ash to resist ASR
1990	BS 5328 Concrete	Includes guidance for durability
1992	BS 6699 Revised	Revised to include combination procedures
1993	BS 3892-1 Revised	Revised to include combination procedures
1996	EN 450 Fly ash for concrete	To supersede BS 3892
2000	EN 197-1 Common cements	Portland cement became CEM I
2001	BRE Special Digest 1 Concrete in aggressive ground	Recommends ggbs and fly-ash to resist the thaumasite form of sulfate attack (TSA)
2002	BS 8500 Standard for concrete	Includes conformity procedure for combinations, durability requirements including ASR and sulfate attack
2004	EN 197-4 Low early strength blastfurnace cements	Required to cover blastfurnace cements
2006	EN 15167 for ggbs published	To supersede BS 6699
2011	EN 197-1 Common cement	Incorporates low early strength blastfurnace and SR cements
2013	EN 206 Concrete — Specification, performance, production and conformity	EPCC incorporated in the European Standard for Concrete

3. 1980 ONWARDS, THE EQUIVALENT PERFORMANCE OF COMBINATIONS CONCEPT

Ggbs had been used as a constituent of Portland blastfurnace cement in the UK since 1914 and the British Standard for this cement, BS 146, was published in 1923. This was less than twenty years later than the British Standard for Portland cement, BS 12, which was first published in 1904. Although BS 146 cement was only manufactured and supplied in Scotland it meant that the option to use it was included on most concrete specifications, and included in the code for the structural use of concrete CP 110. This meant that engineers were open to the argument that if they accepted BS 146 cement then they should also accept the equivalent within-mixer blend of ggbs and CEM I. At first this argument was supported by an Agrément Certificate, an early UK version of a European Technical Approval. The most significant section of these certificates simply listed test results of a particular combination of Portland cement and ggbs with the requirements of BS 146, but also included background notes and guidance on manufacture and use.

In 1985 the ready-mixed concrete industry Associations, BACMI and BRMCA, introduced procedures to demonstrate and certify that combinations of Portland cement and ggbs as equivalent to cement-factory Portland blastfurnace or low-heat Portland bastfurnace cements of the same nominal proportions. Also in 1985 British Standards were published for Portland pfa cement and Portland pozzolanic cement which were quickly followed in 1986 by ready-mixed concrete Association procedures for their equivalent combinations. At that time there was only a single national third party quality assurance scheme for ready-mixed concrete, the Quality Scheme for Ready-mixed Concrete, QSRMC. When QSRMC explicitly accepted the combination procedures as confirmation that mixer-blended material was technically equivalent to a cement-factory material of the same nominal proportions then much of the prejudice against using combinations subsided.

After this it became apparent that the most appropriate place to maintain the combination procedures were in British Standards. For this reason the procedures were placed in the British Standard for each addition, that is BS 6699 for ggbs and BS 3892-1 for fly ash. With time these British Standards were to be superseded by their harmonized European versions, BS 15167 for ggbs and EN 450 for fly ash. These European standards would not cover the combination procedures and so a new home would be required. This tied in with developments within Europe where it was agreed that the standards for additions would be appropriately maintained under the concrete committee, rather than the cement committee. So in 2002 the UK combination procedures were unified and moved to the British complementary concrete standard BS 8500, which is where it is today.

4. UK BS 8500-2 ANNEX A. CONFORMITY PROCEDURE FOR COMBINATIONS

Essentially the procedure is a means for establishing limits on the proportions of a single source of addition with a single source of CEM I cement to ensure that the conformity criteria for strength are met. Four stages are involved:

a) The relationship between compressive strength and proportion of addition are established for each CEM I cement, an example of which is shown in Figure 1 for ggbs;

b) Monthly composite samples of the addition and each CEM I cement are tested in combination, and running means of the early and standard strengths are calculated over not less than 6 months and not more than 12 months;
c) Statistical margins are established, or assumed to be 3 or 5 N/mm^2 for early and 28 day strength respectively;
d) The relationships, the running means and the statistical margins, together with the EN 197-1 or EN 14216 requirements for strength class are used to determine the permitted proportions.

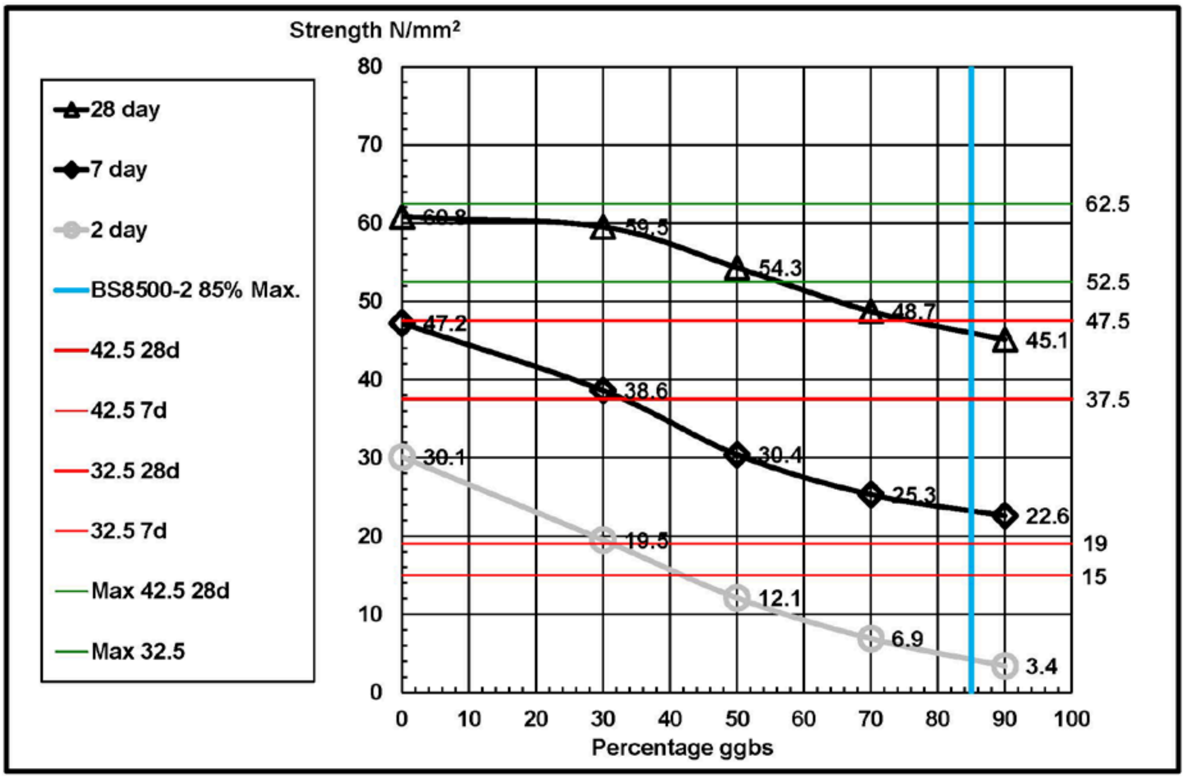


Figure 1: Determination of conformity limits for combinations of CEM I and ggbs

For determining the relationship separate composite samples of the addition and the CEM I are obtained by blending not less than eight spot samples of each material obtained at regular intervals over at least one calendar month. These are used for EN 196-1 strength tests at 2 days, 7 days and at 28 days to cover the range of additions used. For example combinations of CEM I with 0%, 30%, 50% 70% and 90% ggbs to cover the full range for blastfurnace cements. For fly-ash levels up to 60% and for limestone fines up to 20% cover the necessary range.

Once a month bulk average samples of the addition and the CEM I are combined in the ratios:

- 50:50 for ggbs to CEM I cement;
- 30:70 for fly ash to CEM I cement.
- 15:85 for limestone fines to CEM I cement

Again tests for strength are carried out in accordance with BS EN 196-1 at 2 days, 7 days and 28 days. The mean strength of each combination of addition and a specific CEM I cement is the average of the most recent monthly strength tests taken over a period of not less than 6 months and not more than 12 months.

As a simple example to show how the limits on proportions for the conformity of combinations to a strength class are derived it is assumed that the average monthly 2 day, 7 day and 28 day are the same as the values established for the main relationship, as shown in Figure1.

The minimum EN 197-1* requirements for a strength class 42,5L at 7 days and 28 days are 16 N/mm^2 and 42.5 N/mm^2 respectively. With their respective margins these become 19.0 N/mm^2 and 47.5 N/mm^2. From Figure 1 it is evident that even at 90% ggbs the minimum strength at 7 days is 22.6 N/mm^2, higher than the 19 N/mm^2 required for strength class 42,5L. From Figure 1 it is evident that to meet the minimum strength at 28 days of 47.5 N/mm^2 then no more than 75% should be permitted. In practical terms this does not restrict the use of ggbs at 80% because even at this level the combination meets the requirements for strength class 32,5L that is a minimum 28 day strength of 37.5 N/mm^2. Figure 2 is an idealised and simplified example of a Certificate of test in accordance with BS 8500.

Certificate of test in accordance with BS 8500-2: 2006+A1:2012 Annex A

Results of tests carried to determine the EN 196 compressive strength of a combination:

50% ggbs	to EN 15167	Slag Company, West Ham London Works
50% CEM I	to EN 197-1	Cement Company, Manchester Works

Compressive strength at 7 days 30.4 N/mm^2
Compressive strength at 28 days 54.3 N/mm^2

Combination	Strength Class 32.5L min	Strength Class 32.5L max	Strength Class 42.5L min	Strength Class 42.5L max	Strength Class 52.5L min	Strength Class 52.5L max
CII-S			6%	35%		
CIIIA			36%	65%		
CIIIB	66%	80%	66%	75%*		

* Value restricted by 28 day strength

Signature:

Date: 06-Jun-15

Figure 2: Certificate of test for combinations of West Ham ggbs with Manchester CEM I.

5. THE MARKET FOR CEMENT AND ADDITIONS

Figure 3 show the annual cement and addition consumption in Great Britain, GB, from 1980 to 2013, as well as the percentage share of additions compared to the total cementitious sales.

* The current BS 8500-2 Annex A limit is 20 N/mm^2 at 7 days as it was set before EN 197-1:2011 established the value as 16 N/mm^2.

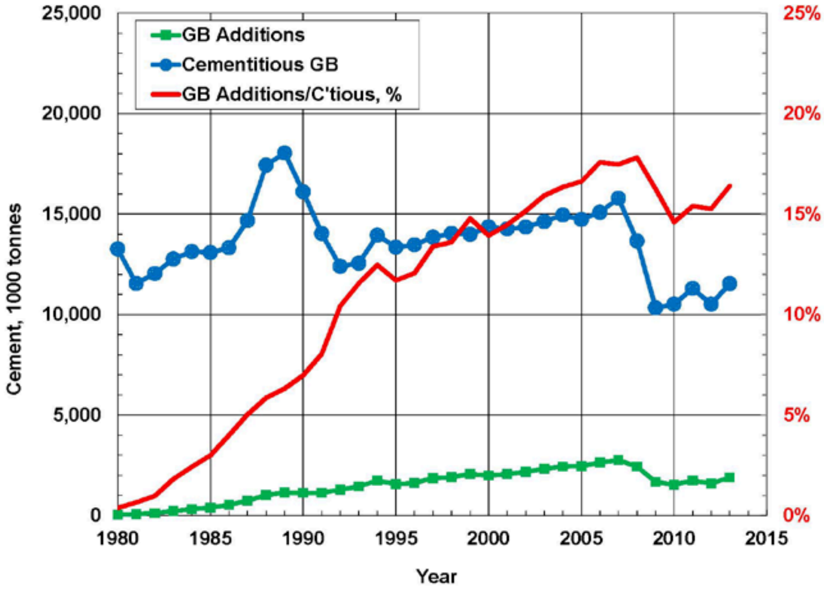


Figure 3: GB cement and addition consumption 1980 to 2013

From Figure 3 it is evident that there has been a steady increase in the market share of additions, peaking at around 18% in 2008 prior to a sharp recession. It is not possible to give a definitive reason why the growth in the use of additions levelled off from 2008. It could be an unknown factor related to the recession, constraints in availability of fly ash and ggbs which depend on the level of pulverised-coal used in power stations and the volume of iron made in the UK, or a combination of both. Notwithstanding this, a 16% or so share of the market is a significant proportion and there are indications that increased sustainability pressures would see the share increase should suitable volumes of additions be available.

6. CONCLUSIONS

There are many advantages in incorporating additions such as ggbs, fly ash and limestone fines with cement for use in concrete and there has been a steady growth in their use in the UK from around 1980. This has only been possible due to substantial technical marketing in terms of making representations to engineers and other specifiers, where a necessary support was the establishment of a formal Equivalent Performance of Combinations Concept. To be credible the EPCC should include some level of third party certification that the combinations used are technically indistinguishable from their equivalent cement-factory products at the same nominal proportions. This was initially achieved in the UK by a third party quality assurance scheme for ready-mixed concrete recognising the ready-mixed concrete Association's own procedures. The ready-mixed concrete industry then helped ensure these procedures were incorporated into National Standards from which recognition was formalised as an EPCC procedure to the European Standard for Concrete, EN 206: 2013.

REFERENCES

[1] Higgins, D. D. Fifty years experience in the UK using ggbs as an addition to concrete. RILEM Conference on Material Science – MATSCI, Aachen 2010.

[2] Taylor I F. Cutting costs with slag cement. Civil Engineering April 1974.

[3] Sear, L. K. A. The properties and use of coal fly ash. Thomas Telford. London, 2001

ENHANCED STRENGTH POROUS CONCRETE FOR SAFETY APPLICATIONS

Ayda. S. Agar Ozbek (1,2), Jaap Weerheijm (2,3) and Klaas van Breugel (2)

(1) Department of Civil Engineering, ITU, Istanbul, Turkey

(2) Faculty of Civil Engineering and Geosciences, TU DELFT, Delft, The Netherlands

(3) TNO Defense, Safety and Security, Rijswijk, The Netherlands

Abstract

Porous concrete is a cementitious material composed of gap graded aggregates, covered with a thin layer of cement paste, assembled by the cement paste layers partially being in contact. A research project was undertaken aiming to design a special type of concrete that fractures into small fragments under impact loading, to be used in safety applications such as protective walls for important structures or storages for explosives. The motive behind this research was the fact that in case of an explosion or impact, the large concrete debris that are formed can be fatal for the exposed environment. Therefore, porous concrete incorporating a high amount of air voids, which facilitate the formation of multiple cracking and the subsequent energy dissipation and multiple fragmentation under impact loading, was selected to be investigated. In the scope of the research, static experiments at different scales were performed while different dynamic testing techniques were used to assess the impact performance. The experimental results were also supported by numerical analyses to be able to better elaborate on the effects of various parameters. Porous concretes that fractured into small fragments under impact loading, while having sufficient static strengths were obtained and investigated both experimentally and numerically.

Keywords: Porous concrete, impact, fragmentation, dynamic, multi-scale analyses.

1. INTRODUCTION

Explosion is broadly defined as a high amplitude and sudden release of energy [1]. In case of an explosion taking place close to or inside a concrete structure, apart from the dangers of the explosive itself, the hazard due to the flying debris from the concrete construction is an important threat. Structures, that carry higher risks of experiencing such extreme loadings during their service lives, such as safety walls outside important buildings or storages for explosives, are of particular interest in terms of mitigating the effects of a probable explosion. In order to accomplish this goal, a research project was undertaken on designing a cementitious material fracturing into small size fragments in an explosion while having sufficient static strength to carry the service loads of the structure.

Responses of cementitious materials to dynamic loading are significantly different from those under static loading mainly due to the complexities associated with strain-rate sensitivity. Especially in the last decades, extensive research efforts have been focused on understanding the behavior of concrete structures under extreme dynamic loadings [2-3]. While most studies aim at designing materials that resist impact loading, developing a cementitious material that is expected to fracture and disintegrate under impact, but at the same time remains intact during its service life was the main concern of this particular project. To accomplish that aim, an organized sensitivity study was conducted on various forms of cementitious materials. As a result, enhanced strength porous concretes, that facilitate the formation of multiple cracks and subsequently fracture into small fragments when exposed to impact loading, were attained. Porous concrete is a construction material that has been investigated by several researchers and has been used in various applications that require permeability, noise absorption or heat insulation [4,5]. However, its dynamic performance was not very much of interest due to its weak mechanical properties as a consequence of its intentionally increased meso-scale porosity. Due to the high percentage of its meso-size air pores, porous concrete normally has a moderate static strength compared to plain concrete while owing to its porous structure and aggregate distribution, it has a favorable characteristic of forming multiple cracks which was the key property that lead to selecting porous concrete in the scope of this research project. The formation of multiple cracking also facilitates more energy dissipation throughout the material. Therefore, in the process of modifying the material, the main focus was to enhance the static strength properties while maintaining the high porosity.

2. STATIC AND DYNAMIC PROPERTIES OF POROUS CONCRETES

2.1 Materials

In the process of modifying the mixture properties, a very large number of different porous concrete mixtures were produced and tested in the scope of the project. The compositions of some selected representative mixtures are given in Table 1. Aiming to enhance the mechanical properties, an organized sensitivity study was conducted where the numerous factors that affect the properties of porous concrete were considered along with their interactions. Experiments at different scales were performed to determine the effectiveness of the various factors while the outcome of the tests guided the modification process of the material. In the scope of the research, the compressive and tensile tests were performed at macro-scale (testing of the porous concrete structure) while the tensile tests were also conducted at meso-scale (testing of ITZ and cement paste phases). The samples that have gone through mechanical testing were also analyzed through computed tomography (CT) to visualize the crack patterns and better understand the fracturing behaviour. Meso-scale porosity was also measured through CT scanning coupled with image processing. The fracture surfaces of the samples that have been tested at meso-scale ITZ testing were observed under environmental scanning electron microscope (ESEM) to be able to see a relation between the ITZ strength and the fracture surfaces formed. The macro scale samples were also observed using ESEM to be able to characterize the materials more thoroughly and support the mechanical test results.

Table 1: Compositional properties of selected representative porous concrete mixtures

Mixture code	PRC1	PRC2	PRC3	PRC4	PRC5	PRC6	PRC7
Crushed basalt (2-4 mm) (gr)	-	2000	1000	-	-	2000	1000
Crushed basalt (4-8 mm) (gr)	2000	-	1000	-	2000	-	1000
River gravel (4-8 mm) (gr)	-	-	-	2000	-	-	-
Cement (gr)	351	351	351	351	298	298	298
Microsilica (gr)	-	-	-	-	53	53	53
Water (gr)	105	105	105	105	105	105	105
Superplasticizer(gr)	0.97	0.97	0.97	0.97	1.30	1.30	1.30
Set retarder (gr)	1.20	1.20	1.20	1.20	1.20	1.20	1.20

2.2 Macro-scale uniaxial tension and compression testing

Macro-scale, deformation controlled uniaxial compression and tension tests were performed at the loading rates of 1 μm/sec and 0.1 μm/sec, respectively. The deformation measurements were made over the whole height of the samples at both tensile and compressive tests and the average of four LVDTs were used as the feed-back signal. In the tests, 83 mm diameter x 160 mm height and 83 mm diameter x 80 mm height cylindrical specimens were used for compression and tension, respectively. Because porous concrete readily incorporates numerous notches due to its porous nature, no notches were made on the tensile test samples. To be able to prevent the snap-back behavior, the tensile sample height was kept at 80 mm. During the tensile tests, the specimens failed at one visible major crack. The macro-scale compression and tension test results of the selected porous concrete mixtures are presented in Table 2.

2.3 Meso-scale uniaxial tension and compression testing coupled with complementary microscopic observations

Proper material design considerations require specific knowledge on every phase present in concrete i.e. aggregates, bulk cement paste and interfacial transition zones (ITZ). Among the three phases, ITZ is the one that has the least known mechanical properties. While the meso-scale testing of cement paste was more straightforward, meso-scale composite ITZ samples were produced using aggregates with an 8 mm x 8 mm square cross-section, taking the 4-8 mm aggregate size range of the porous concretes as reference to be able to have shrinkage conditions as similar as possible to the macro size porous concrete samples. Keeping the natural surface of the crushed aggregates was an important feature of the samples produced, in order to have a more realistic ITZ structure. Displacement controlled meso-scale tests were conducted on composite samples, shown in Figure 1 consisting of aggregate, cement paste and the ITZ phase that is generated in between, and on meso-size cement paste samples. In all of the composite samples tested, the failure occurred in the immediate vicinity of the aggregate which showed that the failure happened at the ITZ. Therefore, the peak load that is measured can be used to determine the tensile strength of the ITZ phase. In the tests of plain cement paste-basalt aggregate composite samples, a mean ITZ tensile or bonding strength of 0.95 MPa was found. The basalt samples containing 15 percent silica fume provided a mean bonding strength value of 1.3 MPa. A clear increase in bonding strength was seen in the results with the presence of silica fume. River gravel on the other hand consists of various types of aggregates with different mineralogical properties. The presence of the aggregates that form very weak bonds (for example about 0.5 MPa measured in this project for feldspar and chert) with cement paste can be said to be the reason for the samples with gravel having lower strengths in the tests. However it should also be noted that among the gravel aggregates that were tested at meso-scale, there was a specific type of gravel that gave especially very high bonding strength values (an average of 1.7 MPa) in the tests which was identified using XRD and microscopical analyses to be a type of quartzite with the inclusions of calcite.

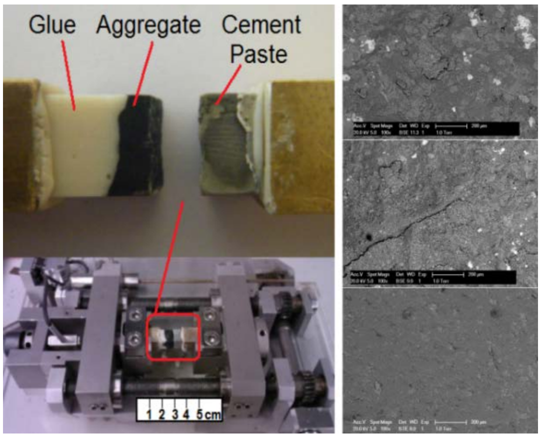


Figure 1: Meso-scale composite ITZ sample, meso-scale testing set-up and ESEM images of the aggregate surfaces of ITZ samples after testing

After the meso-scale testing, both the cement paste surface and the aggregate surface of the interface were characterized using ESEM, as seen in Figure 1, to be able to find a relation between the ITZ strength and the images of the fractured composite samples. Even though making quantitative comparison was not possible, it can qualitatively be said that for composite samples where there is only cement present as binder in the cement paste, as in the top figure, the aggregate surface contains a small amount of cement paste after the fracture. Whereas in samples where the cement paste includes silica fume, on the aggregate side of the broken surface there are large pieces of cement paste present, as in the middle image. The bottom image is taken from the aggregate side of a composite sample containing feldspar aggregate and plain cement paste which gave very low bonding strength values in the tests. It can be observed in that image that there is nearly no remains of cement paste present on the aggregate surface. It can generally be said for the images taken from samples that have gone through ITZ testing that as the bonding strength increases there is more cement paste left on the aggregate surface.

Displacement controlled meso-scale uniaxial tensile tests were also conducted on cement paste samples having the same properties as the cement pastes used in the macro-size samples. Mean tensile strengths of 3.12 MPa and 2.59 MPa were found for plain cement paste and cement paste with 15 percent silica fume samples, respectively, which are lower than expected. This can be caused by the shrinkage cracks that are especially effective because the samples were very small, but it should also be noted that this is a situation that is also valid for the cement paste bridges present in porous concrete.

2.4 Instrumented drop weight impact testing

The impact tests were carried out using an instrumented drop-weight impact test set-up. In the experiments, the specimen was placed vertically on a steel base structure, which also serves as a steel buffer plate that functions as a wave sink at the impact experiments. The impactor was dropped from approximately 1.2 m to provide striking velocities ranging between 4.0 - 4.7 m/sec.

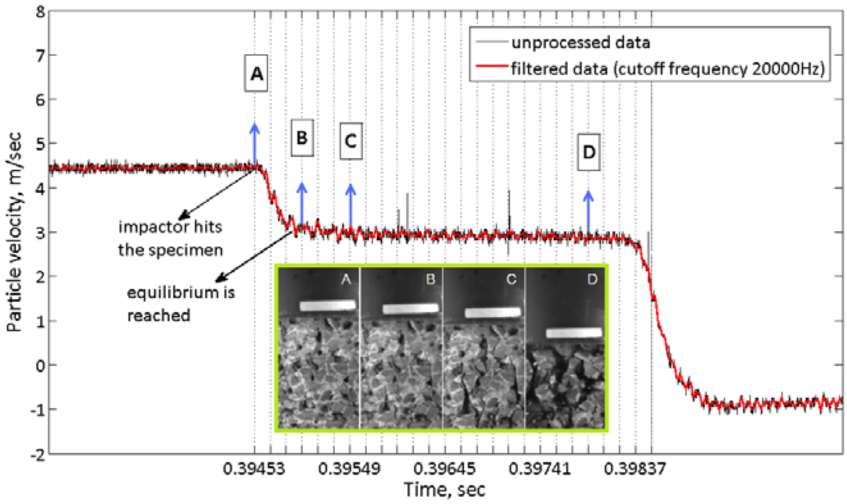


Figure 2: A representative particle velocity time history graph for the interface between the sample and the impactor used in impact stress calculations

For the tests, two measurement techniques were developed, using laser Doppler Velocimetry (LDV) and high speed photography as the monitoring tools, while stress gauge measurements were also taken. As stress gauge measurements directly provide the stress value, in the LDV and high speed photography methods, the particle velocity histories of the interface (between the impactor and the concrete target) were first captured. The particle velocity data (an example of which is seen in Figure 2) were subsequently analyzed using a special reverberation application of the impedance mismatch method [6]. As a result of the analyses, the impact strengths of the target concrete samples were obtained. Since the concretes that were tested in the experiments have failed and reached their dynamic strength values, the stress that is calculated is also the dynamic strength of the specimen tested. The key feature of this technique is that the impact stress is calculated using the dynamic impedance properties of only the impactor and that the properties of the target are not involved in the calculations. Therefore, while all the information is obtained from the well-defined metal drop weight, the target can be an unknown material. The impact strengths of the selected porous concrete mixtures are also presented in Table 2.

2.5 Experimental Results

Table 2: Static and dynamic test results of the selected representative porous concrete mixtures

Mixture code	Compressive strength (MPa)	Meso-scale Porosity (%)	Tensile strength (MPa)	Young's Modulus (MPa)	Fracture energy (N/m)	Impact Strength (MPa)
PRC1	34.8	21.8	1.91	23413	105.1	66.5
PRC2	41.9	20.3	2.73	26644	110.0	76.8
PRC3	50.5	18.8	2.95	32177	110.3	86.0
PRC4	29.6	17.9	1.98	24841	101.2	56.2
PRC5	31.6	22.0	1.85	23361	120.3	53.1
PRC6	44.8	20.1	2.80	26487	107.0	79.7
PRC7	48.8	18.6	2.67	29605	96.7	84.4

According to the test results, it was generally seen that as two sizes of aggregates (50% 2-4 – 50% 4-8 mm) were used instead of using single sized aggregates (2-4 or 4-8 mm), the dynamic strengths of porous concretes as well as their static strengths increased. Among all the mixtures tested, PRC3 and PRC7, which included aggregates of two sizes, offered the highest static and dynamic strengths. As their porosity values (18.8% and 18.6%, respectively), which were obtained using CT scanning and image analysis, are also compared with the porosities of the other mixtures, it is seen that strength increase is mainly caused by the fact that aggregate grading is coupled with the total porosity which is the primary factor that affects the strength.

Meso-scale ITZ testing provided important information on the effects of cement paste composition and aggregate type on the mechanical properties of the ITZ. According to the meso-scale testing results, it was clearly seen that silica fume had an enhancing effect on ITZ properties. This effect is less pronounced in porous concretes where the total amount of ITZ phase present in the material is highly reduced compared to normal concrete due to the lack of fine aggregates. Along with the reason that the total amount of ITZ phase is drastically less, the CT scans of fractured samples revealed that in porous concretes crack patterns are very much influenced by the distinct porous structure where the cracks are forced to propagate into locations guided by the geometry of the skeleton structure determined by the aggregate grading. When the aggregates are finer, cracks tend to go more through the ITZ while they go more frequently through the aggregates as the aggregates are very coarse. This also supports the explanation that silica fume, which enhances the ITZ properties, has a slight enhancing effect on the porous concrete with finer aggregates while it does not have such an effect on the mixtures with coarser aggregates.

3. NUMERICAL ANALYSES ON DYNAMIC PROPERTIES AND FRAGMENTATION BEHAVIOUR OF POROUS CONCRETES

The objective of the numerical work was simulating the behavior of different porous concretes under impact loading in order to understand the effect of various control parameters on their impact behavior. In the numerical analyses, the distinctive properties of the material compared to normal concrete were the presence of the arbitrary shaped meso-scale air pores and the contacts forming between the free surfaces of those pores as the loading proceeds. For the numerical analyses, the finite element analysis software ABAQUS/Explicit was used where explicit time integration was adopted. Concrete Damaged Plasticity Model was used to define the material properties of the cementitious phases that are present in the material [7,8]. In order to realistically represent a porous concrete as a four-phase material (incorporating aggregates, interfacial transition zones (ITZ), bulk cement paste and air), the numerical study first started by acquiring the shapes and distribution of the aggregates in the samples through 3D computed tomography. A mesh generation program was developed to use this data in generating a finite element mesh. The numerical simulations were conducted using an axisymmetric geometry. Because contact properties are very important in the behavior of porous concrete, self-contact was defined between the concrete surfaces coming into contact while surface-to-surface contact was defined between steel and concrete. In the analyses, the crack patterns formed under the impact loading are visualized using the compressive and tensile damage variables (DamageC and DamageT).

The graph on the left of Figure 3 shows the numerical impact stress time history result of a porous concrete mixture with 4-8 mm basalt aggregates (PRC1 in Table 1) under drop weight impact loading. The numerical results obtained for the impact strengths of the samples were in a good agreement with the experimental results, an example of which was given for PRC1 (66.5 MPa experimental impact strength as seen in Table 2). As the damage variable contours of all the mixtures are observed, it can be said that multiple crack patterns have developed and that the porous concretes that have been numerically analyzed have fractured into many small size fragments. The slight differences in the fragmentation behaviors of different mixtures could also be observed in the numerical results.

The crack patterns predicted by the damage evolution contours, where highly damaged (damage variable$\geq$ 0.9) elements were removed, were generally observed to be good estimations of the real crack patterns according to the high speed photography videos and the fragments collected during impact testing. However quantifying them numerically was also important because fragment size distribution was a very important parameter in this research project.

In the fragment size analyses, the 2D fragment sizes, that have been separately determined for every piece of fragment by using the image processing tools and a short code written in MATLAB, were grouped into size intervals as seen in the graph in Figure 4. Since during the experiments the fragments were collected and afterwards sieved to determine the fragment sizes, the numerical results could also be compared with the sieve analysis results by converting the 2D sizes (areas of the fragments) from the numerical analysis to standard sieve sizes by calculating the diameter of circles having the same areas and again grouping them in terms of the sizes of the standard sieves. Fragment sizes estimated by the numerical analyses were seen to be in agreement with the real fragment size distributions extracted from experiments

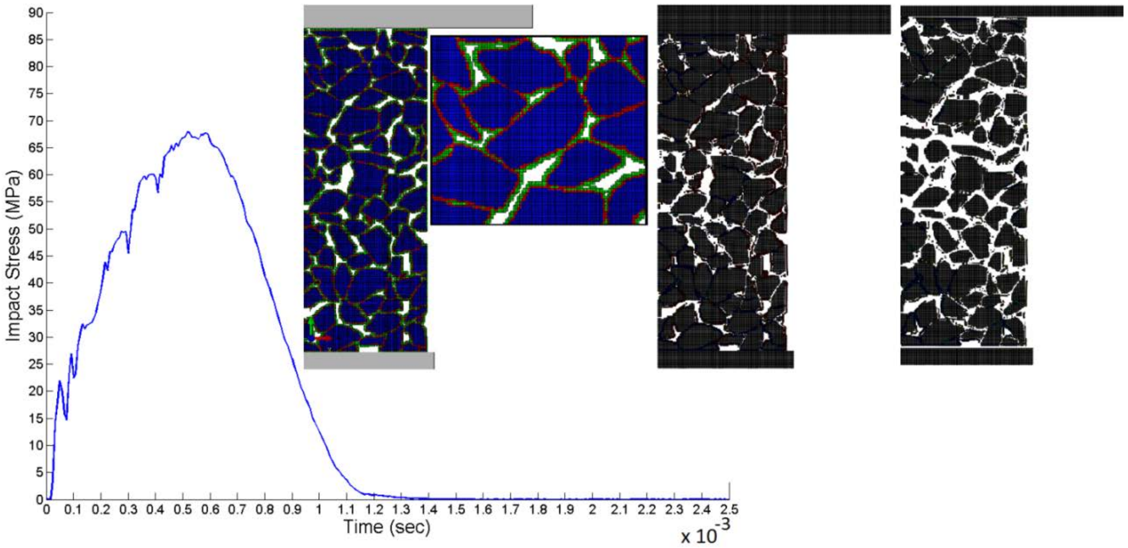


Figure 3: Impact stress time history of PRC1 obtained numerically, the PRC1 finite element mesh and the progress of fragmentation during the analysis (two selected time steps)

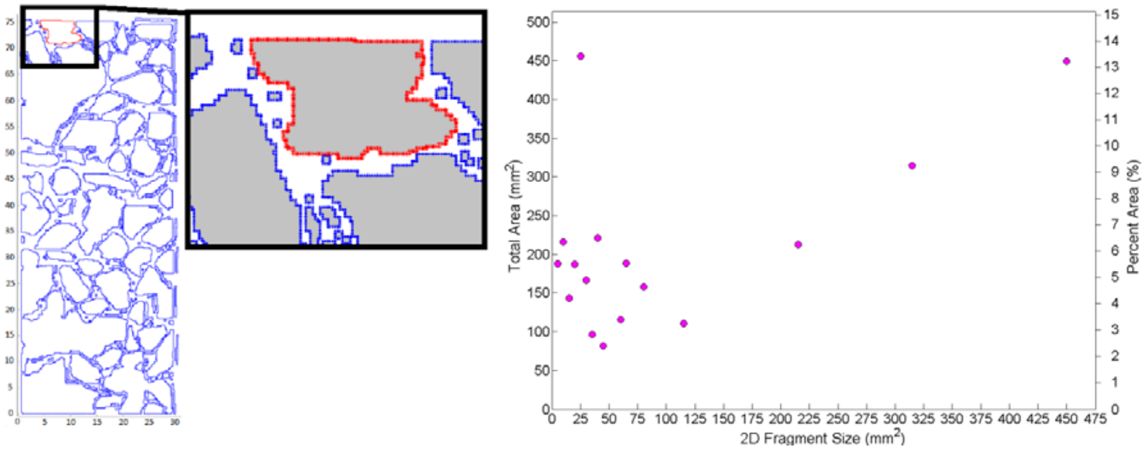


Figure 4: Demonstration of a fragment size analysis conducted on PRC1, 2D fragment size output

4. CONCLUSIONS

The main conclusions that can be drawn from this work can be summarized as follows:

- By modifying the compositional properties as well as the method of compaction, porous concretes with improved static and impact strengths (at the range of 30–50 MPa static and 55-85 MPa dynamic strengths) were obtained which can potentially be used in the specified safety applications.
- Among the parameters that have been investigated, aggregate properties have the most dominant effect on the strength properties of porous concretes due to the coarse aggregates being very effective in the formation of the skeleton structure of the material.
- Experiments at different scales (macro and meso-scale testing supported by microscopical observations) were performed to determine the effect of the various

factors on the performance of porous concretes while the outcome of the tests also guided the modification process of the material.

- In order to quantify the impact stresses in drop weight impact experiments conducted on porous concretes, two experimental measurement techniques, using LDV and high speed photography as the monitoring tools, were developed and sufficiently applied on porous concretes having different dynamic strengths.
- In the numerical part of the study, explicit time integration calculations were used in analyzing the impact properties of the porous concretes investigated.The simulation results were in good agreement with the experimental results both in terms of quantifying the impact strength as well as demonstrating a realistic crack pattern formation for the types of porous concretes that have been analyzed.

ACKNOWLEDGEMENTS

This research project was funded by Delft University of Technology and the Netherlands Ministry of Defense.

REFERENCES

[1] Nolan, D. P., Handbook of Fire and Explosion Protection Engineering Principles, Elsevier. (2010)

[2] Comité Euro-International du Béton,‘Concrete Structures under Impact and Impulsive Loading‘, *CEB Bulletin* 187 (1988).

[3] Weerheijm, J., Vegt, I. and van Breugel, K., ‘Research developments and experimental data on dynamic concrete behaviour‘, C.U. Grosse (ed.), Advances in construction materials; Berlin: Springer, (2007) 765-773

[4] Yang J and Jiang G., ‘Experimental study on properties of pervious concrete’, *Cement and Concrete Research*, **33** (3) (2003) 381-386.

[5] Ghafoori N, Dutta S. ‘Building and nonpavement applications of no-fines concrete’, *Journal of Materials in Civil Engineering*, **7** (4) (1995) 286-289.

[6] Agar Ozbek, A.S., Weerheijm; J., Schlangen, E; and van Breugel, K.’Drop weight impact strength measurement method for porous concrete using laser Doppler velocimetry’, *Journal of Materials in Civil Engineering*, **24** (2012) 1328-1336.

[7] Lubliner, J., Oliver, J., Oller, S. and Onate, E., ‘A plastic-damage model for concrete’, *International Journal of Solids and Structures*, **25** (3) (1989) 299-326.

[8] Jankowiak, T. and Lodygowski, T., ‘Identification of parameters of concrete damage plasticity constitutive model’, *Foundations of Civil and Environmental Engineering*, **6** (2005) 53-69.

A COMPARATIVE STUDY ON THE THERMO-PHYSICAL PROPERTIES OF LIGHT-WEIGHT, NORMAL-WEIGHT AND HIGH-STRENGTH CONCRETE

Fevziye Aköz (1), Nabi Yüzer (2), Nihat Kabay (2), Ahmet B. Kızılkanat (2) and Büşra Aktürk (2)

(1) Hasan Kalyoncu University, Civil Engineering Department, Construction Materials Division, Turkey

(2) Yildiz Technical University, Civil Engineering Department, Construction Materials Division, Turkey

Abstract

In Turkey, the importance of energy efficiency in construction industry during different phases (manufacturing to consumption) is increasing due to the fact that, energy productions resource extensive process which lead to over utilization of limited natural resources and environmental degradation. Concrete is a widely used construction material in Turkey as it is economical and durable in long run. Thus, thermal performance of concrete is an important factor for its widespread applications and it depends on parameters such as density, moisture content and components such as aggregate.

In this study, thermal performance of three different types of concrete; light-weight concrete, normal-weight concrete and high-strength concrete were investigated. Thermal conductivity coefficient and water vapour diffusion resistance factor of these concretes were experimentally determined and their thermo-physical properties were compared. Thermo-physical properties of concrete are affected by mineral additives and aggregate type and polymer fiber existence.

Keywords: High-strength concrete, light-weight concrete, normal-weight concrete, thermal conductivity, water vapour diffusion resistance factor

1. INTRODUCTION

The increasing energy demand is one of the most important problems faced by the developing and the developed world in the 21^{st} century. Currently, fossil fuels are one of the major sources of energy, but they have disadvantages such as environment pollution, uneconomical, non-renewable and ozone layer depletion. Out of the total energy generated for commercial use, approximately 50% of the total energy is either used to light the buildings or maintain the optimum temperature for living. Nearly 40% of the total energy in the United States is utilized by commercial buildings, corporate offices and residential complexes out of which a significant portion is spent on heating or cooling and lighting the interior space. In 2009, the residential and commercial end-use sectors accounted for 22% and 19%, respectively, of carbon dioxide emissions from fossil fuel combustion, of which 70 to 77% of these emissions were attributed to electricity consumption for lighting, heating, cooling and operating appliances [1].

In Turkey, the importance of energy efficiency is increasing due to the fact that, energy production is expensive and also has some disastrous effects on the natural resources and environment. Thus, researchers are conducting many studies to increase the efficiency of energy consumption in buildings [2-5]. Depending on the ambient temperature, the buildings loss or gain heat. Since the amount of these losses or gains determines the energy consumption for heating and cooling, heat loss and gain should be decreased in the living spaces of the buildings.

Different type of concretes such as light-weight, normal-weight and high-strength are common used construction materials in the buildings the knowledge on the thermal properties of concrete provides proper thermal insulation and fire safety design of buildings. Thermal properties of concrete depend on its density, porosity, moisture content, and mix design. Normal-weight concrete's thermal conductivity ranges between 1.0 and 3.6 W/mºK [6]. The oven-dry density of structural lightweight aggregate concretes ranges from 800 to 2000 kg/m^3 compared with 2000to 2600 kg/m^3 for normal weight concretes [7]. Light-weight concrete has a thermal resistance capacity of 6 times more than conventional concrete [8] and can be used in constructions to provide thermal insulation. Generally, concrete with a compressive strength over 50 MPa is classified as high strength concrete (HSC) [9]. Due to their low porosity, HSCs' thermal conductivity coefficient (TCC) is higher than normal weight concrete.

Lower thermal conductivity of concrete, brings delayed heat transmission in case of fire. Condensation may occur in buildings according to the thermal and water vapour resistance of materials and the order of each layer. The condensation control is performed according to TS 825. In this control the main parameters are thermal conductivity and water vapour diffusion resistance factor of the materials [10]. Water vapour diffusion resistance factor (VDRF) is related to the pore structure of materials. More porous structure eases the vapour transmission. Kearsley and Wainwright (2001) reported that a reduction in density leads to a marked increase in water vapour permeability of concrete [11]. In another study it was found that the increase in porosity of concrete resulted in up to 30% reduction in water vapour diffusion resistance factor [12].

In this study, thermal performance of three different types of concrete; light-weight, normal-weight and high-strength concrete were investigated. TCC and VDRF of these concretes were experimentally determined and results compared.

2. EXPERIMENTAL DETAILS

2.1 Materials and mix design

2.1.1 High-strength concrete

HSC mixes were produced by using Portland cement (CEM I 42.5R), two types of pozzolans (silica fume and granulated blast furnace slag), and three types of aggregates. Silica fume and granulated blast furnace slag was incorporated to all HSC mixes by 7% and 10% of cement by weight, respectively. Physical properties of aggregates are used in HSC mixes are listed in Table 1.

Table 1: Physical properties of aggregates

Type of aggregate	Particle density (g/cm^3)	Maximum aggregate size (mm)
Limestone coarse aggregate	2.76	12
Crushed limestone sand	2.72	4
Siliceous sand	2.61	4

Details of concrete mix proportions are presented in Table 2.

Table 2: HSC mix proportions

Materials	HSC1	HSC2	HSC3	HSC4
Cement (kg/m^3)	450.0	450.0	450.0	450.0
Water (kg/m^3)	127.5	127.5	127.5	127.5
Coarse aggregate (kg/m^3)	1128	1128	1128	1128
Crushed sand (kg/m^3)	371.0	371.0	371.0	371.0
Natural sand (kg/m^3)	355	355	355	355
Silica fume (kg/m^3)	31.5	31.5	31.5	31.5
Blast furnace slag (kg/m^3)	45.0	45.0	45.0	45.0
Polypropylene (kg/m^3)	-	0.90	1.80	2.25
Superplasticizer (kg/m^3)	1.70	1.75	1.80	1.90
Unit weight (kg/m^3)	2610	2610	2610	2580

2.1.2 Normal-weight concrete

A total of 4 series of concrete namely NC1, NC2, NC3 and NC4 were made with ordinary Portland cement (CEM I 42.5 R), two types of aggregates; siliceous and calcareous, three types of pozzolans; silica fume (SF), granulated blast furnace slag (GBFS) and fly ash (FA). SF, FA and GBFS were replaced in 5, 20 and 40% by weight of cement, respectively. The water-cementitious material ratio was kept at 0.50, and the workability was adjusted by using a superplasticizer at required dosages (1.1–1.15%). NC mix proportions are given in Table 3.

Table 3: NC mix proportions

Series	Cement (kg)	Mineral additives (kg)			Aggregates (kg)	SP (%)	Unit weight (kg/m^3)
		Silica fume	Slag	Fly ash			
NC1	346	--	--	--	2005	1.10	2522
NC2	328	17	--	--	2001	1.15	2507
NC3	204	--	136	--	1971	1.15	2507
NC4	270	--	--	68	1961	1.15	2488

2.1.3 Light-weight concrete

A total of 4 series of LC were made with constant water to cement ratio and two different cement dosages. Acidic pumice aggregate with 8 mm maximum size was used as lightweight aggregate in all series.

Table 4 shows the mix proportions of LC mixes, in some series air entraining agent (AEA) was used to enhance thermal properties.

Table 4: Physical properties of aggregates

Materials		0–1 (mm)	1–2 (mm)	2–4 (mm)	4–8 (mm)	Crushed Sand
β (kg/m^3) Dry		795	627	579	563	1750
γ (kg/m^3) SSD		1803	1565	1418	1354	2670
Mixture ratio (%)		20	15	30	20	15
Water absorption (%)	10 min	8.7	8.7	11.5	10.5	-
	30 min	9.4	9.1	11.9	10.9	-
	1 hour	10.2	9.5	12.6	11.2	-
	24 hours	10.9	13.9	16.1	16.7	-

Table 5: Light-weight concrete mix proportions

Code	Aggregate (kg)		Cement (kg)	Water (kg)	Unit weight (kg/m^3)	SP (%)	AEA (%)
	Pumice	Crushed sand					
LC1	633	270	350	192.5	1570	2.2	-
LC2	858	270	350	192.5	1455	2.2	0.3
LC3	705	218	500	275.0	1700	1.0	-
LC4	688	217	500	275.0	1465	1.0	0.3

2.2 Specimen preparation

In order to determine compressive strength, TCC and VDRF, cylindrical specimens with dimensions of Φ100/200 mm, plate specimens with dimensions of 300×300×40 mm and cylinder specimens with dimensions of Φ100/40 mm were cast respectively. All specimens were stored in a water tank at 20±2°C until testing. Tests were conducted at 28 days.

2.3 Methods

Compressive strength (CS) was determined according to EN 12390-3 (2010) [13]. The test was conducted on 6 cylindrical specimens for high-strength concrete and on 3 cylindrical specimens for normal and lightweight concrete.

Vapor diffusion resistance factor (VDRF) was determined according to TS EN 12086 [14]. In this test, the specimens were placed in vapor-tight cups containing a sorbent ($CaCl_2$). The cups were then placed in a controlled atmosphere cabinet at constant air temperature and relative humidity and weighed at 24 h time intervals in order to determine the quantity of moisture diffused through the specimen. Weighing was repeated until the mass per unit time was no longer subject to changes and steady state values of mass gain were determined for the last five readings.

The water vapor diffusion factor, μ, was determined using the following equation (Eq. 1):

$$\mu = \frac{Da}{D} \qquad D = \frac{\Delta m \times d \times R \times T}{S \times \tau \times M \times \Delta Pp} \tag{1}$$

where *Da* ($m^2\ s^{-1}$) is the diffusion coefficient of water vapour in the air, *Δm*(kg) is the amount of water vapor diffused through the sample, *d* (m) the sample thickness, *S* (m^2) the specimen surface, *τ* (s) the period of time corresponding to the transport of mass of water vapour (Δm, Δpp) (Pa) the difference between partial water vapor pressure in the air under and above specific specimen surface, *R* ($J\ mol^{-1}\ K^{-1}$) the universal gas constant, *M* ($kg\ mol^{-1}$) the molar mass of water and *T* (K) is the absolute temperature.

The TCC of concrete was determined according to TS ISO 8302 [15]. Test is performed using a double sided apparatus. Specimens were placed on either side of the hot surface assembly. The heat transferred through the specimens is equal to the power supplied to the main heater. Thermal equilibrium is established when temperature and voltage readings are steady and after that thermal conductivity was determined according to Eq. (2).

$$\lambda = \frac{\Phi d}{A(T_1 - T_2)} \tag{2}$$

In this equation, λ is the thermal conductivity, Φ is the power, d is the thickness of the specimen, A is the specimen area and T_1 and T_2 are the surface temperatures of the specimens.

3. RESULTS

The compressive strength of HSC series varied between 111-118 MPa. HSC3 series possessed the maximum compressive strength of 118 MPa while HSC2 series possessed the minimum (111 MPa). Compressive strength of normal-weight concrete series varied between

30.9-38.4 MPa. NC1series possessed the minimum and NC2 containing silica fume possessed the maximum compressive strength of 38.4 MPa. It can be seen from Table 6 that addition resulted in an increase in compressive strength. Compressive strength of light-weight concrete series varied between 13-28.5 MPa.The compressive strength increased with an increase in cement dosage but inversely affected by air entraining admixture addition.

The TCC values of all concrete mixes are presented in Table 6. The TCC of HSC series ranged between 1.85 and 2.62 W/mºK. HSC2, HSC3, HSC4 series exhibited lower TCC values compared to HSC1 series owing to the lower TCC of PP (0.036-0.086W/mºK). As it can be seen from Table 6, TCC of normal-weight concretes varied between 1.89 and 2.02 W/mºK and addition of different type of mineral additives did not cause considerable change. TCC of lightweight concretes varied between 0.42 and 0.52 W/mºK. Addition of air entraining agent created additional pores in concrete and this phenomenon caused decrease in TCC. In LC2 and LC4, decrement was about 6% and 17%, respectively. Saygılı and Baykal (2011) reported that the decrease in the thermal conductivity is due to the increase of void ratio that decreased the unit weight of concrete [16]. Since air is the poorest conductor compared to the solid and liquid due to its molecular structure it leads to a lower TCC in concrete [17].

Table 6 shows the VDRF of the high strength concretes where the values ranged between 86 and 132. The highest and lowest values belong to the HSC1 mix and the HSC4 mix, respectively. The increase in the PP fiber amount caused a decrease in the VDRF of the concretes. In HSC4 series decrease was 35% compared to reference series. It was found that, VDRF values of NCs varied between 31 and 51. The reason of increase in VDRF in mineral additive series could be attributed to the denser microstructure and lower porosity. The VDRF results for the NC are below than the values given in TS 825. Today, mineral additives are often used in production of concrete. These additives react with the calcium hydroxide produced by cement hydration and form additional C-S-H. Thus, porosity both in cement paste and in interface between cement and aggregate decreases [18]. Previous studies indicated that VDRF increases by decrease in porosity and increase in density [11, 12]. According to test results, VDRF results of LCs ranged between10-15 (Table 6). Since addition of air entraining causes additional pores in concrete, VDRF decreased occasionally; it was especially remarkable in LC4 series (31%).

Table 6: Experimental test results for concrete series

Concrete Type	D (kg/m³)	f_c (MPa)	λ (W/mK)	μ
LC	1189-1468	13.0-26.0	0.42-0.52	10-15
NC	2488-2522	30.9-38.4	1.89-2.02	31-51
HSC	2520-2540	114.0-118.0	2.09-2.62	86-132

3.1 Comparison of thermo-physical properties

Considering the test results, it can be deduced that compressive strength, TCC and VDRF results are related with unit weight of concrete. LC mixes yielded the lowest compressive strength, TCC and VDRF values, on the other hand the maximum compressive strength, TCC and VDRF results were obtained in HSC.

As seen in Figure 1 and 2, the compressive strength of LC series ranged between 13.0 and 28.5 MPa. On the other hand, TCC and VDRF values ranged between 0.42 and 0.52 and 10-16, respectively and they did not show a distinct variation.

Despite the slight differences in compressive strength of NC series (30.9-38.4 MPa), VDRF results changed substantially (31-51). Here, it is possible to observe the mineral additives' effect. However, this effect was not valid for TCC results.

TCC and VDRF values of HSC series having similar compressive strengths were between 2.09-2.62 W/mK and 86-132, respectively. Differences in VDRF results might be attributed to the increase in PP fiber amount.

The condensation analysis performed on the same structural member according to TS 825 showed different results for different concrete properties.

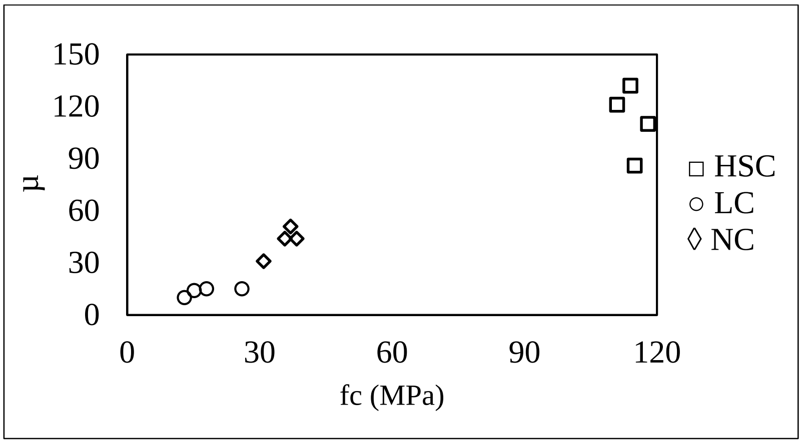


Figure 1: Comparison of compressive strength-vapor diffusion resistance factor relationship of series

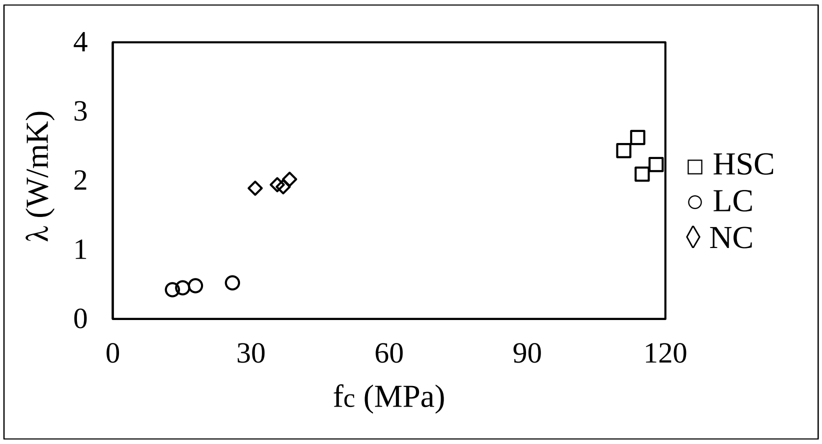


Figure 2: Comparison of compressive strength-thermal conductivity coefficient relationship of series

4. CONCLUSIONS

Based on the results obtained in this study, the following conclusions can be summarized.

- Light-weight concrete showed the minimum results in terms of unit weight, compressive strength, thermal conductivity and vapour diffusion resistance factor while high strength concrete showed the maximum results.
- Normal-weight concrete's VDRF values ranged between 31 and 51. These values are lower than those stated in TS 825. In addition, TS 825 assigned no specific values for thermo-physical properties of HSC.
- Thermo-physical properties are affected not only by compressive strength and unit weight but also mineral additive and aggregate type and polymer fiber addition. This situation should be considered in related standards and in building physics problems' analysis.

REFERENCES

[1] Inventory o f U.S. Greenhouse Gas Emissions and Sinks: 1990 – 2009, April 15, 2011.

[2] Pérez-García, A., Víllora, A.G. Pérez andG.G., 'Building's eco-efficiency improvements based on reinforced concrete multilayer structural panels', Energy and Buildings, 85, (2014) 1–11.

[3] Hall, M.R., Najim, K.B. and Hopfe, C.J.,''Transient thermal behaviour of crumb rubber-modified concrete and implications for thermal response and energy efficiency in buildings'', Applied Thermal Engineering, 33–34, (2012) 77–85.

[4] Drochytka, R., Zach, J., Korjenic, A. andHroudová, J., 'Improving the energy efficiency in buildings while reducing the waste using autoclaved aerated concrete made from power industry waste', Energy and Buildings,58, (2013) 319–323.

[5] Jerman, M., Keppert, M., Výborný, J. andČerný, R.,'Hygric, thermal and durability properties of autoclaved aerated concrete', Construction and Building Materials, 41, (2013) 352–359.

[6] Bazant Z.P. and Kaplan M.F., '*Concrete at High Temperatures'*, Longman Group, 1. Edition, (1996) London.

[7] TS EN 206-1

[8] Newman, J. and Choo, B.S. *Advanced Concrete Technology-Processes*, (2003),ISBN: 978-0-7506-5105-9.

[9] ACI 363R-92, Report on High Strength Concrete, American Concrete Institute, 55 pages, (1997).

[10] TS825Binalarda Isı Yalıtım Kuralları, (2013).

[11] Kearsley, E.P and Wainwright, P.J., 'Porosity and permeability of foamed concrete', Cement and Concrete Research, 31(5), (2001) 805–812.

[12] Kabay, N.,'Hafif Agregali Betonun Boşluk Yapisinin Mekanik ve Fiziksel Özelliklere Etkisi', in Turkish, Doctorate Thesis, Yıldız Technical University, (2009), Istanbul.

[13] BS EN 12390-3: Testing hardened concrete, Compressive strength of test specimens,(2009).

[14] TS EN 12086, Thermal insulating products for building applications - Determination of water vapour transmission properties, (2013).

[15] TS ISO 8302 Thermal insulation; determination of steady-state thermal resistance and related properties; guarded hot plate apparatus, (2002).

[16] Saygılı, A., Baykal. G., 'A new method for improving the thermal insulation properties of fly ash', *Energy and Buildings,* 43(11), (2011) 3236–3242.

[17] Ng, S.C. and Low, K.S., 'Thermal conductivity of newspaper sandwiched aerated lightweight concrete panel, *Energy and Buildings*, 42(12), (2010) 2452–2456.

[18] Mehta P.K and Monteiro, P.J.M., *Concrete: Microstructure, Properties, and Materials,* McGraw-Hill, third edition, (2006).

EFFECT OF HEAT INSULATION ON TEMPERATURE DEVELOPMENT OF MASS CONCRETE

İsmail Akkoyun (1), Hilmi Aytaç (1), Hüseyin L. Sevin (1), Can Bilal (2) and Yılmaz Akkaya (2)

(1) Bursa Beton (Ready-Mixed Concrete Company), Turkey

(2) Istanbul Technical University, Civil Engineering Department, Turkey

Abstract

Mass concrete requires a different approach in design, production, transporting, placing and curing compared to the traditional concrete. In recent years, due to increasing of investments on such large scale structures, investors and engineers require extra precautions for handling of mass concrete.

In general, it is required to keep the temperature differences between the core and the surfaces as low as possible in mass concrete to prevent thermal cracks. QA/QC activities focus on not only to achieve the design compressive strength class, but rather more importantly, to keep the fresh concrete temperature and heat development at lower levels. Thus, besides the properties of constituent materials and mixture design, concrete production operations and curing are also of great importance for the control of in-situ temperature development and differentials.

In this study, concrete mixtures were prepared by using ordinary Portland cement, ground granulated blast furnace slag and fly ash in the binder phase. 14 cubic structures (with 150 cm edge) were built up with different insulation conditions, and internal and surface temperatures were measured. Fresh and hardened concrete properties were also tested. It was found that mineral admixture type, content and initial concrete temperature have great influence on the maximum core temperature. Additionally, insulation property is one of the most important factors to lower the temperature differential within the structure.

Key words: Mass concrete, cement content, heat development, mineral admixture, thermal crack

1. INTRODUCTION

Thermal cracks occur especially when concrete is of young in age and when the volumetric instability is at its highest. With the increase in thermal strains and autogeneous shrinkage strains, the stresses develop above the tensile strength capacity of concrete, and thus, cracks occur, depending on restraining conditions. These cracks, in turn, widen due to external loads, and increase the risk of permeability against aggressive ions and humidity. It may be possible to prevent damages arising from freeze-thaw action, chloride and sulphate attacks, if early age cracks can be kept under control [1].

Maximum temperature, achieved at the core of the structure, is one of the most important factors that cause temperature differentials and thermal cracking. Fresh concrete temperature and hydration heat are important factors which affect the time and degree of maximum temperature. TS 13515 states that for mass concrete structures, fresh concrete temperature should be kept in between 10^{o}C - 30^{o}C, where the limit is stretched up to 35^{o}C for conventional structures [2]. Many specifications limit the maximum fresh concrete temperature with 20^{o}C. Cooling of concrete constituents, using mineral admixtures and adjusting production time of concrete are among the methods to keep fresh concrete temperature under control.

Heat development during cement hydration depends mostly on mixture design parameters. Keeping low cement content, using high content of mineral admixtures or low-heat cement is advisable most of the time. Post cooling is also an important application which helps control over the heat development. TS 13515 limits the max. temperature in the structure with 65^{o}C.

Much research can be found on how to control the fresh concrete temperature and/or maximum temperature at the core of the structure. However, literature on controlling the temperature differential is limited. This research investigates the effects of protection and curing activities during post production on the temperature differentials. By focusing on these activities, it was possible to train and increase the experience of all related parties involved in construction. Concrete mixtures were prepared by using ordinary Portland cement, ground granulated blast furnace slag and fly ash in the binder phase. 14 cubic structures (with 150 cm edge) were built up with different insulation properties, and internal and surface temperatures were measured. Fresh and hardened concrete properties were also tested.

2. EXPERIMENTAL STUDY

2.1 Materials

Chemical and physical properties of the fly ash (FA) and ground granulated blast furnace slag (GGBFS) are given in Table 1.

Table 1: Chemical and physical properties of FA and GGBFS

	SiO_2 (%)	Al_2O_3 (%)	Fe_2O_3 (%)	CaO (%)	SO_3 (%)	L.O.I (%)	Free CaO (%)	Spec. Grav.	React. CaO (%)	45µ residue (%)	28 Day Act. (%)
FA	55.97	19.44	10.49	3.54	0.38	1.52	0.06	2.32	2.23	25.0	79.7
GGBFS	41.91	10.96	0.59	36.91	0.29	0.00	-	2.89	-	0.8	84.5

CEM I 42,5 R was used and its properties are presented in Table 2.

Table 2: Chemical and physical properties of cements

		Phase 1*	Phase 2*	Phase 3*
SiO_2 (%)		18.45	18.78	18.64
Al_2O_3 (%)		5.63	5.51	5.44
Fe_2O_3 (%)		3.01	3.16	2.95
CaO (%)		63.30	63.28	63.28
SO_3 (%)		2.96	2.63	2.60
Na_2O (%)		0.32	0.28	0.31
K_2O (%)		0.63	0.52	0.61
Loss on Ignition (%)		3.40	3.66	3.87
45 μ residue (%)		9.0	8.4	7.8
Blaine (cm^2/g)		3420	3650	3820
Compressive Strength (MPa)	2 days	24.0	29.3	25.2
	7 days	47.0	54.1	45.4
	28 days	58.3	61.5	53.0

*Refer to 2.3

Crushed lime stones obtained from a single source were used. Phosphonated polycarboxylate based super plasticizer admixture (confirms to TS EN 934-2) was used in the concrete production.

2.2 Mixture designs

All mixtures contained 400 kg/m^3 of cementitious material in the binder phase. The activity index of fly ash and GGBFS were taken as 0.4 and 0.8, respectively, for calculation of water-to-binder ratio. Mixture proportions are presented in Table 3.

Table 3: Mixture proportions (kg/m^3)

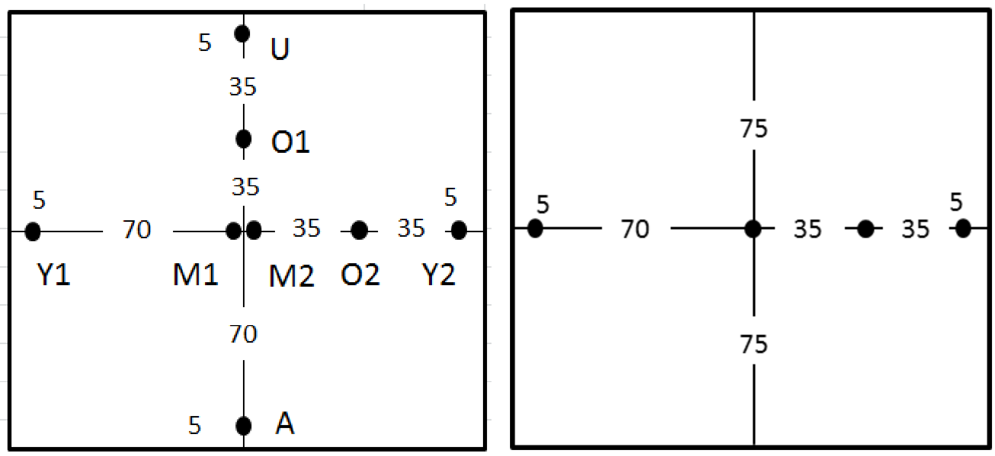
Code	**Cement**	**Fly Ash**	**GGBFS**	**Super Plast.**	**Water**	**w/b**	**Sand**	**5/12**	**12/22**
C100-GBFS300-40	100	0	300	3.60	136	0,40	1135	324	440
C100-GBFS300-45	100	0	300	2.40	153	0,45	1100	316	428
C150-GBFS250-40	150	0	250	4.40	140	0,40	1131	323	438
C150-GBFS250-45	150	0	250	2.80	158	0,45	1101	314	427
C300-FA100-45	300	100	0	5.20	153	0,45	1089	312	424
C200-FA200-50	200	200	0	5.20	140	0,50	1081	306	414
C150-FA250-50	150	250	0	7.20	125	0,50	1087	312	423

Generally high cementitious content and low water-to-binder ratio are preferred for durability of mass concrete. Considering the workability, strength and durability requirements, a cementitious content of 400 kg/m^3 was selected. Considering TS EN 206 and TS 13515 maximum water-to-binder ratios of 0,40 - 0.45 were selected. However, due to low activity index of fly ash, and resulting high viscosity concrete, water-to-binder ratio was selected as 0,50.

2.3 Research method

In order for effective use of data loggers, form-work and space, the experimental work was completed in 3 phases. 14 cubic structures of 1,5x1,5x1,5 m. with different insulation were cast. Due to mobilation issues and temperature measurement period, each phase was started 2 weeks after the preceeding phase. Seperate silos were reserved for fly ash and GGBFS in the plant. Aggregates were used from the same stock, reserved and kept under protection until the castings were finished. Cement used in the daily production was used during casting of cubic structures. Although sampling was made from new cement shipment before each production (Table 1), it was not possible to separate the new shipment from the existing cement in the silo. All castings were performed on Sundays, so that the regular production activities could continue during other working days without interruption. The plant was reserved for testing activities during all day. Prior to each production phase, all bunkers were emptied, cleaned and aggregates from the special stock were loaded. Superplasticizer was used from the IBC tank for all phases. Slump and temperature of the fresh concrete were recorded. Compressive and tensile strength development, and heat development of the hardening concrete were measured.

For each phase, several meters of thermocouples and 11 data loggers with 4 channels were used. The temperature was measured at 5, 40 and 75 cm (core) away from the edges in 8 locations within the cubic structure. The measurement locations are presented in Figure 1.



Side view Top view

Figure 1 : Temperature measurement locations (distances are given in cm.)

Temperature increase, due to heat of hydration of the concrete blocks, were measured in 6-hour periods by thermocouples. Air temperature was measured at 4 different locations, and average was used in the analysis. Temperature development at the surfaces (top, bottom and sides), at the core, and at a location between them (O1, O2) were measured. Analysis, with graphs, was only made on selected locations, due to high correlation among similar measurement points. Attention was paid on comparison of measurement locations with different thermal insulation.

3. RESULTS AND DISCUSSION

Fresh and hardened concrete properties are presented in Table 4 and Table 5. The flow value was measured and recorded when it was 40 cm or above. Slump was measured at the plant (t=0 min), before pumping (t=30 min) and after pumping (t=60 min). Increase in the workability of concrete in time can be attributed to the superplasticizer properties. Mixtures with fly ash presented high viscosity and pumping difficulties.

6 cubes of 150 mm edge length were sampled to get compressive and splitting tensile strength (Figure 2). All concrete mixtures provided the C40/50 compressive strength class.

Table 4: Fresh concrete properties

Phase No	Cast. No	Code	Air (ºC)	Concrete (ºC)	Slump/Flow (cm/cm)		
					t=0 min	t=30 min	t= 60 min
1	1	C100-GBFS300-40	27	31	22/40	22/40	24/45
	2	C100-GBFS300-45	30	32	22/-	23/40	23/40
	3	C150-GBFS250-40	33	32	24/45	24/45	25/60
	4	C150-GBFS250-45	30	33	22/-	22/-	25/50
2	5	C100-GBFS300-40	18	23	20/-	23/40	24/45
	6	C100-GBFS300-40	17	23	15/-	22/-	23/45
	7	C100-GBFS300-40	16	23	19/-	24/45	25/50
	8	C300-FA100-45	16	25	23/-	23/-	25/45
	9	C150-FA250-50	16	24	24/45	25/60	25/65
3	10	C150-GBFS250-40	13	20	16/-	20/-	22/-
	11	C100-GBFS300-40	13	21	10/-	19/-	22/-
	12	C300-FA100-45	13	21	14/-	22/-	23/-
	13	C200-FA200-50	13	22	19/-	23/40	25/50
	14	C100-GBFS300-40	13	21	10/-	20/-	22/40

Figure 2: Sampling

Table 5: Compressive and splitting tensile strength development

Phase No	Casting No	Design Code	Compressive Strength (MPa)				Splitting Tensile Strength (MPa)			
			24 h.	28 day	56 day	91 day	24 h.	28 day	56 day	91 day
1	1	C100-GBFS300-40	7.6	55.5	65.0	66.3	0.5	4.6	4.8	4.6
	2	C100-GBFS300-45	5.4	49.0	52.7	54.0	0.4	4.2	4.5	4.9
	3	C150-GBFS250-40	8.4	64.6	67.9	72.8	0.5	4.7	5.5	5.7
	4	C150-GBFS250-45	7.4	55.5	60.4	65.4	0.6	4.5	5.1	5.3
2	5	C100-GBFS300-40	3.0	51.5	60.1	64.5	0.2	4.1	5.0	5.0
	6	C100-GBFS300-40	3.5	53.8	61.6	63.5	0.2	4.1	4.9	4.9
	7	C100-GBFS300-40	- [1]	- [1]	- [1]	- [1]	- [1]	- [1]	- [1]	- [1]
	8	C300-FA100-45	11.2	57.0	60.0	65.5	1.1	4.0	5.1	5.2
	9	C150-FA250-50	0.0	51.9	59.0	62.4	0.0	4.2	4.8	4.8
3	10	C150-GBFS250-40	6.2	67.6	75.0	79.1	0.4	5.1	4.7	4.5
	11	C100-GBFS300-40	3.5	56.4	64.5	65.2	0.2	4.6	5.2	4.4
	12	C300-FA100-45	11.3	50.6	60.6	64.0	1.0	4.6	4.3	4.4
	13	C200-FA200-50	4.6	50.8	59.8	66.8	0.1	4.5	4.3	3.9
	14	C100-GBFS300-40	- [1]	- [1]	- [1]	- [1]	- [1]	- [1]	- [1]	- [1]

(1) Sampling for this mixture was made previously.

Temperature developments are stated in Table 6.

Phase 1 : Results of Mixtures 1 and 2 indicate that regardless of w/b ratio, the max. temperature in the structrue is the same when the fresh concrete temperatures, cement contents and insulation properties are similar. Since all faces of the structures were insulated, the temperature differentials were minimal. Same observations can be made for Mixtures 3 and 4.

All Mixtures 1, 2, 3 and 4 presented a very slow cooling period due to effective insulation. The cooling rates of these mixtures are in between 0.05 ^{o}C/h - 0.07 ^{o}C/h.

Phase 2 : When the mixtures No 5 and No 1 are compared, it can be concluded that decrease of fresh concrete temperature can reduce the maximum temperature up to 20^{o}C. Another reason for this decrease is that mixture No.5 was totally uninsulated.

When mixtures No 6 and No 7 are compared, it can be observed that the temperature recordings are very similar, therefore 10 cm EPS and 3 cm XPS insulations present the same performance.

When mixtures No 5, 6, 7 are compared; it can be seen that maximum temperature differentials are similar. This indicates that in order to keep the maximum temperature differential at low values, the insulation should be made all over the structural members.

The maximum difference between concrete cover depth and air temperature occured in mix No 5 (Y1-AIR). This is a result of the insulation effect of formwork. For mixtures No 6-10 the maximum difference between concrete cover depth and air temperature occured between U-AIR, since the sides were insulated.

Table 6: Temperature developments

Phase No	Casting No	Design Code	Insulation Properties					Fresh Con. Temp. (C)	Max. Temp. at core (C°)	Max. Temp. Difference °C			
										Core - Cover		Cover - Air	
			Bottom	Top	Sides	Thickness (cm)	Type			M1-U	M1-A	U-AIR	Y1-AIR
1	1	C100-GBFS300-40	*Yes*	*Yes*	*Yes*	10	EPS	31	57.2	3.2	-	-	-
	2	C100-GBFS300-45	*Yes*	*Yes*	*Yes*	10	EPS	32	57.3	3.3	-	-	-
	3	C150-GBFS250-40	*Yes*	*Yes*	*Yes*	10	EPS	32	65.4	4.5	-	-	-
	4	C150-GBFS250-45	*Yes*	*Yes*	*Yes*	10	EPS	33	67.1	4.1	-	-	-
2	5	C100-GBFS300-40	No	No*	No	-	-	23	37.2	12.6	-	-	20.7
	6	C100-GBFS300-40	No	No*	*Yes*	3	XPS	23	40.5	14.4	-	16.9	-
	7	C100-GBFS300-40	No	No*	*Yes*	10	EPS	23	41.8	14.5	-	18.0	-
	8	C300-FA100-45	No	No*	*Yes*	3	XPS	25	59.2	22.7	-	28.9	-
	9	C150-FA250-50	No	No*	*Yes*	3	XPS	24	46.3	15.1	-	21.5	-
3	10	C150-GBFS250-40	No	No*	*Yes*	10	EPS	20	46.2	15.7	-	18.7	-
	11	C100-GBFS300-40	*Yes*	*Yes*	*Yes*	10	EPS	21	46.7	3.4	-	-	-
	12	C300-FA100-45	*Yes*	*Yes*	*Yes*	10	EPS	21	66.3	8.2	-	-	-
	13	C200-FA200-50	No	*Yes*	*Yes*	3	XPS	22	53.3	4.6	14	-	-
	14	C100-GBFS300-40	No	*Yes*	*Yes*	3	XPS	21	43.4	4.5	10	-	-

*Top of these concretes were covered with plastic sheet to prevent evaporation

For mix No 5 internal temperature difference is measured 12.6°C (M1-U), where the temperature difference between (Y1-AIR) is 20.7 °C. This indicates that when there is no insulation the cracking risk increases due to temperature difference between concrete cover depth and air.

Although mix No 8 has the highest cement content (150 kg/m^3 higher than No.3 and 4), the maximum temperature did not rise significantly compared to mixes with high fresh concrete temperature, No 3 and 4. It is also important to note that mixture No 8 did not have the insulation at the top and bottom. For this case temperature difference between concrete cover and air is the highest (28,9 °C). This implies that conventional concrete mixtures have potentially higher cracking risk if they are not insulated.

Mixtures No 6 and 8 have similar insulations and fresh concrete temperatures. However, an increase in the cement content of 200kg/m3 increase the maximum temperature up to 20°C (40.5°C - 59.2°C)

Phase 3: For mix No 10 due to low fresh concrete temperature and no insulation at the top and bottom surfaces, maximum temperature decreased up to 19°C compared to mix No 3. Although the maximum temperature was lower, temperature differentials were higher. This indicates the importance the insulation at the top surface.

When mixes No 1 and 11 are compared the effect of fresh concrete temperature on maximum temperature can be observed. When fresh concrete temperature is 10°C lower, the maximum concrete temperature is also 10°C lower. It is observed that temperature differentials depend on insulation properties, rather than fresh concrete temperature.

When mixtures No 12 and 8 are compared it is seen that an effective insulation can reduce the temperature loss at the core and therefore increase the maximum temperature up to 7°C. Although the maximum temperature of mix No 12 higher than No.8 the temperature differential is lower due to effective insulation.

When mix No 11 and 12 are compared, it is observed that higher cement content increase the maximum temperature up to 20°C, when fresh concrete temperature and insulation were the same. The cooling period of these mixtures were long, 0.05 °C/hr and 0.08 °C/hr, respectively.

When mix No 7 and 14 are compared, maximum temperatures are similar although the insulation properties are different, fresh concrete temperatures are similar. When the top surface is insulated the temperature difference decreased up to 10°C (from 14.5 °C to 4.5 °C).

4. CONCLUSIONS

Generally the top surface of the mass foundation concretes are not insulated and the sides are insulated with low heat conductivity materials. The curing sheets applied at the top surface, to prevent evaporation, are in effective in terms of heat insulation. On the other hand in many projects only the maximum temperature is limited and no precautions are taken to reduce temperature differences, even at the side surfaces. This study presented the importance of heat insulation to limit the temperature differences.

- Generally for mass concrete the maximum temperatures are achieved at 48 hours, depending on structural dimensions, concrete constituents and fresh concrete temperatures.
- The occurrence time of maximum temperature difference may not be the same as maximum temperature.
- Maximum temperature depends on cement contents and fresh concrete temperature, rather than insulation properties.
- Heat insulation provides a longer cooling period and lower temperature differences. Insulation on all surfaces of the structure is very effective in lowering temperature differences and keeping constant cooling rates. If no insulation is performed maximum temperature may decrease.
- For temperature differences in concrete, the ground temperature should also be taken into account as much as side and top surfaces.
- The top surfaces can be insulated EPS, XPS or any other material with low heat conductivity. By the way, 10 cm EPS and 3 cm XPS insulations present the same performance.
- With cement contents of 100-150 kg/m^3 in slag concrete, it is possible to keep the maximum temperatures below 65°C and achieve C40/50 in 28 days and C50/60 in 56 days.
- Due to low activity index of fly-ash, the total water volume in the concrete is decreased and the viscosity of fresh concrete is increased considerably. Therefore a water to cement ratio of above 0,50 is needed. With 200kg/m^3 of fly-ash, with a fresh concrete

temperature of 20^{o}C, heat development can be controlled in mass concrete provided that insulation is applied on the structure. It is possible to achieve C40/50 in 28 days and C50/60 in 56 days.

REFERENCES

[1] Akkaya, Y., ‘Sertleşmekte olan beton özelliklerinin çatlak riski açısından değerlendirilmesi’, Beton 2013 Proceedings, THBB Publications, (2013), 135-143.

[2] TS 13515, ‘TS EN 206’nın Uygulamasına Yönelik Tamamlayıcı Standard’, 2014.

EFFECTS OF MACRO-SYNTHETIC FIBERS ON MECHANICAL BEHAVIOR AND PERMEABILITY PROPERTIES OF CONCRETE

Mertcan Sarı (1), Cengiz Şengül (1), Yılmaz Akkaya (1), Fatih Özalp (1,2), Ömer Kaya (1,2) and Mehmet Ali Taşdemir (1)

(1) Istanbul Technical University, Civil Engineering Faculty, Istanbul

(2) Iston, Istanbul Concrete Elements and Ready Mixed Concrete Factories, Turkey

Abstract

The main objective of this research is to investigate the mechanical behavior, fracture and permeability properties of concretes containing steel fibers, basalt fibers and two types of macro-synthetic fibers and concrete without fibers. Concrete mixes have the same water-cement ratio and same fiber volume fraction except one of the steel fiber volume fraction which is approximately half of other volume fractions. Surface permeability of all concretes and rapid chloride permeability of normal concrete and concretes with macro-synthetic fibers were investigated and the results obtained were compared. Concretes which include different types of fibers were compared with each other after determining the compressive strength, modulus of elasticity, splitting tensile strength, fracture energy and bending strength. Based on the test results, following conclusions can be drawn: Fracture energy of plain concrete increased up to 10 and 4 times owing to the addition of steel and macro-synthetic fibers, respectively. Steel fiber reinforced concretes (SFRCs) presented enhanced toughness and ductility when compared to the plain matrix; however, the net bending, compressive strength and elastic modulus of the concretes were not significantly affected by the addition of fibers. The residual strengths of concretes containing macro-synthetic fibers decreased dramatically compared to those of SFRCs.

1. INTRODUCTION

Macro-synthetic and steel fibers are commonly used in the production of prefabricated reinforced concrete production, shotcrete applications, and industrial floors. Moreover, these fibers are added to high strength/high performance concretes in order to transform them to more ductile materials. In this study, mechanical behavior, fracture and some permeability properties of plain concrete, macro-synthetic and steel fiber reinforced concretes were investigated. These fibers function as a bridge to retard the formation of cracks and their propagation. Hence, fiber reinforced concretes absorb much more energy at failure and guarantee structural integrity as already pointed out by a number of studies [1-11].

2. EXPERIMENTAL WORK

2.1 Materials and mix proportions

Portland cement CEM I 42.5 R was used as binder in this study. The specific gravity, compressive strength and the specific surface area of the cement were noted to be 3.15 g/cm^3, 58.9 MPa and 3720 cm^2/g, respectively. Natural sand, two types of crushed limestone coarse aggregates, having 19 mm maximum size, as well as limestone fines were used. Superplasticizer was ligno sulfonate based high range water reducing admixture. Steel, basalt and two types of macro-synthetic fibers were used in the study. Properties of the fibers are given in Table 1.

Table 1**:** Physical and mechanical properties of fibers.

Type of fiber	Length (mm)	Diameter (mm)	Tensile strength (MPa)	Young's modulus (MPa)	Density (kg/m^3)
Steel	60	0.75	1050	210000	7800
Basalt	50	0.015	4150-4800	98100	1160
Polyolefin	50	0.5	618	10000	910
PP-PE* copolymer	54	0.667	550-750	5750	910

* PolyPropylen- PolyEthylene.

Concrete mixes have the same water-cement ratio of 0.6 and the same fiber volume fraction (V_f) of 0.445%, except one of the steel fiber volume fraction, which is approximately half of other fiber volume fractions. Notation and definition of mixes are shown in the Figure 1. In the notation, NC represents normal concrete; CB basalt, CCP PolyPropylene-PolyEthylene copolymer, CPO polyolefin and CSF steel fiber reinforced concretes, respectively. The last digits indicate the weight of the fiber used in the mixture. The curing regime was the same and specimens were kept in water at 20 °C for 28 days before testing.

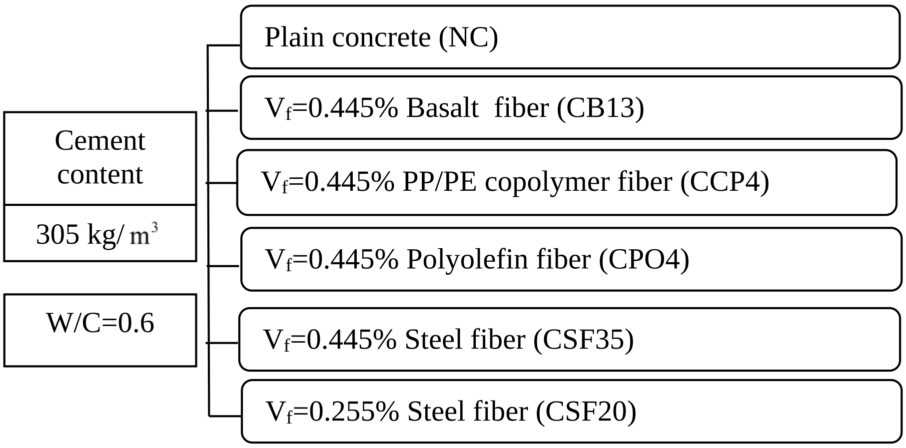


Figure 1: Notation and definition of concrete mixes.

2.2 Test procedure

In order to investigate the mechanical behavior, fracture and permeability properties of concretes, various tests were performed. Compression tests were conducted on Ø150/300 mm cylindrical specimens with displacement controlled test machine. The modulus of elasticity was also calculated through the slope of stress-strain curve between 5 and 45% of the maximum stress.

The tests for determining fracture energy (G_f) were performed in accordance with the recommendation of RILEM 50-FMC Technical Committee [12]. The deflection was measured using a linear variable displacement transducer (LVDT). The load was applied to the beam using a closed-loop testing machine (Instron 5500R 100 kN maximum capacity). The beams prepared for the fracture energy tests were 600 mm in length and 150x150 mm in cross section. The depth of the notch of specimens was 25 mm and the length of support span was 550 mm. The load (P)-crack mouth opening displacements (CMOD) and P-displacement at the mid span curves were determined, simultaneously. The fracture energy was calculated by using the following formula given by RILEM 50-FMC Technical Committee [12]:

$$G_f=(W_0+mg\delta_0)/A_{lig} \quad (1)$$

where W_0, m, g, δ_0, and A_{lig} are area under load-deflection curve, weight of the specimen between supports, acceleration of gravity, deflection of the specimen at failure for normal concrete and at the deflection specified for fibered concretes (i.e. 3,15 mm) and effective cross-section, respectively.

Bending strength of specimens was calculated with the equation given below.

$$F_{net}=(3PL)/BD^2 \quad (2)$$

Where P, L, B, D are the ultimate load, distance between supports, width of cross-section and depth of cross-section.

In order to define surface permeability, measurements are carried out at the surface by clamping a stainless steel chamber on the smooth surface of the concrete specimen. A measurement of the time required for related amounts of air to permeate through the concrete was used as an index of the surface conditions in the Figg method [13]. The classification of protective quality of cementitious materials is given in the following table.

Table 2: Surface air permability classes [13].

Quality category	Interpretation	Time for pressure change (s)	Type of material
0	poor	≤30	Porous mortar
1	moderate	30-100	20 MPa concrete
2	fair	100-300	30-50 MPa concrete
3	good	300-1000	densified, well-cured concrete
4	excellent	≥1000	polymer-modified concrete

Rapid chloride permeability tests were performed in normal concrete and concretes with macro-synthetic fibers in accordance with ASTM C 1202-97 [14] on concrete disc specimens with 100 mm diameter and 50 mm thick.

3. RESULTS AND DISCUSSION

Cylinder compressive strength values are given in Figure 2. Basalt and 20 kg/m³ steel fibers added concretes had less compressive strength than that of plain concrete. The maximum strength values were obtained in 35 kg/m³ SFRC concretes. PP-PE copolymer and polyolefin fiber reinforced concretes showed nearly the same strength performance of 36.8 and 37.1 MPa respectively. As seen in Figure 2 that the addition of fiber caused 10% change in the compressive strength of plain concrete.

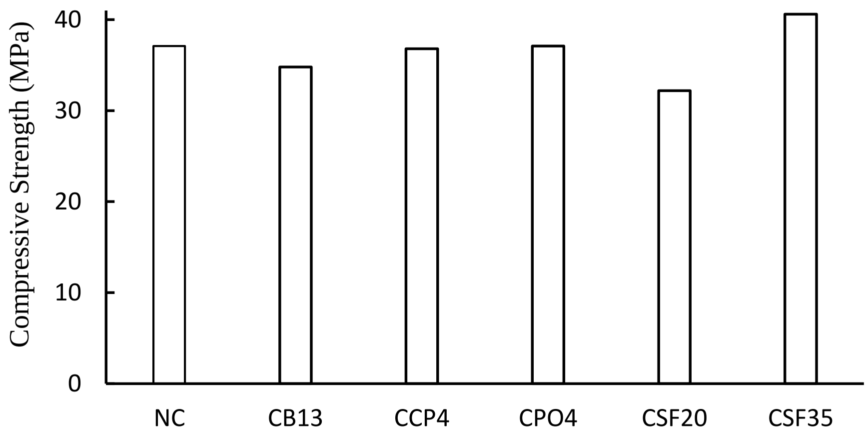


Figure 2: Compressive strengths of mixes.

In the modulus of elasticity test results, concrete containing 35 kg/m^3 steel fiber had the highest value among all mixes. The copolymer fiber added concretes and normal one had close values with polyolefin added concretes. Addition of basalt fibers to the plain concrete caused some minor changes. Compressive strength and modulus of elasticity decreased with the decrease of fiber content in SFRC concretes (Figure 3).

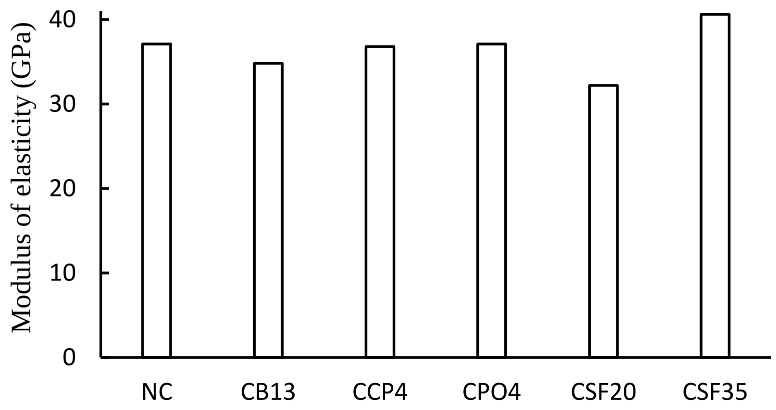


Figure 3: Elastic modulus of concrete mixes.

In the bending test results, addition of fibers except basalt fiber to the plain concrete increased the fracture energy significantly. Load versus deflection curves are shown in the following figure.

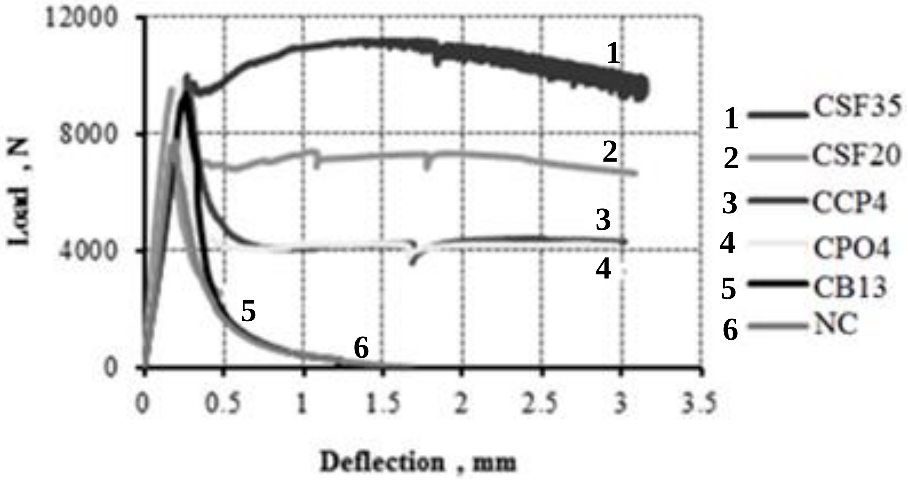


Figure 4: Load versus deflection curves of fiber reinforced and plain concretes.

Fracture energy calculated using the equation (1) is shown in the Figure 5. PP-PE copolymer fiber and polyolefin fiber reinforced concretes have almost the same fracture energy of 663 J/m² and 669 J/m², respectively. Both two groups of SFRCs had higher fracture energy than the other concretes. As the amount of steel fiber in unit volume increased, the fracture energy increased as well. The concretes with the steel fiber content of 0.445% and 0.225% had the fracture energy values of 1466 J/m² and 1112 J/m², respectively.

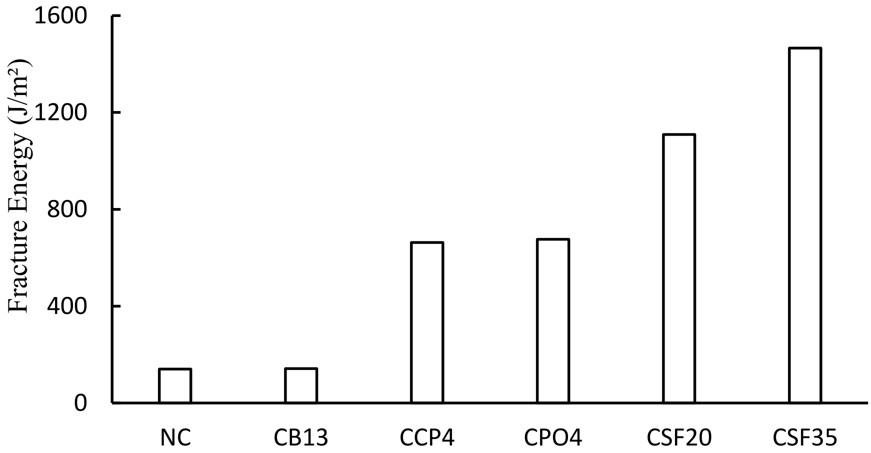


Figure 5: Fracture energy of plain and fiber reinforced and plain concretes.

As seen in the Figure 6, all types of fiber reinforced mixtures have slightly higher bending strength than plain concrete. SFRCs with the dosage of 35 kg/m^3 showed the maximum flexural strength. The bending strength of SFRCs increased with the increasing dosage of steel fibers. Polyolefin fiber reinforced concretes had higher strength (i.e. 3,58 MPa) than other macro-synthetic fiber reinforced concretes. Basalt and copolymer added concretes had nearly the same strength values of 3.33 MPa and 3.30 MPa, respectively.

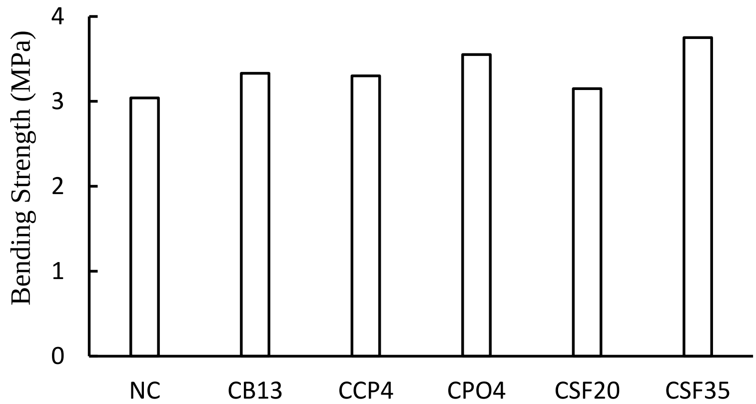


Figure 6: Bending strength of fiber reinforced and plain concretes.

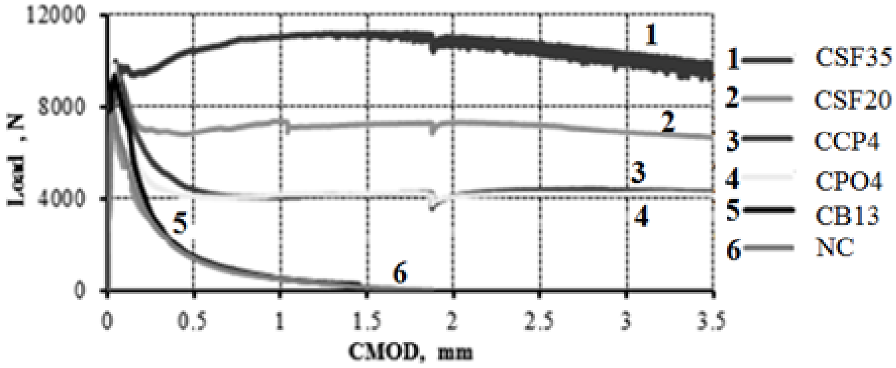


Figure 7: CMOD-load graphics of fiber reinforced and plain concretes.

Crack mouth opening displacements (CMOD) were also measured during bending tests. Load-CMOD curves, fracture energy and residual flexural strength values are given in Figures 7, 8 and 9, respectively. Load-CMOD curves of plain and basalt fiber reinforced concretes could not reach a CMOD of 2 mm values. SFRCs had the highest load, energy and residual flexural strength values within all fiber reinforced concretes. However, the fracture energy and residual flexural strength increased with the increasing steel fibers content. The residual strengths of concretes containing macro-synthetic fibers, obtained from the relation of load versus crack mouth opening displacement, decreased dramatically compared to those of SFRCs (Figure 9).

Air permeability test results and related classifications are given in the Table 3. Based on the results obtained, it can be concluded that concretes stayed below the third category, which is interpreted as fair and moderate, according to Table 2.

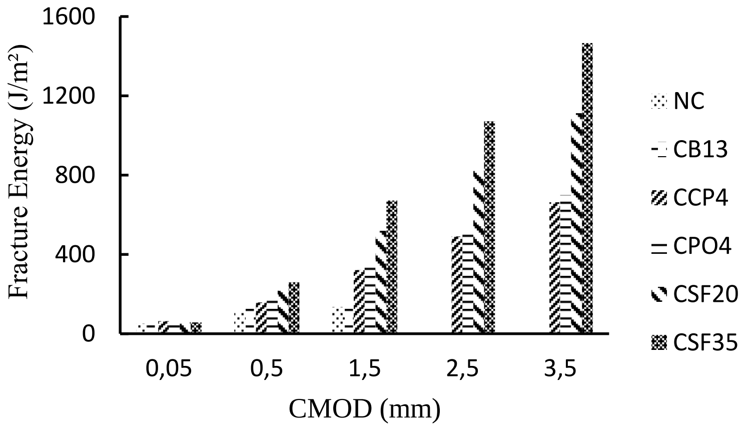


Figure 8: Fracture energy-CMOD of fiber reinforced and plain concretes.

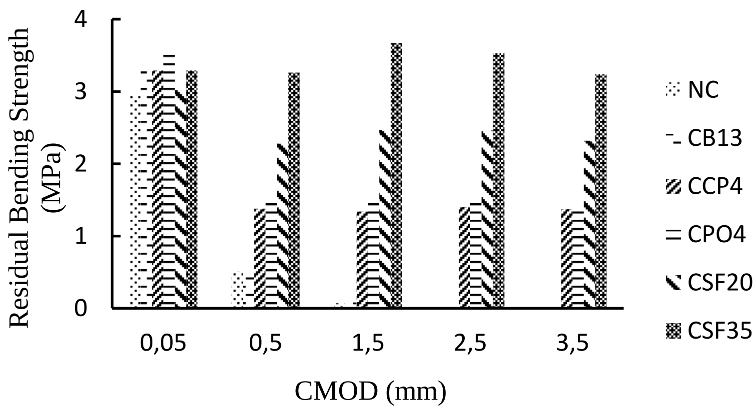


Figure 9: The residual strengths-CMOD of fiber reinforced and plain concretes.

The highest surface air permeability results were obtained from SFRCs which might be due to insufficient placeability after addition of fibers. PP-PE copolymer reinforced concretes showed lowest air permeability among all synthetic fiber reinforced concretes.

Table 3: Air permeability of fiber reinforced and plain concretes.

Mixture code	t (sec)	Interpretation
CCP4	229	Fair
CB13	168	Fair
CPO4	180	Fair
CSF35	67	Moderate
NC	150	Fair
CSF20	74	Moderate

Rapid chloride permeability test results are given in the Table 4. SFRC concretes could not been tested because of high current due to steel fibers during tests. Basalt added concretes are classified in the low permeability class as plain concretes. The addition of copolymer and polyolefin fibers increased the permeability compared to that of plain concrete. Insufficient placeability in concretes with macro-synthetic fibers except basalt fiber, may play role in high permeability.

Table 4: Rapid chloride permeability of concretes.

Mixture code	Total current passed (C)	Permeability class
CCP4	2424	moderate
CB13	1851	low
CPO4	2687	moderate
NC	1675	low

3. CONCLUSIONS

Based on the results obtained in this study, the following conclusions can be drawn:

Cylindrical compressive strength of the concrete containing steel fiber of 35 kg/m^3 is higher than the other mixtures. Compressive strengths of the normal concrete and concrete mixtures produced with PP/PE copolymer and polyolefin fibers are also high compared to the rest of the mixtures. The change in modulus of elasticity, depending on the type of fiber, has a similar trend as the compressive strength. Bending strengths of concretes produced with steel fiber of 35 kg/m^3 is higher than those of the other mixtures.

Concretes with steel fiber of 35 kg/m^3 as well as the concretes containing 20 kg/m^3 steel fiber have higher fracture energies, also higher residual bending strengths and therefore absorb more energy compared to the other mixtures.

At a specific deflection of 3.15 mm, fracture energy of concretes with macro-synthetic fibers increased up to 4.8 times. SFRCs, however, showed a behavior of enhanced toughness and ductility when compared to both plain concrete and concretes with macro-synthetic fibers. Fracture energy of SFRCs increased up to 9.5 times that of plain concrete. There was no contribution of basalt fibers into fracture energy of plain concrete; normal and basalt concretes exhibited almost the same mechanical behavior.

Rapid chloride permeability tests were conducted in all concretes except SFRCs. The RCP class of normal concrete and concrete with basalt is low, however, that of other concretes with macro-synthetic fibers is moderate.

The surface air permeability of SFRCs is higher than that of the concrete mixtures with and without macro-synthetic fibers.

ACKNOWLEDGEMENTS

This research was supported by the Scientific and Technological Research Council of Turkey under Program 1509 (project number: 9120021). Production of concretes used in this study, mechanical properties of hardened concrete, and the other physical tests such as rapid chloride and air permeability tests were realized in İSTON Quality Control Laboratories. Fracture behavior of concretes under bending was carried out in the Building Materials Laboratory of Civil Engineering Faculty at Istanbul Technical University (ITU).

REFERENCES

[1] Balaguru P, Narahar R, Patel M., 'Flexural toughness of steel fibre reinforced concrete', ACI Mater J 1992; 89(6):541–6.

[2] Richard P, Cheyrezy MH.,‘Composition of reactive powder concrete' Cem ConcrRes 1995; 25(7):1501–11.

[3] Bonneau O, Lachemi M, Dallaire E, Dugat J, Aitcin P-C., ' Mechanical properties and durability of two industrial reactive powder concretes', ACI Mater J 1997;94(4):286–90.

[4] Graybeal B., 'Material property characterization of ultra-high performance concrete', FHWA-HRT-06-103, U.S. Department of Transportation; 2006. p. 176.

[5] Bayard O, Ple O., 'Fracture mechanics of reactive powder concrete: material modeling and experimental investigations', Eng Fract Mech 2003;70(7– 8):839–51.

[6] Thomas J, Ramaswamy A., 'Mechanical properties of steel-fiber reinforced concrete', J Mater Civ Eng 2007; 19:385–92.

[7] Song PS, Hwang S., 'Mechanical properties of high-strength steel fiber reinforced concrete', Constr Build Mater 2004; 18:669–73.
[8] Sivakumar A, Santhanam M., 'Mechanical properties of high strength concrete reinforced with metallic and non-metallic fibres', Cem Concr Compos 2007; 29:603–8.
[9] Koksal F, Altun F, Yigit I, Sahin Y., 'Combined effect of silica fume and steel fiber on the mechanical properties of high strength concretes', Constr Build Mater 2008;22:1874–80.
[10] Taşdemir, M.A., İlki, A. and Yerlikaya, M., 'Mechanical behaviour of steel fibre reinforced concrete used in hydraulic structures', HYDRO 2002 International Conference of Hydropower and Dams, November 4-7 2002, 159-166
[11] Bayramov F., Taşdemir C.and Taşdemir M.A, 'Optimisation of steel fibre reinforced concretes by means of statistical response surface method', RILEM 50-FMC., 'Determination of fracture energy of mortar and concrete by means of three-point bend tests on notched beams. Mater Struct 1985; 18(4):285–90.
[12] RILEM 50-FMC, 'Determination of fracture energy of mortar and concrete by means of three-point bend tests on notched beams. Mater Struct 1985; 18(4): 285–90.
[13] Figg J.W., 'Methods of measuring the air and water permeability of concrete' Mag. Concrete Research, 25, No.85, 213-219, 1973
[14] ASTM C1202 – 12 'Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration', 2012

EFFECT OF SO_3 CONTENT IN THE MIXTURE ON DRYING SHRINKAGE OF HEAT-TREATED AND NON-HEAT-TREATED ULTRA-HIGH PERFORMANCE FIBER REINFORCED CONCRETES

Ali H. Nahhab (1,2), Mehmet Gesoğlu (1), Erhan Güneyisi (1) and Halit Yazıcı (3)

(1) Department of Civil Engineering, Gaziantep University, Gaziantep, Turkey

(2) Department of Civil Engineering, Babylon University, Babylon, Iraq

(3) Department of Civil Engineering, Dokuz Eylül University, İzmir, Turkey

Abstract

Excessive amounts of sulfate in concrete can originate from either modern cement or contaminated aggregate, which is a main aggregate in many parts of the Middle East. In this work, drying shrinkage of ultra-high performance fiber reinforced concrete (UHPFRC) with different total SO_3 contents of up to 5.77% was investigated. The water/ binder ratio and volume fraction of micro steel fibers were kept constant at 0.174 and 2%, respectively for all the mixtures. Samples for drying shrinkage were either air dried directly after demolding or steam cured for 48 h at 80°C two days after casting and then subjected to air drying. Free shrinkage strains as well as corresponding weight losses were measured over 180 days. The compressive strengths of water and steam cured UHPFRC were also determined at 28 and 180 days. The results indicated that the non-heat-treated mixes with excess sulfates showed a lower shrinkage than those with normal sulfate levels; particularly the effect of sulfates being more obvious at early ages. In general, the shrinkage of heat-treated mixes was slightly increased by the addition of sulfates. Furthermore, applying steam curing significantly decreased the drying shrinkage and weight loss of all UHPFRC mixes.

Keywords: Ultra-high performance fiber reinforced concrete (UHPFRC), drying shrinkage, weight loss, steam curing, gypsum.

1. INTRODUCTION

Advances in the science of cement-based materials have resulted in the development of a new generation of concrete, namely ultra-high performance fiber reinforced concrete (UHPFRC). As compared to ordinary concrete, the primary improvement of UHPFRC is achieved by a micro-structural engineering approach, including the removal of coarse aggregate, limiting the water-to-cementitious material ratio, introducing micro fine materials such as silica fume, and incorporation of micro steel fibers [1, 2].

Shrinkage of concrete is a complex phenomenon which depends on many factors, including the ingredients of the concrete, the temperature and relative humidity of the environment, the age at which the concrete is exposed to drying, and the size of the concrete member or structure. The shrinkage of fresh concrete is referred to as plastic shrinkage. The term of autogenous shrinkage is used to describe the shrinkage when a concrete can self-desiccate during hydration process and which becomes more significant with a higher compressive strength of concrete. The time-dependent volume change owing to the drying of concrete is referred to as drying shrinkage. This volume change of the concrete is related to the volume of water lost. The loss of free water, which occurs first, may lead to produce shrinkage. As the drying process of the concrete progresses, the adsorbed water held by hydrostatic tension in the small capillary pores is decreased significantly. When drying shrinkage is restrained, the loss of free water and adsorbed water may induce tensile stresses, which cause concrete to shrink initiating cracks that can adversely affect the structural performances, such as serviceability and durability, if not accurately taken into account during the design stage. Indeed, drying shrinkage cracking is related not only to the amount of free drying shrinkage but also to the tensile strength, modulus of elasticity, and creep of the concrete [3]. Garas et al. [4] studied a short-term free shrinkage of UHPFRC. Their results indicated that the application of thermal curing (at 90°C) decreased the 14- day drying shrinkage by more than 82%. Work by Graybeal [5] showed that the steam cured UHPFRC did not exhibit any measurable free shrinkage after thermal treatment. In contrast, the air cured samples continued to shrink with time.

The contamination of fine aggregates with sulfates, mostly with gypsum is a frequent problem in the Middle East region and similar locations [6]. In the Middle East, it is difficult to find well-graded sand with allowable sulfate content despite the availability of many quarries of natural sands. In Iraq, for example, due to the scarcity of sulfate- free aggregates, Iraqi code allows only 2.8% SO_3 in ordinary Portland cement, which is considerably lower than the limits in other specifications such as British specification (3.5%) and ASTM specification (3-3.5%). The presence of an undesirable amount of gypsum in the aggregate is potentially deleterious because it may lead to composition-induced internal sulfate attack [6-8]. Crammond [7] substituted fine aggregates by various levels of coarsely crystalline gypsum. Based on the expansion results of ordinary Portland cement mortars, it was concluded that the quantity of gypsum allowed in the aggregate would not be much larger than 2.5% by weight of fine aggregate. Atahan and Dikme [9] proved the effectiveness of different types of mineral admixtures such as silica fume, fly ash, slag, and nano silica in mitigating composition-induced internal sulfate attack on mortars containing gypsum- contaminated aggregates.

The use of gypsum- contaminated aggregate with a limited amount of sulfate in the conventional concrete suggests a possibility of utilizing it to produce UHPFRC with outstanding mechanical and durability characteristics in the Middle East like regions. As with the conventional concrete, drying shrinkage of UHPFRC may lead to cracking, and even though it may not have an influence on the structural integrity, the durability of concrete is perhaps adversely affected. The main purpose of the present work is to investigate the effect of the sulfate content of the mixture on the drying shrinkage as well as on the compressive strength of heat-treated and non-heat treated UHPFRC.

2. EXPERIMENTAL PROGRAM

Natural river sand (0-4 mm) and commercial quartz sand (0.6-2.5 mm) with a specific gravity of 2.66 and 2.65, respectively were used as fine aggregates. In order to simulate the issue of gypsum-contaminated aggregates in the Middle East, the natural crushed gypsum of nearly 38% SO_3 was used as a partial replacement for the natural river sand to raise the initial SO_3 content of the natural sand from 0.11% to 0.75, 1.5, 3, and 4.5%. A type F polycarboxylate-based superplasticizer (SP) in accordance with ASTM C 494-13 was used. The cementitious materials used in the present work were ordinary Portland cement (CEM I 42.5 R) with C_3A content of 8.8% as well as silica fume. 6 mm long and 0.16 mm diameter brass-coated steel fibers were also utilized to provide fiber reinforcement.

The UHPFRC mixtures were designed with a constant flow of 270 ± 10 mm and a constant water/binder (w/b) ratio of 0.174. Five UHPFRC mixes with five different SO_3 contents in natural sand of 0.11, 0.75, 1.5, 3, and 4.5% were prepared, thus yielding total SO_3 content by weight of cement of 2.76, 3.2, 3.71, 4.74, and 5.77%, respectively. Table 1 presents the mix proportions by weight of cement.

Table 1: Composition of UHPFRC (by weight of cement)

Mix designation		**SO_3 content in the mixture**	**Cement**	**Silica fume**	**0.6-2.5 mm quartz sand**	**0-4 mm natural sand**	**Micro steel fiber**	**Water**	**SP**
Normal curing	Heat curing								
NUH1	HUH1	0.0276	1	0.13	0.69	0.69	0.174	0.196	0.079
NUH2	HUH2	0.0320	1	0.13	0.69	0.69	0.174	0.196	0.079
NUH3	HUH3	0.0371	1	0.13	0.69	0.69	0.174	0.196	0.079
NUH4	HUH4	0.0474	1	0.13	0.69	0.69	0.174	0.196	0.082
NUH5	HUH5	0.0577	1	0.13	0.69	0.69	0.174	0.196	0.086

UHPFRC was mixed in a Hobart mixer. Initially, dry ingredients namely cement, silica fume, fibers and aggregates were mixed at the low speed for 5 min. Water was then added, and the mixture was remixed at the low speed for 5 min. Superplasticizer was added and mixed at the low speed for 5 min, and thereafter at the medium speed for 2 min. The mixtures were then poured into the molds and compacted by using a vibrating table. All specimens were covered with polyethylene sheets and left in the laboratory environment for about 1 day.

Drying shrinkage was monitored on 25 × 25 × 285 mm prismatic bars while compressive strength was determined on 50 mm cubes. For drying shrinkage measurements, a group of samples was air-dried after demolding at 23±2°C and 50±5% relative humidity until the end of the curing period. The other group was subjected to heat treatment for two days after casting at 80°C for 48 h. Thereafter, the heat-treated specimens were also exposed to air curing until the end of the curing time. A standard length comparator with a digital display accurate to 0.001 mm was used to measure the length change of bars. The initial length for drying shrinkage was measured after demolding. Additional length measurements were carried out periodically on the bars up to the age of 180 days. The compressive strength samples were also subjected to two curing methods, namely water curing and steam curing. The samples for water curing regime were stored in water after demolding until the testing age. The samples for accelerated curing were steam cured at 80°C for 48 h two days after casting and then stored in water until the age of testing.

3. RESULTS AND DISCUSSIONS

3.1 Compressive strength

Figure 1 presents the effects of sulfate content in the mixture and curing regime on the compressive strength of UHPFRC at the ages of 28 and 180 days. Under water curing, the sulfate addition generally resulted in a small increase of up to 4% in the compressive strength of the corresponding reference concretes. Under steam curing, the addition of sulfates also increased the compressive strength of concrete though the effect of increasing sulfate content was more pronounced on the strength of steam-cured specimens compared to water-cured ones. The compressive strength under steam curing condition was increased by up to nearly 8% due to the addition of sulfates. As seen in Figure 1, the steam-cured samples showed a higher compressive strength than the water-cured samples at both 28 and 180 days. The gain in strength due to the sulfate addition may be as a result of early formation of ettringite, which partially fills some pores reducing porosity of the cementitious paste. Larger amounts of the early ettringite may be formed under steam curing compared to water curing because the elevated temperatures accelerate the interaction of gypsum, C_3A and water.

3.2 Drying shrinkage and weight loss

Figure 2 shows the development of drying shrinkage of heat-treated and non-heat-treated UHPFRCs for 180 days of drying period. During the first 14 days of air curing, free shrinkage of all fabricated bars was generally observed to occur at a higher rate. Besides, the non-heat-treated samples continued to shrink for about 90 days, while the heat-treated specimens reached the asymptotic shrinkage strain value after about 56 days of drying due to the physical loss in water content of the samples.

The long-term drying shrinkage of non-heat-treated UHPFRC was slightly decreased (up to about 4%) by the addition of sulfates. However, the early age drying shrinkage was appreciably decreased by the addition of sulfates. At 7 days, for example, a decrease of about 11-17% was observed. On the other hand, the heat-treated samples showed a slight increase in drying shrinkage due to the addition of sulfates, in contrast to the non-heat-treated specimens. Of all the UHPFRC mixtures, however, the minimum drying shrinkage was observed for mixes with 3.71% SO_3 irrespective of the curing method. Despite a slight drying shrinkage difference between concretes containing additional gypsum and comparable concretes not containing additional gypsum, increasing the sulfate level seems to have two opposite effects on the magnitude of drying shrinkage of UHPFRC. While the additional sulfates may provide shrinkage compensation, the formation of ettringite may aggravate the self-desiccation influence and thus may raise the drying shrinkage. Which one of these two effects is predominant appeared to be dependent on the curing method.

The results also indicated that the application of steam curing diminished the long-term drying shrinkage by 41-45%. This was confirmed by the significant decrease in the moisture loss due to the consumption of free water associated with steam curing as shown from the weight loss results in Figure 3. One more possible explanation of the reduced shrinkage strains in steam-cured samples is due to the microstructural refinement accompanied with this curing regime. As seen in Figure 3, the moisture loss under normal curing was lower for mixes with the additional sulfates as compared to the corresponding plain mix. In contrast, the weight loss under heat treatment was higher for gypsum-incorporated mixes except for the mix HUH3.

In Figure 4a and b, the drying shrinkage is plotted against the corresponding weight loss for the heat and non-heat treated mixes, respectively. Obviously, the shrinkage was directly proportional to the weight loss. Furthermore, the high coefficient of correlation, R^2, indicated a very good correlation between the two phenomena. Other investigators [10, 11] also studied the relationship between the drying shrinkage and weight loss for other types of concrete. They also observed that the free shrinkage strain was almost proportional to the weight loss regardless of the matrix type.

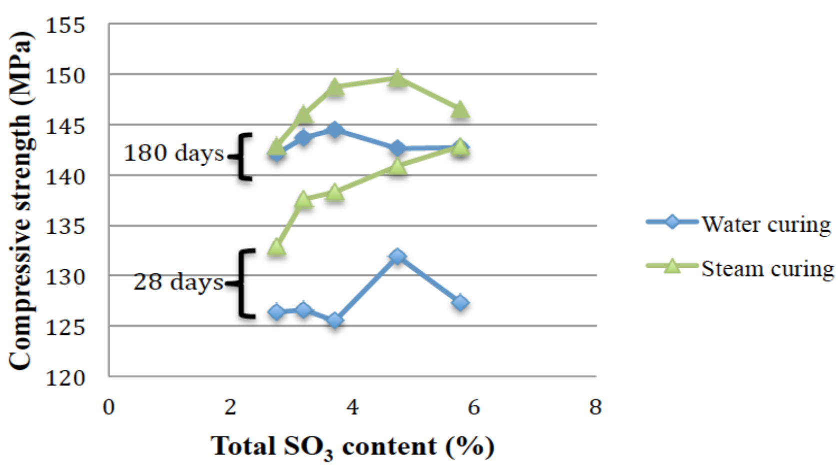


Figure 1: Effect of sulfate content on the compressive strength of UHPFRC.

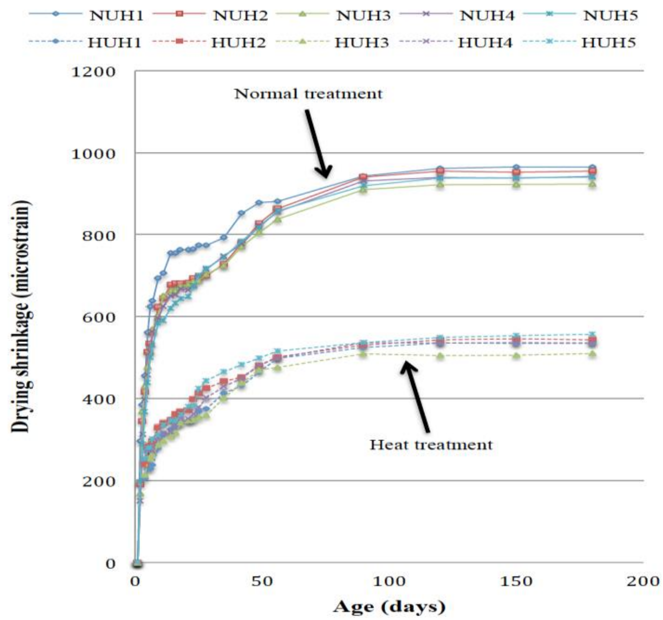


Figure 2: Drying shrinkage of UHPFRC.

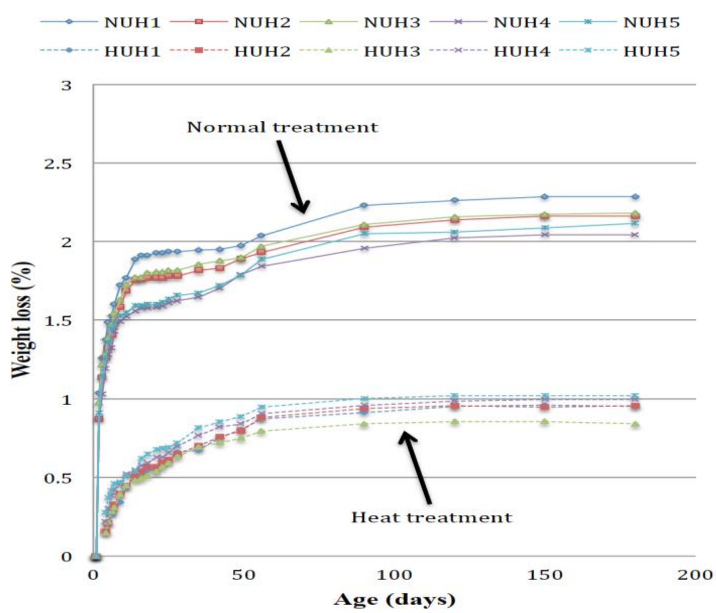


Figure 3: Weight loss of UH Figure PFRC.

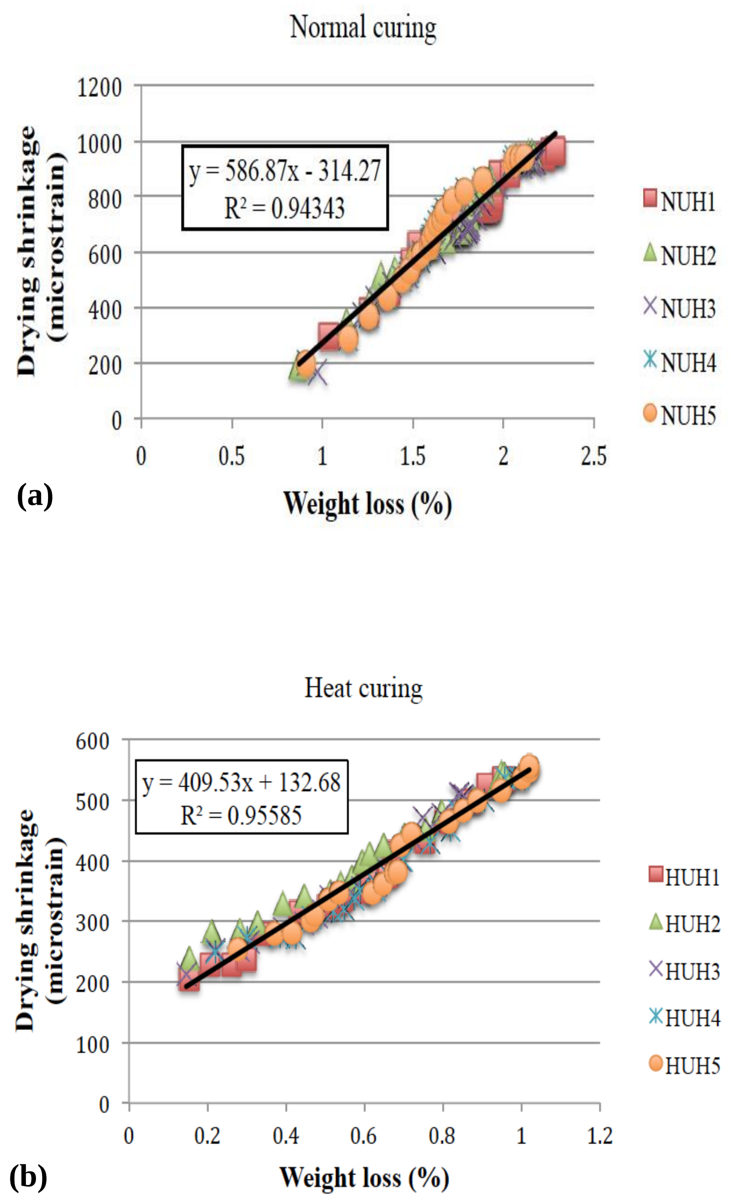


Figure 4: Relationship between drying shrinkage and weight loss, (a) normal curing, (b) heat curing.

4. CONCLUSIONS

From the experimental results presented in this research, the following conclusions can be drawn:

1. The effect of increasing sulfate content in UHPFRC mixtures on the drying shrinkage was found to be dependent on the curing regime.

2. Under normal curing condition, the mixes with additional gypsum showed a lower drying shrinkage and weight loss compared to the plain mixes though the positive effect of sulfates was more obvious at early ages.
3. In general, the inclusion of gypsum slightly increased the drying shrinkage and weight loss of steam cured UHPFRC.
4. The UHPFRC mixes with additional sulfates generally showed a higher compressive strength than those without additional sulfates irrespective of the curing method. However, the increase in compressive strength was more noticeable under steam curing compared to water curing.

REFERENCES

[1] Richard, P. and Cheyrezy, M.H., 'Reactive powder concretes with high ductility and 200-800 MPa compressive strength', (ACI SP 144-24, 1994) 507-518.

[2] Richard, P. and Cheyrezy, M., 'Composition of reactive powder concretes', *Cem. Concr. Res.* **25** (1995): 1501–1511.

[3] Mokarem, D.W., Weyers, R.E., and Lane D.S., 'Development of a shrinkage performance specifications and prediction model analysis for supplemental cementitious material concrete mixtures', *Cem. Concr. Res.* **35** (2005) 918-925.

[4] Garas, V.Y., Kahn, L.F. and Kurtis, K.E., 'Short-term tensile creep and shrinkage of ultra-high performance concrete', *Cem. Concr. Compos.* **31** (2009) 147-152.

[5] Graybeal, B.A., 'Characterization of the behavior of ultra-high performance concrete', PhD thesis, University of Maryland, 2005.

[6] Samarai, M.A., 'The disintegration of concrete containing sulphate-contaminated aggregates ', *Mag. Concr. Res.* **28**(96)(1976) 130-142.

[7] Crammond NJ., 'Examination of mortar bars containing varying percentages of coarsely crystaline gypsum as aggregate', *Cem. Concr. Res.* **14**(2) (1984) 225-230.

[8] Skalny J., Marchand J. and Odler I., 'Sulfate attack on concrete', Modern Concrete Technology: Spon Press; 2002.

[9] Atahan, H.N. and Dikme, D., 'Use of mineral admixtures for enhanced resistance against sulfate attack', *Constr. Build. Mater.* **25** (2011) 3450-3457.

[10] Bissonnette, B., Pierre, P. and Pigeon, M., 'Influence of key parameters on drying shrinkage of cementitious materials', *Cem. Concr. Res.* **29** (1999) 1655-1662.

[11] Güneyisi, E., Gesoğlu, M., Mermerdaş, K., 'Improving strength, drying shrinkage and pore structure of concrete using metakaolin', *Mater. Struct.* **41**(2008) 937-949.

CONCRETE WITH COARSE AGGREGATE FROM GLASS POROUS GRANULES - GPG

Davidjuk A.N.

Doctor of Technical Sciences, Director of the Institute, NIIZHB, Russia

Abstract

The glass porous granules as coarse aggregate (GGA) for light weight concrete have been investigated. The increased content of glass phase and uniform distribution of fine closed porous of correct form impart to granules the raised strength and the lowered parameters of heat conductivity and water absorption.

In comparison with the most widespread analogue burning expanded clay, GGA has the twice higher strength and 30-50 % lower heat conductivity and water absorption. Realization of raised mechanical characteristics of GGA and low bulk density predetermines their effective and rational use first of all for light weight concrete structures and for heat isolation of concrete building walls envelope.

Keywords: High strength lightweight concrete, glass porous aggregate, thermal isolation

1. INTRODUCTION

The polyfunctionality requirements for properties of enclosure walls demand from one side bearing strength and from other side enough thermal isolation determine the natural contradictions in the selection of materials for the implementation of these structures. However, in any case lightweight structural and insulating structures (LWSIS) can and should be their element, because of the possibility of such concretes is far from be exhausted. In this aspect it is very important the improvement of their properties through the use of new types of aggregates, in particular GPG.

2. PRODUCTION TECNOLOGY

Production technology of glass porous aggregates or GPG (GPG) has their own features for each case, but in general it consists from the mixing of source rocks with liquid alkaline component, drying, crushing granulation mass and calcination in rotary kilns, similar to receiving the roasting of foams from pyroclastic masses, such as expanded clay (keramzit).

The original source of raw materials for the production of the GPG can serve sedimentary and volcanically rock containing active silica and alumina: silica clay, diatomite, perlite, varsity and many others. Stocks of such rocks is truly inexhaustible, and are available in all regions of the country, unlike expansive clays for the production of keramzit.

Of alkaline components can be applied chemical waste and extractive industries, but the most alkaline reserve are the hydroxides of alkali metals (NaOH, KOH). They dissolve well in water, give a strong alkaline reaction and intensive react with the rock and sodium hydroxide in comparison with the potassium hydroxide has a more effective expanding capacity. Under conditions of high temperature (800 to 900 0C) and excess silica formed stable connection - hydrosilicates and hydroelasticity sodium of different basicity.

3. STRUCTURAL PROPERTIES

The structure of the GPG, as shown by microscopic examination, is a C-system of the cells predominantly of rounded shape with a size of 0.15 to 0.33 mm. Great co-holding the glass phase and a uniform distribution of fine pores closed provide the correct form of the GPG increased strength and reduced heat conductivity and water absorption (Table 1)

It should be noted that in comparison with the most common analogue of the roasting aggregates of the same density like expanded clay strength GPG is above to 100% and the thermal conductivity and water absorption is lower by 30-50%. Implementation of high physical and mechanical characteristics of the GPG with a low bulk density of the pre-determines their effective and efficient use primarily in structural insulating concrete enclosure walls density of 500-800 kg/m3 and strength up to 2,5 MPa [1].

Technological features of lightweight insulating concrete with GPG (LWC-GPG) consist in design of concrete mix components, with high consumption of coarse aggregate fraction of 5-20mm of concrete mixture with air-entraining and plasticizers (AE and SP) admixtures and various fine aggregates: natural, technological, roasting and crushed from the GPG, the consumption of which is reduced to the minimum possible - up to 200L/m3 of concrete, depending on the specified strength.

The strength of lightweight concrete is affected by consumption of cement the strength of the aggregates. To account for the influence of the tensile strength of aggregates on tensile strength of concrete dependence on the tensile strength of the mortar component and aggregates is proposed:

$$R_{Б} = (1{,}32R_{З} + 3{,}65)\ln R_{p} - 2{,}26R_{З} - 3{,}54 \quad (1)$$

where

R_{B} - strength of concrete and R_{3}, R_{P}, respectively, mortar component.

It is known that air entraining of mortar component reduces the ultimate strength of the concrete in proportional to the volume of the entrained air (EA). For the investigated concretes reduce strength is 3-5% for one percent of air entraining. In connection with this it is advisable to limit the amount of air entraining in the range of 6-8%.

Experimental data the ratio of the strength-density obtained at 5 types of GPG show that according to this indicator, the coefficient of structural quality, LWC with GPG is not inferior to the best analogues. Experiments shows that to describe the required dependencies it is expedient to use formula of Ryshkevich with updated coefficients of the form:

$$R = f(P) = K\left(\frac{P}{2670}\right)^{2.92} \quad (2)$$

where

R - compressive strength, MPa

P - relative porosity

K - coefficient of influence of the form GPG

To date, a large volume of experimental and theoretical material accumulated on the study of the deformation characteristics LWC with GPG, that make possible to create a normative documents for design of such structures [1]. Modulus of elasticity LWC on the GPG 10-30% higher than that of concrete; shrinkage deformation of concrete strength classes C3.5 to C7.5 significantly, up to 2 times lower, and the value of creep below up to 40%. The design of structures made of lightweight concrete on the GPG can be made using design standards for traditional LW concrete with keramzite.

Concrete with glass aggregate has sufficient protective capacity to the steel reinforcement with limitations in the use of fine aggregates or sand and moderate air entraining of the mortar component, not to exceed 6-8%.

As for alkaline corrosion for which, of course, there are prerequisites, due to increased hydraulic filler activity, it can be stated that this process has an attenuation nature and does not lead to cracking and loss of strength over time.

An example of attenuation of alkaline corrosion is more than 20 years of operation of the external wall panels made of lightweight concrete with GPG with strength class C50 and density D 800 kg/m3 in the Norilsk industrial area of the Russian Federation.

The most important performance characteristic LWC on the GPG is isolation properties, to the study of which was paid much attention.

For the thermal conductivity of the concrete in the dry state $\lambda_{б}$ in dependence on the coefficients of thermal conductivity and volumetric concentration of the matrix and filler first proposed to use a modified model of Hirsh -Netsvetaev obtained previously for the estimation of elastic modulus of concrete.

$$\lambda_{Б} = \frac{2}{\left(\frac{1}{\lambda_З V_З + \lambda_{ЦК} V_{ЦК}} + \frac{V_З}{\lambda_З} + \frac{V_{ЦК}}{\lambda_{ЦК}}\right)}, \qquad (3)$$

where $V_3 V_{цк}$ - volume concentration of the filler and cement stone $\lambda_б$, λ_c - coefficients of thermal conductivity of the aggregate and cement paste, W/m^2 ^{0}C

The resulting model allows to estimate the contribution of each element of a two-level system "matrix - filler" in the formation of the thermal conductivity of concrete. The analysis of the model led to the conclusion that the key to reducing the thermal conductivity of lightweight concrete you should consider reducing the thermal conductivity of the porous filler. The 10% reduction of the thermal conductivity of the filler in LWC on GPG brings 4 times greater effect of reducing λб than the same reduction of the thermal conductivity of cement paste.

In this respect, the use of vitreous aggregates provides undeniable advantages over traditional roasting fillers because the amorphous phase is "dispersing screen" on the way of "phonons- quanta of energy" vectors of heat waves in the solid body.

A calculation shows that in this case, the thickness of the outer wall should be no more than 0,42 m for Moscow region (design outside temperature in the most cold period - minus 27 degrees Celsius) . Thus, the specified value will provide minimum requirements for the thermal protection of the exterior walls of LWC on the GPG condition of energy conservation in the climatic area of Moscow. Lists the parameters of enclosing structures are not limiting and quite acceptable for the manufacture of a single layer of exterior walls with thickness up to 45cm in precast and monolithic variants. From the available reserves to reduce, the thermal resistance LWC on the GPG should be noted the possibility of using high quality cements, blast-furnace cement, silica fume additives and polymer fibers. Finally there is a technological possibility of making single-layer panel with an inner insulating layer of large-pore lightweight concrete on the GPG with density D200-D300.

It should be particularly emphasized that the presented experimental and theoretical material for concrete on the GPG, was first investigated in Moscow Scientific Research Institute on Concrete and Reinforced Concrete - NIIZHB named after professor A. A. Gvozdev – the founder of limit state design conception for building structures. This results have been confirmed and refined in recent years by many researchers in this country.

4. CONCLUSIONS

High coefficient structural quality (relationship between strength and density), high thermal resistance, widespread availability of raw materials for the production of the GPG, the absolute sustainability with the possibility of recycling of industrial waste, durability and fire resistance, a complete rejection of the use of polymer thermal insulation, which has a lot of serious shortcomings, all these factors are the apparent advantages of LWC with using the GPG.

To date, there is sufficient experimental and theoretical material in the form of test results and study of properties of the LWC- GPG and pilot and industrial technologies and recommended literature.

Innovative breakthrough, combining the efforts of researchers, investors and representatives of local authorities is demanded. In this regard, the ambitious slogan "Back to the single-layer panels!" can and should be the main direction vector of the industrial production of high-performance exterior walls of LWC on the GPG.

5. TABLE AND FIGURES

Table 1: Physico-Mechanical Properties of GPG

No	Performance characteristics	Glass porous granules			Expanded clay	
1.	Fraction, mm	5-10			5-10	
2.	Bulk density, kg/m3	130	300	600	300	700
3.	Strength when compressed in the cylinder, MPa	0,4	1,8	6,0	0,8	4,0
4.	The thermal conductivity of backfill W/m °C	0,040	0,075	0,11	0,11	0,16
5.	Water absorption, % by mass, per 1 hour	40	18	8	25	12
6.	The content of the glass phase, %	95			85	
7.	Freeze -thaw cycles not less than	15			15	

Table 2: Coefficient of Concrete Thermal Conductivity of LWC with GPG in Dry State

Kind of concrete, the dependence	The values of λ_0, W/m °C at an average density, kg/m^3			
	500	600	700	800
Concrete on the GPG $\lambda_0 = 0{,}0002\rho + 0{,}0207, \ldots R^2 = 0{,}9$	$\frac{0{,}12}{85}$	$\frac{0{,}14}{87{,}5^*}$	0,16	$\frac{0{,}18}{85{,}5^*}$
Expanded clay concrete		0,16		0,21
Polystyrene concrete		0,145		
Cellular concrete		0,14		0,21

* - % relative to expanded clay concrete

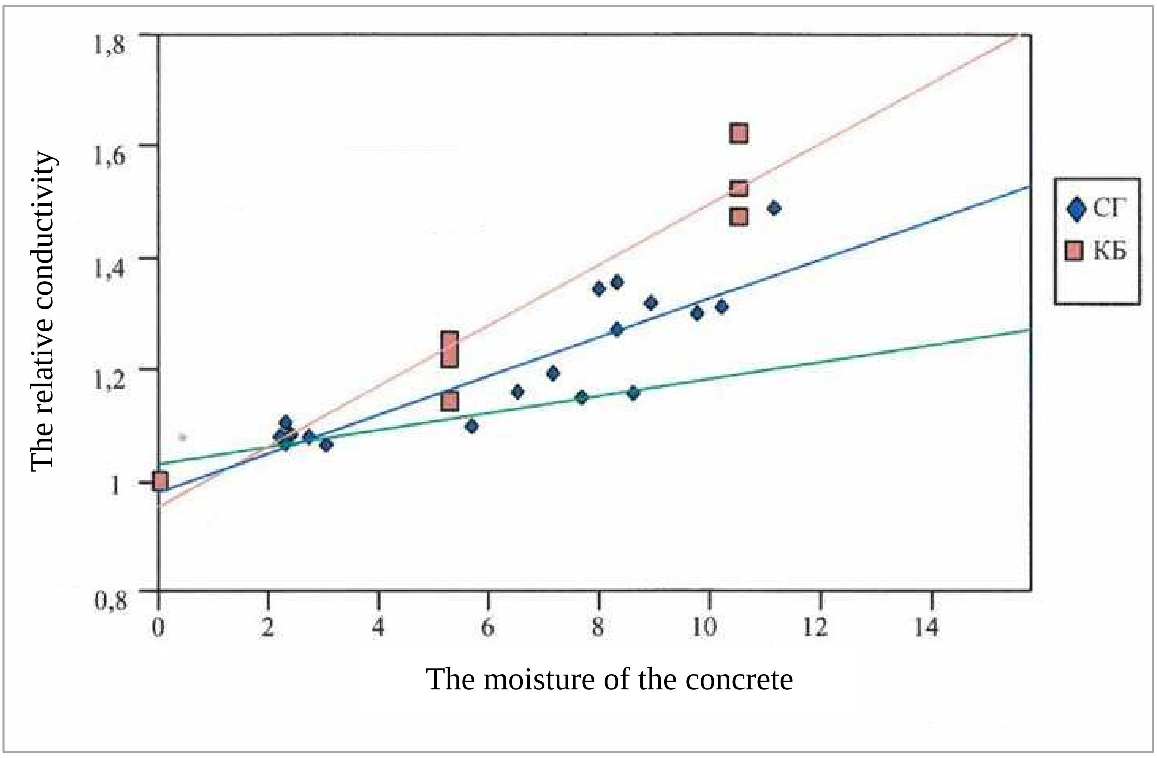


Figure 1: Effect of Moisture Content on the Coefficient of Relative Thermal Conductivity of Lightweight Concrete on GPG

The concrete on the GPG y = 0,0366x + 0,9778

R^2= 0,778

Lightweight aggregate concrete y = 0,0572x + 09513

R^2= 0,9205

GPG - LW concrete on the GPG;

LWC – expanded clay - concrete

REFERENCES

[1] Davidyuk A.n. Light construction-insulation concrete on porous glassy aggregates. Publisher “Krasnaya Zvezda” Moscow, 2008. -208 pp.
[2] Karimov, I.Sh. Proceedings of BGAU, Concretes, November 2008 page. 14-18.
[3] Thermal protection of buildings. Building Code of Practice, SNiP 23-02-2003.
[4] Kazantseva L.K., Storozhenko G.I. Special properties of foam glass from natural raw materials. Journal of building materials, no 9 p. 34-38
[5] Proceedings of Research Center “Stroitelstvo” (Building), no. 7-8, 2013, p143.

EFFECT OF AIR VOIDS ON HYBRID CONCRETE FOR SUSTAINABLE BRIDGE DECK OVERLAY

Kyong-Ku Yun (1), Kyong Namkung (2), Kyeo-Re Lee (2), Seung-Yeon Han (2) and Gyu-Young Han (2)

(1) Professor and Director, Center for Regional Construction Technology, Department of Civil Engineering, Kangwon National University, S.Korea

(2) Graduate student, Department of Civil Engineering, Kangwon National University, S.Korea

Abstract

The purpose of this research is to investigate the effect of air content on the strength development of hybrid concrete, which contains an organic material of latex and a mineral of silica fume. The addition of silica fume on Latex-modified concrete (LMC) makes it possible to reduce the quantity of latex for an economic reason, while keeping the required performance for bridge deck overlay. The main experimental variables of this study are latex contents, silica fume contents and air contents. The results are as following:

The measured air is higher at the fresh concrete than that at the hardened, which may be due to hydration and compaction. The differences of these two are between 0.5 to 3.1 %.

The compressive strength is much higher at a specimens of less air, as expected. The differences of those two are low at shorter curing days but become higher at longer curing days. The compressive strength of hybrid concrete is much affected from air void content. The biggest difference is 9.7MPa at 6% of silica fume and 7% of latex. The compressive strengths increases as silica fume content increases at all levels and at all curing ages. This is well expected by filling effect and pozzolan effect of silica fume. However, there is no trend in latex content variation. This is, also, well anticipated in compressive strength development because of the flexibility of the latex component named by Butadiene.

The flexural strength with high air and less air does not show a significant difference as compressive strength. The flexural strength of hybrid concrete is not affected from air void content.

Keywords*:* Hybrid concrete, air content, bridge deck, overlay

1. INTRODUCTION

Latex-modified concrete (LMC) is a good overlay material for new bridge construction in most conditions for its excellent physical and mechanical properties and durability. It was reported that latex modification of concrete provides the material with higher flexural strength, as well as high bond strength and reduced water permeability [1–4]. Since the introduction into Korea in 2000, thousands of new bridges have been overlaid with latex-modified concrete [5]. While it has many advantages in bridge deck overlay, the disadvantage of LMC is cost. It is very expensive, because it include 15% of latex and this results in 4 times of the price of OPC.

Silica fume is used in concrete to improve its properties. It has been found that silica fume improves compressive strength, bond strength, and abrasion resistance; reduces permeability; and therefore helps in protecting reinforcing steel from corrosion [6]. Silica fume has been used as an addition to concrete up to 10 percent by weight of cement, although the normal proportion is 7 to 8 percent. Silica fume works on two levels, the first one described here is a chemical reaction called the "pozzolanic" reaction. The second one is filling effect of silica fume, because silica fume is 100 to 150 times smaller than a cement particle it can fill the voids created by free water in the matrix. While the concrete containing silica fume has many advantages over OPC, it has, also, disadvantage. It is vunerable to dry shrinkage cracking.

The purpose of this research was to investigate the effect of air content on the strength development of a hybrid concrete, which contains an organic material of latex and a mineral of silica fume. The addition of silica fume on LMC makes it possible to reduce the quantity of latex for an economic reason compared from LMC, while keeping the required performance for bridge deck overlay. The hybrid concrete might have both properties of latex-modified concrete and silica fume-contained concrete.

2. EXPERIMENTAL PROGRAMS

The main experimental variables of this study were latex contents (3, 5 and 7%), silica fume contents (6, 7 and 8%) and air amount (4% and over 10%), in order to investigate the effect of air voids on a hybrid concrete containing organic latex and inorganic silica fume.

2.1 Materials

For an optimized mix proportions of hybrid concrete both of latex and silica fume were carefully selected from a series of preliminary tests. Latexes were made of 48 percent solid suspension of styrene butadiene rubber [4]. The proportions of styrene and butadiene were 66 percent and 34 percent, respectively. Latexes are colloidal dispersions of small spherical organic polymer particles in water. Silica fume was a very fine powder with a very high specific surface area (20,000–30,000 m^2/kg) and with a 94% of SiO_2 content.

The maximum size of coarse aggregate was 13mm considering the thickness of bridge deck overlay. The coarse and fine aggregates were crushed limestone and natural sand, respectively. The specific gravities of coarse aggregate and fine aggregates were 2.61 and 2.63, respectively. It is believed that a latex content of 15 percent is the optimum ratio for LMC, considering a various performance [3]. This study adopted three levels of latex contents (3, 5 and 7%), three levels of silica fume contents (6, 7 and 8%) and two levels of air amount (4% and over 10%) for a hybrid concrete. The concrete mix proportions are shown in Table 1.

2.2 Concrete mixing and curing

The hybrid concrete was mixed at a fixed water-cement ratio of 39%, binder content of 390kg/m^3, and fine aggregate ratio(S/a) of 56%. An additional super plasticizer of 1.0% was mixed to ensure workability of hybrid concrete. An additional defoamer of 1.0% was mixed for 4% air content concretes or was not added for high air contents of concrete. The compressive strength and flexural strength were measured according to specifications, respectively.

Concrete cylinders of 100×200 were cast from each mixture for compressive strength test and air void test. Beam specimens of 100×100×400mm were fabricated for flexural bending test. After casting, all the molded samples were covered with water-saturated burlap, and left in the casting room at 22 Celsius and 50±2 % relative humidity. After 24 hours, the specimens were demolded and cured at a controlled curing room at the temperatures of 20 Celsius in 80% relative humidity until testing [7-9].

An air content of fresh concrete was measured by pressure method according to ASTM C231 [10]. An air content of hardened concrete was measured by image analysis [11]. A concrete cylinder specimen, having 100x200mm (4x8in), was cut into cylinder having 50mm thickness with a water-cooled diamond saw. One face of the sample was polished on a water-cooled rotating lap followed by 60, 100, 220, 320, 400 and 600 grit fixed SiC papers, and was done by image analysis for air void content and spacing factor.

Table 1: Mix proportion of hybrid concrete

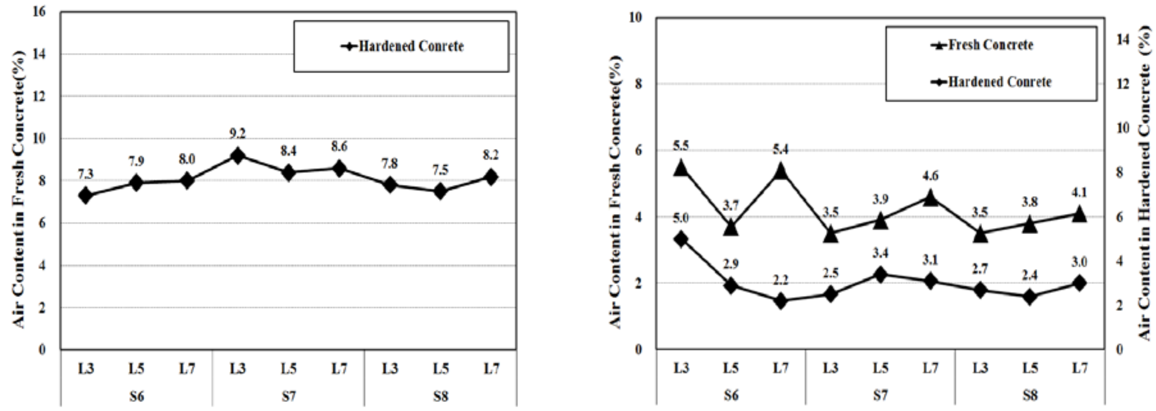
Mix Type	Unit Weight (kg/m^3)						SP (%)	Antifoamer (%)
	Water	Cement	Silica Fume	Latex	Sand	Gravel		
S6L3	139.4	367	23.4	24.4	971	768	1.0	1.0 or N/A
S6L5	131	367	23.4	40.6	959	760		
S6L7	122.5	367	23.4	56.9	948	751		
S7L3	139.4	363	27.3	24.4	970	768		
S7L5	131	363	27.3	40.6	959	759		
S7L7	122.5	363	27.3	56.9	947	750		
S8L3	139.4	359	31.2	24.4	969	767		
S8L5	131	359	31.2	40.6	958	758		
S8L7	122.5	359	31.2	56.9	947	750		

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

3.1 Air content and air spacing factor

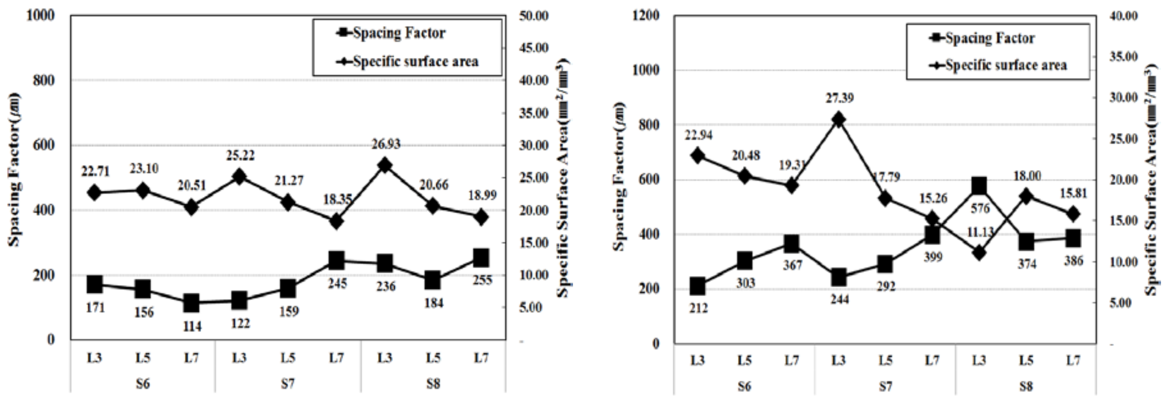
Figure 1(a) and 1(b) compares the air content of fresh and hardened hybrid concrete with high air and less air, respectively. The air content of fresh concrete at high air was not measured because it exceeded 10%. The air is higher at fresh concrete than that at the hardened. The differences of those two are between 0.5 to 3.1 %, but they show similar trends.

The spacing factors of the hybrid concretes were measured to be between 172 and 255 μm at high air content, while it was measured to be between 212 and 576 μm at less air content.



(a) With high air content (b)with less air content

Figure 1: Air contents of hardened concrete

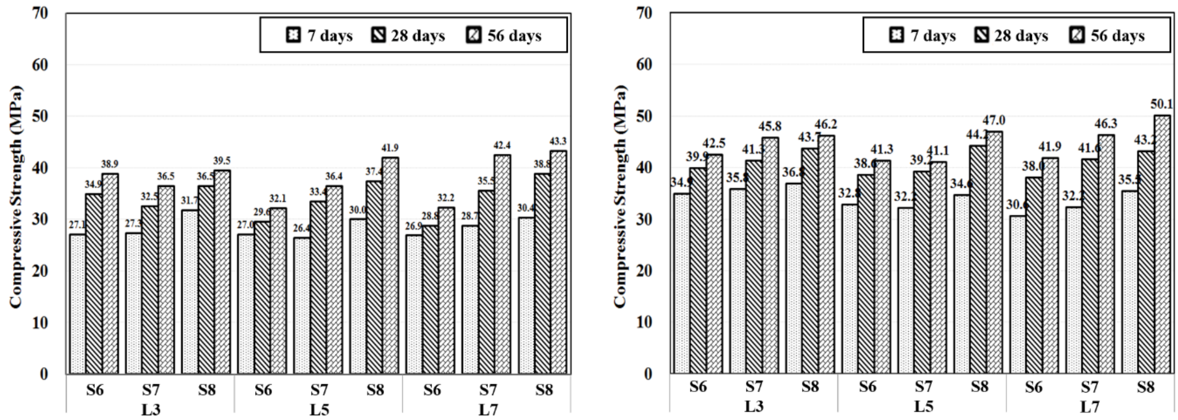


(a) With high air content (b)with less air content

Figure 2: Air spacing factor and specific surface of hybrid concrete

3.2 Compressive strength development

Figure 3(a) and 3(b) compares the compressive strength development with high air and less air, respectively. The compressive strength is much higher at a specimens of less air, as expected. The differences of those two are low at shorter curing days but become higher at longer curing days. The compressive strength of hybrid concrete is much affected from air void content. The biggest difference is 9.7MPa at 6% of silica fume and 7% of latex. The compressive strengths increases as silica fume content increases at all levels and at all curing ages. This is well expected by filling effect and pozzolan effect of silica fume. However, there is no trend in latex content variation. This is, also, well anticipated in compressive strength development because of the flexibility of the latex component named by Butadiene. The latex films between the hydrated cement and aggregates, specially the flexibility of Butadiene, affected to compressive failure mode. However, the latex film might act as bridging between aggregate and cement paste.

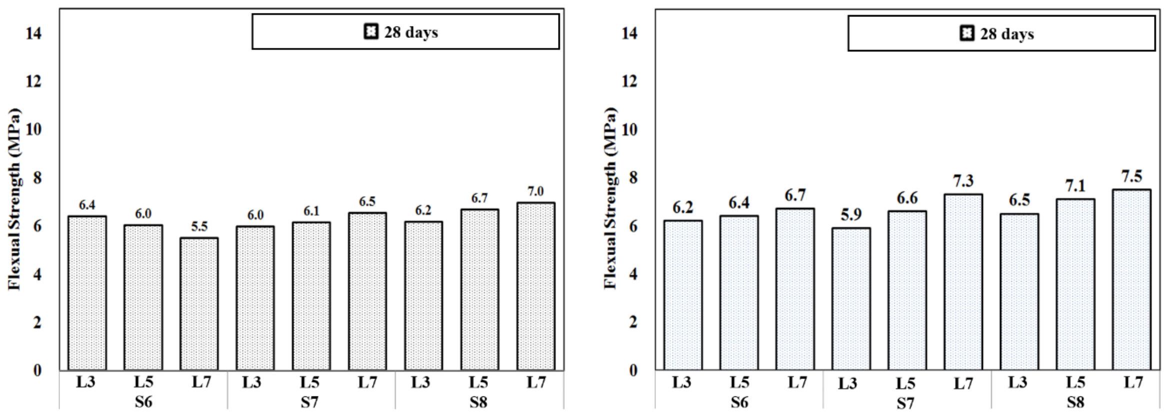


(a) With high air content (b) with less air content

Figure 3. Compressive strength development of hybrid concrete

3.3 Flexural strength

Figure 4(a) and 4(b) compares the flexural strength development with high air and less air, respectively. The differences of those two are not significant as compressive strength. The flexural strength of hybrid concrete is not affected from air void content. They increase as silica fume content and latex content increase at both cases. This is well expected by filling effect and pozzolan effect of silica fume and bridging effect of latex films between aggregate and cement paste.



(a) With high air content (b)with less air content

Figure 4. Flexural strength development of hybrid concrete

4. CONCLUSION

The purpose of this research was to investigate the effect of air content on the strength development of a hybrid concrete, which contains an organic material of latex and a mineral of silica fume. The main experimental variables of this study were three levels of latex contents (3, 5 and 7%), three levels of silica fume contents (6, 7 and 8%) and two levels of air amount (4% and over 10%). The results were as following:

1. The air is higher at fresh concrete than that at the hardened. The differences of those two are between 0.5 to 3.1 %, but they show a similar trends.
2. The compressive strength is much higher at a specimens of less air, as expected. The differences of those two are low at shorter curing days but become higher at longer curing days. The compressive strength of hybrid concrete is much affected from air void content. The biggest difference is 9.7MPa at 6% of silica fume and 7% of latex.
3. The compressive strengths increases as silica fume content increases at all levels and at all curing ages. This is well expected by filling effect and pozzolan effect of silica fume. However, there is no trend in latex content variation. This is, also, well anticipated in compressive strength development because of the flexibility of the latex component named by Butadiene.
4. The flexural strength with high air and less air did not show a significant difference as compressive strength. The flexural strength of hybrid concrete is not affected from air void content.

ACKNOWLEDGEMENTS

This research was supported by a grant (13RDRP-B066780) from Regional Development Research Program funded by Ministry of Land, Infrastructure and Transport of Korean Government, and was performed using the facilities of the Institute for Advanced Construction Materials at Kangwon National University, Chuncheon, South Korea.

REFERENCES

[1] Kuhlmann, L.A. and Foor, N.C., “Chloride Permeability Versus Air Content of Latex Modified Concrete,” Cement, Concrete, and Aggregates, CCAGDP, Vol.6, No.1, Summer 1984, pp.11-16.
[2] Ohama, Y., Handbook of Polymer-Modified Concrete and Mortars Properties and Process Technology, NOYES PUBLICATIONS, 1995.
[3] Sprinkel, M. M., Twenty Year Performance of Latex Modified Concrete Overlays, Transportation Research Record 1335, TRB, National Research Council, Washington, D.C., 1980, pp. 27-35.
[4] Yun, K.K. Kim, D.D., Choi, S.Y., Durability of Very-Early Strength Latex-Modified Concrete against Freeze-Thaw and Chemicals, 83rd Transportation Research Board Annual Meeting, Paper No. 04-3895, National Research Council, Washington, D.C., 2004.
[5] Yun, K.K., Lee, J.H., Hong, C.W., Kim, K.H. and Kim, T.K., "Field Applicability Evaluation of SB Latex-Modified Concrete for Concrete Bridge Deck Overlay", *Korean Society of Pavement Engineers*, Vol.3, No.4, December 2001, pp93~103.
[6] Walters, D. Geny, “Silica Fume, Latex-Modified Portland Cement Mo1tar-Concrete”, Transportation Research Record 1301, 1991, pp. 36-41.
[7] KS, Aggregates for Concrete, KS F2526, Korean Standard, Korea, 2002.
[8] KS, Test for Compressive Strength of Concrete, KS F2405, Korean Standard, Korea, 2001.
[9] KS, Test for Flexural Strength of Concrete, KS F2408, Korean Standard, Korea, 2000.

[10] ASTM C 231 ; Standard Practice for Air content of freshly mixed concrete by pressure method, ASTM annual book of ASTM Standards, 2013.
[11] ASTM C 457-82a ; Standard Practice for Microscopical Determination of Air-Void System in Hardened Concrete, ASTM annual book of ASTM Standards, 1982.

CREATING CUSTOMER AND COMPANY VALUE BY USING CONCRETE TRANSPORT OPTIMISATION DURING PLANNING AND REAL TIME

Dr. Paul Flachskampf (1), Julien Drevon (1) and Thomas Bergmans (1)

(1) INFORM Institute for Operations Research and Management GmbH, Germany

Abstract

Transportation costs are a major expense for concrete suppliers, and so many try to make best use of their trucks in order to reduce costs by improving productivity. However, truck productivity optimisation does not necessarily imply a return on investment (ROI) for the concrete supplier, as the ROI greatly depends on the way hauliers are paid in the day to day business. This paper addresses the issue of the accessible ROI through productivity optimisation in concrete transportation. Based on several payment schemes and typical order books, the suitability of the payment schemes towards optimisation will be compared. The results show large variations between the schemes, some becoming more expensive while others imply a natural ROI. As a conclusion, the paper will provide a condensed table that presents the consequences of each scheme along broad strategic recommendations assessing the expected ROI caused by the productivity optimisation.

Keywords: Logistics, optimisation, haulier-management, ROI, transport cost

1. INTRODUCTION

Concrete distribution is a complex logistics management task. Small and large order quantities must be delivered quickly, accurately and as cost-effectively as possible to geographically widely-distributed and frequently-changing unloading points. The objectives are therefore conflicting. 'Cost-effective' means minimising transport cost and driving distances and maximising truck utilisation [1]. 'Fast and accurate' means creating robust and reliable plans, taking unforeseen eventualities into account, as well as exclusive deliveries.

If all the constraints that need to be considered when balancing these objectives are added together, one of the most interesting issues of operations research emerges: the vehicle routing problem (VRP). The task is to design the optimal set of routes for a fleet of vehicles to serve a given set of customers. In this case VRP is even extended to the vehicle routing problem with time windows (VRPTW), meaning that the delivery locations have time windows within which the deliveries must be made. Furthermore, the vehicle fleets are heterogeneous and there are many more complex factors to be considered, such as splitting bulk orders into deliveries.

It is not difficult to imagine the data volumes that concrete suppliers with a large number of locations and hundreds of trucks have to compute to 'only' find a set of optimised routes. The human brain is not up to this sort of challenge. This is why even experienced schedulers or dispatchers are stretched to their limits with this task if they only work with an Excel spreadsheet or software that does not have its own intelligence.

Conventional solutions for building materials logistics offer support to logistics coordinators in arranging loads and truck schedules by providing a wide range of functions. They copy order-relevant data from the ERP systems automatically, provide user-friendly graphical interfaces on which loads can be assigned to trucks, calculate schedules including the journey times and visualise them on maps. If they have a GPS telematics component, they can also track the position of the truck and the delivery status in real time. Compared to planning using a scheduling plan on paper or spreadsheet, this is already a giant leap forwards in planning technology. Even with all the support provided, thanks to the digitalisation of the scheduling process, the decision as to which (special) truck to use for a delivery to a specific customer is still made based on incomplete knowledge by a human. Consequently, it can neither be the best decision in terms of cost nor with regard to service quality.

Intelligent optimisation software uses algorithms that analyse a virtually endless number of scheduling decisions in real time and identify those that are ideal for minimising costs and maximising service quality – based on the business criteria defined. Intelligent optimisation means that scheduling is automated. As the software identifies the best decision in accordance with the optimisation criteria anyway, it can suggest the decision to take to the schedulers and dispatchers or action this decision automatically and forward the corresponding instructions to the drivers, hauliers and loading stations.

This relieves the logistics coordinator from the burden of making routine decisions, allowing him to focus on difficult cases and scheduling for the next days. This real-time capability enables the schedulers and dispatchers to deliver orders placed at short notice, to allow for changes to orders during the day of delivery and to react to unforeseen events (e.g. queues, heavy traffic, production issues, etc.) whilst maintaining an economic schedule to ensure delivery to the customers on time in full (OTIF).

2. PROBLEM PRESENTATION

Fleet management optimization is a key step toward operational excellence. Most concrete suppliers want to improve their business efficiency in its broad definition [2] by optimizing their deliveries with dedicated software. In this way, they want to improve their level of service and their flexibility, expecting the financial results to do the same through cost reduction.

However, if the genuine operational costs do decrease the benefits fall to the organization that incurs these costs, which is typically the haulier. They will therefore be the only ones, in the first instance, to benefit from these falling costs. Whether it will affect the supplier's costs or not depends on the existing agreement between the supplier and the haulier about the payment scheme used for the service provided, making this kind of agreement even more business sensitive that it already is [3]. Far from being the usual situation to see a financial benefit, the supplier rather usually observes little to no effect on their own costs and questions the optimization: many payment schemes are simply not built to yield results from operational logistics efficiency.

The purpose of this study is to highlight these issues and to provide some insights and general recommendations about them. This study was conducted using several real payment schemes provided by various partners and a unique set of orders used as a reference for every scheme studied. The optimized plan was obtained using INFORM's software for transport planning optimization, SyncroTESS, and was also used with every scheme. The operational results obtained are the following:

Table 1: Operational results

Operational results	Manual plan	Optimized plan	Difference
No. trucks used	28	18	– 35.71 %
Loads/truck/day	5.14	7.89	+ 53.40 %
Loaded ratio	50.41 %	52.15 %	+ 1.74 %
Empty mileage	3,524 km	3,218 km	– 8.68 %
Overall distance traveled	7,106 km	6,725 km	– 5.37 %
Overall working time	226 h	215 h	– 4.52 %

3. SCHEME COMPARISON

As explained before, these operational results solely imply a cost reduction for the haulier, the impact on the concrete supplier depending on the scheme used. The four schemes presented in this paper were selected considering that they are typical examples (either broadly used or especially illustrative). Each of them is carefully described and analyzed below.

3.1 m³-priced model

Scheme definition:

$$p_{m^3} = f(d, truck) \qquad \begin{cases} p_{m^3} = price\ per\ cubic\ meter \\ d = distance\ loading\ plant\ \text{-}\ customer \\ truck = individual\ truck\ type \end{cases} \tag{1}$$

The cost of a delivery is defined in this scheme per m^3 transported: the unit cost depends solely on the distance between the loading plant and the customer's site, as well as the type of truck used. This scheme is built by the haulier from an expected turnover: the cost per m^3 is calculated for each route in order to meet this expected turnover. When the distance increases, so does the cost per m^3.

Analysis & optimization suitability:

This scheme does not adequately reflect the cost reality: it is an artificial way to "price" the service, on a volume-based method – called "traditional" [4] – to mitigate cost risks for poorly utilized fleets, calculated from the required turnover. In this scheme, the transportation cost is defined in the order definition and cannot be changed for most of the orders. In particular, the number of trucks used and the effective distance travelled do not impact the transportation costs. Thus, using one truck per delivery or having tremendous backhaulage routes do not change in any way the transportation costs: the two main levers of operational optimization are worthless when using that scheme for ready-mix concrete.

Expected ROI from transport optimization: Close to none.

3.2 Order-defined pricing

Scheme definition:

$$p_{order} = qty \cdot rate_{carriage}(d) + rate_{fuel} \cdot d \tag{2}$$
$$p_{order} = f(d, qty)$$
$$where: d = distance\ loading\ plant - customer$$

The costs are divided in two parts: The "genuine" transportation costs, that are proportional to the quantity transported and that depend on nothing but the distance between the loading plant and the customer; the "fuel compensation" part, which covers the price variations of the fuel. For the concrete supplier, it ends up as a global cost for each delivery that is directly defined by the order and that depends on the quantity delivered and the distance plant-customer.

Analysis & optimization suitability:

This scheme depends solely on parameters defined in the customer orders (delivery point, quantity, etc.): hence, each order has a defined cost which cannot be change without changing the rates. Whichever way the trucks are managed, it won't change the final transportation costs: operational optimization is "useless" by itself in terms of financial results (no ROI to be expected). Therefore this scheme is completely "planning-proof". Planning cannot modify the transportation costs – in a greater extent than the first scheme – and thus making any effort to reduce costs through operational optimization pointless.

Expected ROI from transport optimization: None.

3.3 Fixed fees & bonuses model

Scheme definition:

$$p\_truck = FlagFall + Productivity\ Bonus + Overtime\ Fees \tag{3}$$

$$where: \begin{cases} FF = fixed\ costs\ per\ driver \\ PB = proportional\ to\ the\ number\ of\ delivery \\ \qquad performed\ over\ the\ objective \\ OF = Cost\ of\ the\ overtime \end{cases}$$

There are three main parts in this pricing scheme: The flagfall, which is a fixed cost and does not depend on the amount of work performed; the productivity bonus, which grants a financial bonus for each delivery performed over the objective; the overtime fees, which are paid to the driver for the overtime worked. Ultimately, the supplier perceives the pricing scheme as a flagfall per truck plus a variable part that depends on the driver's performance, shaped as bonuses.

Analysis & optimization suitability:

The "fixed part" of this scheme ensures the best results from the reduction in the number of trucks used that occurs with the operational optimization, while the bonuses reward the work done and encourage efficiency (although they increase the overall costs, it still costs less than using additional trucks). Overall, the scheme reacts to the optimization in a good way. The benefits from the reduction in the fixed part exceed the additional costs incurred.

Expected ROI from transport optimization: Good (depending on the weight of the variable part).

3.4 Cost-based scheme

Scheme definition:

$$p_{truck} = p_{flagfall}(truck) + t \cdot p_{hour} + d \cdot p_{km} \tag{4}$$

$$where: \begin{cases} d = distance\ effectively\ traveled\ during\ the\ day \\ t = time\ effectively\ worked\ during\ the\ day \end{cases}$$

The price paid by the concrete supplier is divided in 3 parts: The first one is a fixed fee paid for using the truck for a day; the second one is based on the time worked, proportional to the number of hours worked; the last one is proportional to the distance effectively travelled during the day. Thus, the supplier pays both for using the truck for the day and the amount of work performed. The final cost is calculated for each truck.

Analysis & optimization suitability:

This scheme is cost-based, meaning that it reflects the reality behind the operational costs. Thus, it covers both fixed and variable costs, and each part carries a profit-margin for the haulier (ensuring that he will always earn money, whatever happens). Moreover, this scheme is built in a way that operational optimization leads naturally to a reduction in the transportation costs, and the benefits from optimization are then shared between the haulier and the concrete supplier. Therefore this scheme is highly suited to operational optimization and naturally encourages it through financial reward.

Expected ROI from transport optimization: As high as the optimization is efficient.

4. SUMMARY, RECOMMENDATIONS & CONCLUSION

Table 2: Summary results

Schemes	Results	Recommendations for improving ROI when investing into optimization
m^3-priced	C-	Nearly "Planning-proof": Change scheme or negotiate rates.
Order-defined	D	"Planning-proof": Change scheme or negotiate rates.
Fixed fees & bonuses	B+	Good enough. However, best would be to move from the bonus logic to a variable fees logic (like the cost-based model) – for simplicity and transparency.
Cost-based	A+	Best practice. Pay attention to parameters settings (best if based on costs).

From the results above, we can divide these four schemes into two distinct groups which show different critical properties:

Table 3: Two different mindsets

Schemes	Results	Transportation costs' nature	Impact of the planning on costs
- m^3-priced - Order-defined	Bad	- No fixed costs - Effective work not taken into account → Not cost-based	None: Costs are essentially defined by the orders
- Fixed fees & bonuses - Cost-based	Good	- Fixed fees per truck - Effective work charged →Cost-based	Significant: Costs depend strongly on planning and fleet management

Almost every existing pricing scheme can be included in one of these two groups. Unfortunately, the most common schemes [5] – the "traditional ones – are the least appropriate to optimization, their unsuitability being displayed in the need to enforce special rates, minimum work-loads, and other additional payments to the agreement in order to ensure an acceptable level of productivity and efficiency.

These schemes are noticeable by the impact of planning not being taken into account during cost calculations. The overall costs depend solely on the orders: however the trucks are managed, the final transportation costs will stay the same. Even something as important as the number of trucks used is unrelated to the final costs to the supplier. In these schemes, the genuine operational costs are overlooked and an "artificial" pricing, unrelated to the genuine costs, is enforced. No direct financial benefit through optimization can be expected from these schemes, and improving the fleet management won't improve the financial results.

On the other hand, the most relevant schemes with respect to operational excellence are under-represented [5].

These “best-practice” cost-based schemes – calculated from the execution of a plan – present several positive aspects, the first of them being the possibility to reduce costs through operational excellence and to generate profit out of it. However, they carry other critically positive properties compared to the other ones:

- Benefits of the optimization are shared with the haulier – it’s a win-win situation.
- Operational excellence is naturally encouraged through the expected financial benefits, creating a virtuous circle.
- The simplicity of these schemes: special rates are not needed to control efficiency (the costs reflect what effectively happens).
- Financial safety for the haulier: the best use of its fleet grants him more potential activity and customers. Moreover, its fixed costs (especially high in the business) are covered by the scheme’s flagfall.

In such a situation, the supplier – who is responsible for the workload distribution – is also sensitive to it, naturally promoting operational excellence. Those schemes are created over a simple idea: sharing, with the company in charge of the trucks’ management, the core cost drivers of the business.

4.1 Conclusion

Currently, two mindsets are opposed, with the traditional one being the one most represented. In this mindset, deliveries are perceived as goods that can be bought, like regular consumers goods: one delivery has a defined price (depending on several parameters), whichever the way it is managed. This mindset, by its very own nature, does not consider the planning in its costs calculations. Hence, the benefits accessible through optimization are incomplete: yes, the level of service provided to the client will increase, and the flexibility as well as the operational visibility will improve, but the potential financial profits are totally overlooked or must be recovered in tedious negotiations at a later date by offsetting any increase in haulage rates to reflect increased haulier costs with the improved efficiency that has become the new norm for the haulier [4].

The other mindset, based on costs analysis, considers the deliveries made by the fleet as a service. The delivery is rewarded for the effective work needed when considered with all other deliveries carried out that day. With this kind of mindset only will the supplier benefit from every aforementioned advantage brought by operational optimization in their relation with their haulier, notably the critical profit share and scheme simplicity, by sharing the same cost drivers through an appropriate pricing scheme. Logistics companies have seen a growing attention around those since they play an important role in the management decisions taken [6], and sharing common cost drivers will help them to have the same goals and objectives than their customers in order to support a common strategy of costs reduction in their new business relationship.

REFERENCES

[1] Bokor, Z., ‘Elaborating cost and performance management methods in transport’, *PROMET - Traffic&Transportation* **21** (3) (2009) 217-224.

[2] Markovits-Somogyi, R., ‘Efficiency in transport logistics’, *www.impgroup.org* (2011).

[3] Langley, C.J., Morton, J., Wereldsma, D., Swaminathan, S., Murphy, J., Deakins, T.A., Hoemmken, S., Baier, J.M., *Third-Party Logistics Annual Study: The State of Logistics Outsourcing* **13** (2008).

[4] Garrison, R.H., Noreen, E.W. and Brewer, P.C., 'Managerial accounting' **11** (2006) 83-228.
[5] Lukassen, P.J.H., Wallenburg, C.M., , *Pricing third-party logistics services: integrating insights from the logistics and industrial services literature* **49** (2010).
[6] Bokor Z., 'Cost drivers in transport and logistics', *Periodica Polytechnica Transportation Engineering* **38** (1) (2010) 13-17.

COMMUNICATION CAMPAIGN FOR VOCATIONAL OCCUPATION

Olaf Assbrock (1) and Michael Buchmann (2)

(1) Managing Director of the German Ready Mixed Concrete Association (BTB)

(2) Marketing and Public Relations Officer of BTB

Abstract

The new job of “Process Technician for mineral construction products, specializing in Ready-Mixed Concrete” was brought into being in the 1990s by the German Ready-Mixed Concrete Association (BTB). It is the very thing needed for the “War for Talent” characterising the German vocational occupation market. However, having an occupation available is not enough – it also has to be marketed. According to a study commissioned by the BTB, apart from the demographic transition and overall shrinking number of 'Haupt- and Realschule' school-leavers, it seems the lack of suitable applicants is due to the fact that only few are aware of this particular occupation. The BTB has started a communication campaign to remedy matters. To get things started several workshops were conducted with trainees. The aim was to find out from them what they like about their profession, how they view it, how they want it to be viewed, what appeals to them about it, and what doesn't appeal to them about it.

Keywords: Recruiting, quality, communication, employees, construction products

1. INTRODUCTION

Many companies in Germany can no longer fill their apprenticeship positions. Thus, the often cited "battle for the best brains" doesn't just refer to academics, but to all personnel areas, including trainees, who are especially important for the future. Particularly problematic, according to the latest studies, is the fact that not only has the number of applicants for apprenticeships declined significantly (see Figure 1), the quality of incoming applications too has decreased according to the companies advertising the vacancies.

As a result, companies in almost all sectors have begun to engage in fierce competition for trainees. They do this by offering substantial financial incentives during training, or perks such as travel abroad or even company cars. Company cars for trainees are naturally not the rule, but such extremes do currently exist.

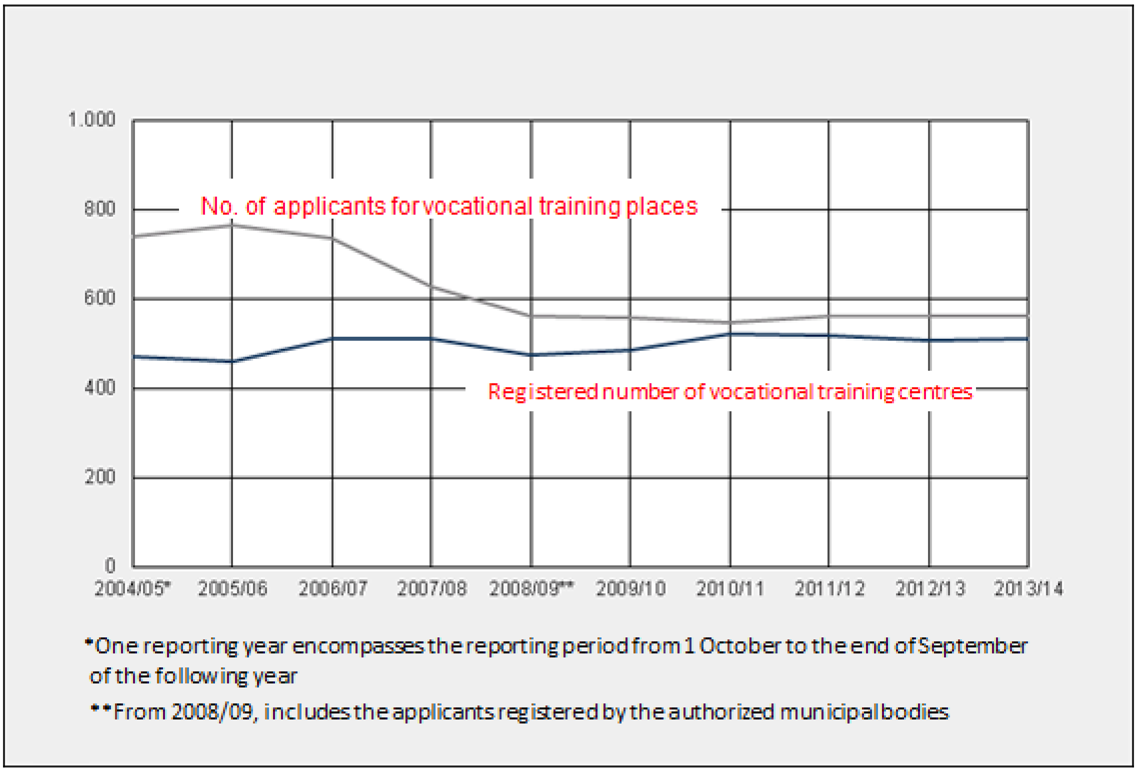


Figure 1: Trend in the number of vocational training centres and the number of candidates for vocational training centres, in thousands. (Source: Federal Employment Agency)

Due to the demographic situation in Germany, it is likely that this situation will worsen in the coming years. See Figure 2 on the next page.

In order to have a chance of even being noticed in this fiercely competitive situation and to find interesting candidates for their own companies, organizations need to communicate and raise their own profiles and that of their training occupations. In addition to the activities carried out by companies, the industry associations too have a duty to provide active support. The German Ready-Mixed Concrete Association (Bundesverband der Deutschen Transportbetonindustrie - BTB) has for years been attempting to raise the profile and promote the trainee occupation "Process Technician for Ready-Mixed Concrete" in order to win over high-quality trainees for the industry. For the past two years, these activities have been intensified.

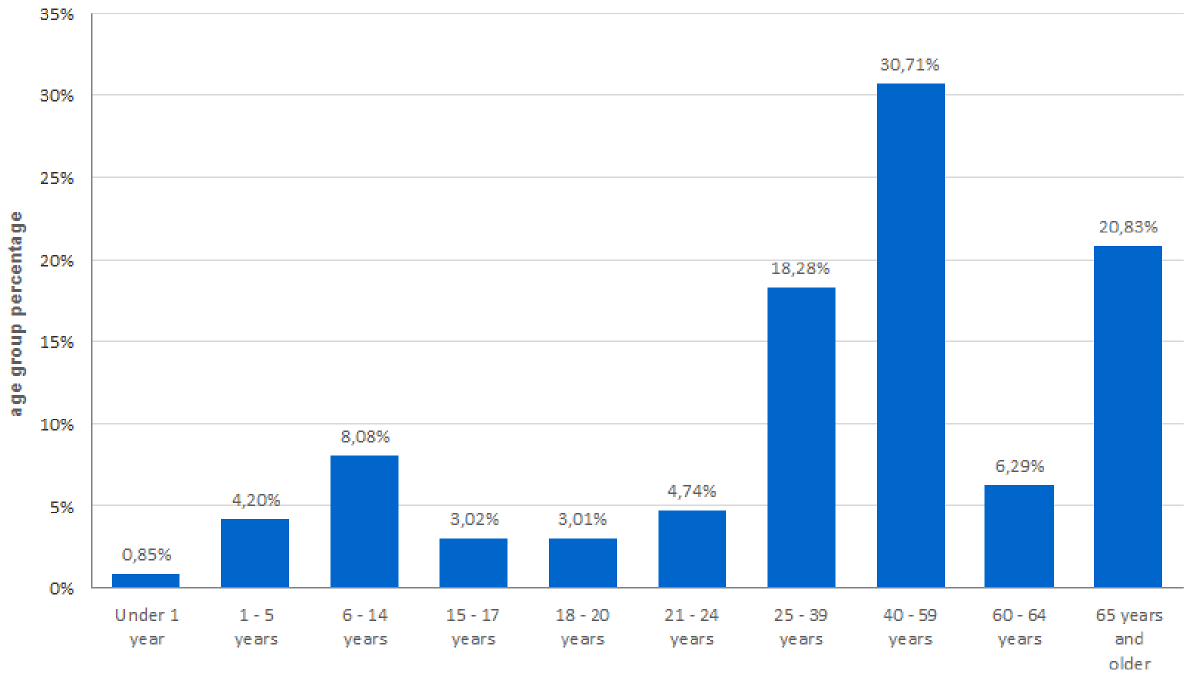


Figure 2: Distribution of the German population by age group on 31.12 2013
(Source: Statista)

2. THE BTB CAMPAIGN

2.1 Analysis through study and workshops

A survey commissioned by the BTB among industry-association member ready-mixed concrete companies has revealed that the training situation is perceived very differently depending on the company size and region. Key reasons for the lack of suitable candidates include the perceived low profile of the profession as well as demographic change and the overall decline in the numbers of secondary-school leavers. In Germany "Process Technician for Ready-Mixed Concrete" is a recognised occupation requiring formal training. Trainees for apprenticeships are often obtained through word of mouth and personal contacts. If no such contacts exist, or if a company lacks experience in appointing trainees, this can be a problem. The relatively new profession of "Process Technician for Ready-Mixed Concrete" is a specialization within the building materials sector, and thus represents only a relatively small number of training places nationwide. Advisers in employment offices and chambers of industry and commerce accordingly rarely seem to draw attention to this apprenticeship in the mineral construction products industry. This changes when firms actively point out the need for such apprentices in the region. For the BTB, this is an important result of the study, and is our mandate for supporting the training needs of our members with various communication tools and measures.

Following the study, several workshops were conducted with trainees. The aim was to find out from them what they like about their profession, how they view it, how they want it to be viewed, what appeals to them about it, and what doesn't appeal to them about it. Special attention was paid to the language and imagery that appealed to the young trainees, which were later incorporated into the used communication tools.

Figure 3: One of the workshops carried out by the BTB in the vocational training colleges

2.2 The basic measures - brochure, website, film, media relations

"I want a career that incorporates a large number of different elements: working on the computer and on machinery, office and manual work, electronics and mechanics, indoor and outdoor work - a job for an 'all-rounder!'". This and other quotes are included in the BTB's new brochure informing prospective trainees about the training occupation. It was developed on the basis of the workshop together with the workshop participants. The tone and imagery reflects their desire for it (and all the following measures) to be objective and serious. Supposed "youth-speak", "comic strips" and "gimmicky phrases" have all been avoided completely. The first print-run of 15,000 copies has been sent to all the association's members, all chambers of industry and commerce, and all career guidance centres in Germany, with more copies able to be ordered if needed.

The contents of the brochure can also be found on the Internet. The Training section of the BTB website, transportbeton.org, has been significantly expanded using the contents of the brochure. The pages with information about the profession can also be accessed directly via the domain www.verfahrensmechaniker.de. In a second step, work has begun on providing new material to other websites. Here, in addition to creating corresponding pages in Wikipedia, the focus is primarily on the Federal Employment Agency (Berufenet.de). The Federal Employment Agency has also agreed to consider the BTB's proposals for revising its pages.

In September 2014, the BTB released a film on the profession. This was produced in consultation with the Federal Employment Agency for Berufe.TV, the Federal Agency's video portal. Two trainees from the group of workshop participants guide the audience through a ready-mixed concrete plant and describe the activities they typically carry out there. The film is also available on Youtube (http://bit.ly/btb-film) and can be integrated by association members into their own corporate websites.

The featured content has also been used to shape internal communications on the need for increased promotion of the training occupation by companies. Another component in the communication mix is press work. In 2014 there were more than 40 publications in regional newspapers with a total circulation of over 4 million nationwide.

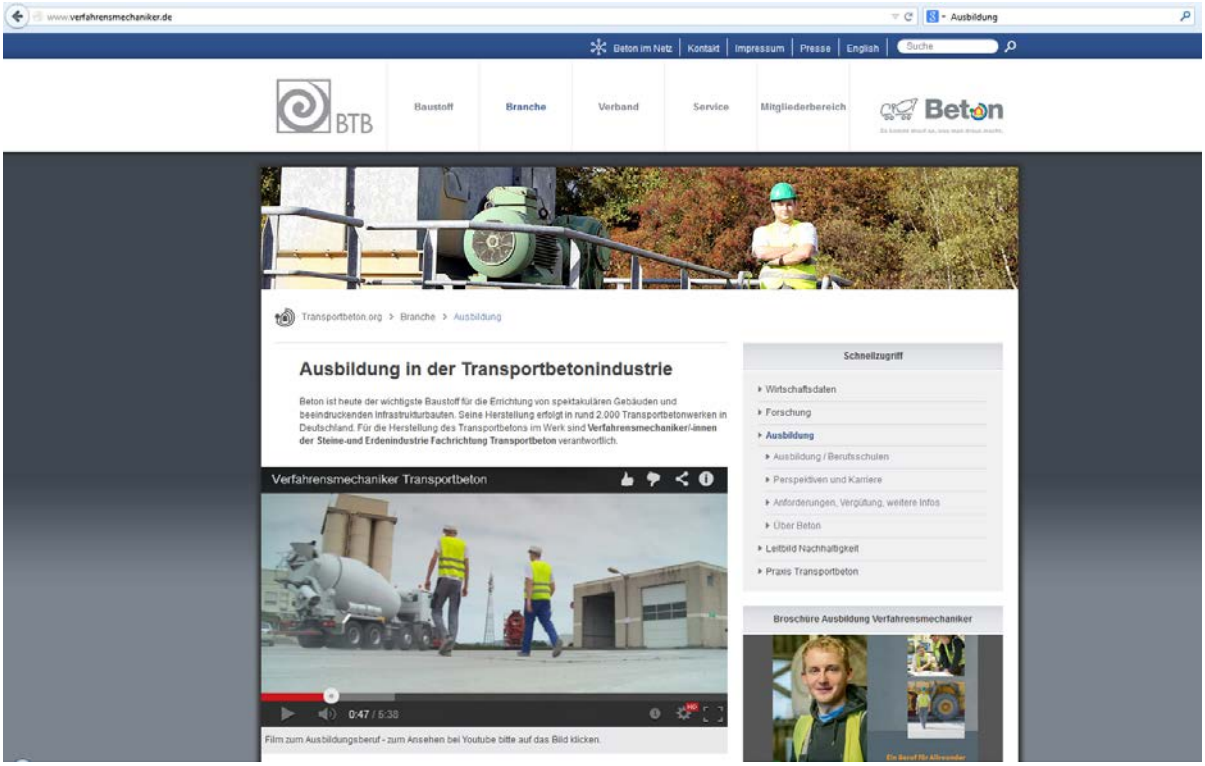


Figure 4: The BTB's three basic materials at a glance: The new website www.verfahrensmechaniker.de includes both the Youtube film and the brochure (image - right column), which can be ordered free-of-charge or downloaded as a PDF.

2.3 Parallel activities - the BTB learning platform

Persuading potential trainees to join the industry through communication measures is important, but it can only succeed in the long term if the actual training conditions are good and the young people feel that they belong to a company that appreciates them and are working in an industry that takes training seriously.

To help ensure the latter, the BTB has alongside its communication campaign totally revised its teaching materials about work in a ready-mixed concrete plant, and incorporated them into an online learning platform. It is based on a training paper handbook BTB published in the 1990´s. The new BTB Learning Platform is part of a knowledge network called "Cement, Lime, Concrete" (www.elearning-vdz.de), whose development is being supported with research funds from the Federal Ministry of Education and Research (BMBF) and the European Social Fund (ESF). The aim of the project is to use Web 2.0 technologies in vocational training in the mineral construction products industry. The knowledge network provides trainees with an online platform with precisely-constructed training modules, including visualizations, self-tests and digressions with additional information.

3. NEXT STEPS: REPETITION, EXPANSION, GUIDELINE

In 2014 BTB has created a very good basis for strengthening the communication of information regarding its training occupation. Now and in the near future, what is needed is for the best possible use to be made of the developed material, for it to be distributed further, and for it to be adapted and supplemented if necessary.

In 2015 and 2016 BTB plans to send out the brochure again to the entire mailing list to replace used copies and possibly reach new contacts in offices.

Besides Berufe.TV, the career-related video portal most frequented by young people, the BTB now plans to offer the material to other media and professional services, so that it is distributed as widely as possible on the Internet.

Other media should be developed from the existing material. Posters and roll-ups are currently being prepared that will be suitable for use by companies at career fairs and during school visits.

The main instrument currently being prepared is a guideline for companies, aimed at medium-sized companies in particular, informing them what needs to be kept in mind when becoming a training company, and in particular, how to find the right apprentices for the company.

CONCRETE: VITAL ASSET TO OUR LOCAL ECONOMIES: A COMMON ACTION OF CEMENT, PRECAST CONCRETE, READY-MIX CONCRETE AND ADMIXTURE ASSOCIATIONS

Alain Camus

Cemex - SNBPE, France

Abstract

In the field of its action of communication "concrete trump cards of our town and country planning" pushed in direction of the French elected representatives and technical services of local administrations, we are together, 100,000 employees in the concrete industry, so many ambassadors of the image of our material, a material with several uses, a chameleon material, clever and innovative but which sometimes we get some difficulties to speak about it, because it is multifaceted and versatile.

To help everyone to prepare an interview, a presentation, a speech, we wanted to gather summarizes all the issues faced by actors in sustainable land development and the answers we can manage with precision to solve their problems with our concrete solutions.

"Living better, moving better, protecting better the planet thanks to solutions provided by an industry fixed firmly in the countries near by the resources and needs, true model of circular economy " : that is the positioning which we should be proud.

The SNBPE hopes for to explain its strategy an present its mediums purposed to the intern and extern targets when it is starting its new campaign.

1 INTRODUCTION

Concrete industry has to go with a new scope when the markets, the society, the networks are moving: a leader material cannot lag behind.

2. WHO ARE THE ACTORS OF THE CHANGE? THE AUTHORITIES PREDOMINANCE

We have had to define by whom and where in the process chain decisions are taken directly or indirectly into the choice of the materials and construction solutions and we have had to understand the behaviour changes:

- Construction companies – direct customers of concrete industry but who are searching for technical and economical solutions
- Architects, urbanists, town planners, economists: who are looking for technical solutions but also interesting by image and sustainable development
- Technical departments of the local authorities who are searching for technical, sustainable and economical solutions
- Elected representatives without a keen interest in technical matters but interested by image
- Public opinion who are influencing elected representatives in their positions.

The objective for our industry is to speak to each actor according to his needs with a single voice, to find right words and to get visibility.

2.1 Elected representatives & technical departments: priority targets

Our most important conclusion is that the elected representatives and technical departments emerge as the priority targets.

On one hand, a lot of power is delegated to local authorities for example the power of decision or financials means, but know-how has been eroded and items are following by non-specialized people, there are fewer technical experts and a large tendency to subcontract the work.

On the other hand, a political posture overrides technical discourse conditioned by the favourable attitude of the opinion to « natural and ecological » materials than sensitive to their installation ability. Consequences are a high level authorities investment incentives, a temptation to get a very favourable welcome from the ecological tendencies of the electors and any various subsidies from the regional and local powers

Opportunities to provide relevant technical information and to prove our material performances need to be found

2.2 The contradiction between the needs and the solutions

Another issue is the contradiction between the needs and the solution.

The needs of regional and local authorities could be resumed in few items:

- Economic territory management (decrease building and operational costs)
- Control over the urban development (densify and structure land use, collective passengers transport)
- Living environment improvement (clean air, noise, thermal and acoustic comfort, landscaping spaces)
- Environmental protection (water cycle, waste management, biodiversity)

- Developing regional economy (local employment)
- Insure proper social mix,
- Renewable energies production (wind turbine, anaerobic digestion ...).

Frequently the response of local authorities and technical services are wood, bitumen or PVC although the concretes solutions fit every needs.

3. HOW TO MOVE FORWARD AND MANAGE TO CHANGE THE RULES?

We apply common sense to demonstrate the benefits of concrete solutions in the economic development of the territories, its values to society, well-being and the environment.

Thereby we insist on raising awareness that ecological solutions are not necessarily where expected and to evolve from original cost to overall cost because concrete needs less maintenance work (e.g. roads and sanitation systems).

3.1 To speak with a single voice

We aim to speak with a single voice to strengthen and harmonize the statements made by the whole concrete industry by sharing messages and adapting the content for various audiences with the intention of improving efficiency and optimizing the resources available.

Our common message is structured around a matrix.

Traditionally we developed our offer following three sectors: building, civil engineering, and roads and utilities. Today, with our new approach, we choose to present our construction solutions according to three spheres: enhanced living, enhanced mobility and enhancing the preservation of the Earth.

To bring our messages to the identified audience, we forecast a 3 year communication campaign supported by the whole concrete industry and we expect to turn the 100 000 employees of our industry into ambassadors

3.2 Concrete, an effective partner of the circular economy

First item: concrete is an effective partner of circular economy.

So, we can confirm it, because we are able to reduce the use of natural resources by resorting, recovery and recycling materials.

Moreover 3 Mt by-products of other industries are used in cement kilns and 80% of concrete from demolition is recovered or recycled.

Consequently, the concrete industry is one of the most explicit examples of the Circular Economy with a real regional anchoring and a prudent management of natural resources; we can prove it and assure that concrete provides sustainable building solutions for the local authorities.

I can add that in France the local presence of our industry is valuable for the local economic activity with 4500 plans, 100 000 local jobs that cannot be outsourced, a million of local customers and strongly reduced transport impacts.

3.3 Enhanced living

Second item: enhanced living

With concrete you can live, work, relax, to study in your local area. Concrete options provide technical solutions to accompany building and city developments:

- safer with buildings as concrete solutions are fire-resistant, earthquake-resilient and storm-proof, suffering minimal damages in case of heavy rainfall or flooding and resistant to fungi and insects;
- more beautiful with all kinds of architectural possibilities, integration to landscape and local architecture and contribution to green and leisure facilities.

To add to the list of the concrete qualities we have the contribution to a comfortable area with a really ability to reduce noise pollution, the ability to have plants on urban roofs and walls, improvement air quality, contribution to thermal comfort, reduction of heat islands, accessibility for all and reduction of disturbances due to infrastructure works.

To conclude this item, we shall insist on three points: concrete is the best solution for building and city developments of the future characterized by the densification of the cities, upgradable buildings and the layout of the underground space in order to release space on the ground.

3.4 Enhanced mobility

The third item: enhanced mobility

The aim is to improve the movement of goods, people and fluids with the purpose of promoting economic development in the territories and making travelling safer and more pleasant.

To achieve these objectives, concrete is the natural choice for roads, bridges and tunnels that provide lasting connections for territories, underground networks and high-speed rail network, maritime and river development schemes and rural areas development; but concrete contribute also to make travel safer and more pleasant structuring urban space and improvement of its readability, making transport toward leisure places easier and improving road safety.

3.5 Fourth item: enhance earth’s preservation

The objectives are to limit the environmental footprint of our fellow countryman, participate in the reduction of energy and resources consumption and contribute to flora and fauna protection. Concrete offers many solutions for these objectives.

It is enough to take some examples in the field of the societal impact on the natural environment to note that concrete is the best material for water cycle constructions, the constructions adapted for waste treatment, large volume potable water transport, renewable energy and serving biodiversity and fauna .

Working concrete reduces the consumption of natural resources and energy and preserves natural resources with soil treatment and in-situ pavement treatment. For example, whiter pavements reduce public lightening and procure the advantages of “albedo effect”, rigid pavements reduce vehicles fuel consumption, well designed concrete buildings consume less energy and cities are more compact with a better accessibility reducing urban travel.

4. HOW TO DELIVER THESE MESSAGES?

To communicate to the market, SNBPE implemented this campaign into its own promotional tools by setting up a multimedia library in order to provide its members with a practical promotional website.

The layout of the multimedia library is based on the framework document: circular economy, enhance living, enhance mobility and enhance the preservation of earth.

For each subject, the multimedia library features the whole documents (technical guidance documents, guidelines, slideshows, trade publications) published by cement, ready-mix, concrete, pumping concrete and admixture associations and useful links which redirect to standards and our partner issues.

How to deliver these messages?

- To internal targets we presented our tools to the Boards of concrete associations and during commercial meetings. We took part also in trainings for managers. To help our internal targets various position papers are provided.

Now our challenge is to be sure that the tools are displayed and “sold” through the companies.

- To contact external targets we have at our disposal presentations (PPT) for elected representatives, dedicated papers on each subject, a film, a video animation and a multimedia library.

Everyday management and commercial workers of our members companies can develop our items and use them in their commercial actions and the teams of the associations (Cimbéton-SNBPE-FIB-SYNAD) have the use of the same in the field of the Institutional communications.

INCREASING THE TOTAL WEIGHT OF FOUR-AXLE VEHICLES: IMPACT ON INFRASTRUCTURE, ENVIRONMENT AND ECONOMY

Thomas Hoffmann

Assistant of Economic Policy at the German Ready-Mixed Concrete Association (BTB)

Abstract

This paper analyses the impact of four-axle vehicles with two twin axles and a total weight of 35 tons while maintaining the maximum permitted weight per axle of 9.5 tons on the infrastructure, environment and economy. The evaluation of two related studies proves that the modified truck mixer has no negative impact on the life span, safety and durability of bridge structures and that negative impacts on road pavements are, if existing, overcompensated by the enhanced transport efficiency. Further arguments regarding transport efficiency, modernization of the transport fleet, fuel savings, road safety and environmental friendliness are given.

Keywords: Truck mixer, weights, dimensions, directive 96/53/EC, legislation, transport efficiency, environment, fuel savings, CO_2-emissions, road safety

1. INTRODUCTION

The provisions on permitted weights and dimensions of heavy-duty vehicles are regulated in the EU-Directive 96/53/EC. Concerning four-axle vehicles with two twin axles, the directive still contains specifications established in the 1980s. Since then there have been many technical improvements and innovations in vehicle construction and safety, which allow for more environmental friendliness and transport efficiency if considered in a revision of the EU-Directive 96/53/EC. To increase the efficiency of four-axle vehicles with two twin axles, the European Organization of Ready-Mixed Concrete (ERMCO) proposes a modest increase of the maximum permitted weight of four-axle vehicles with two twin axles from currently 32 to 35 tons, while maintaining the maximum load per axle of 9.5 tons. Maintaining the maximum load per axle requires a minor modification of the vehicles, which only comprises the broadening of the wheelbase from the second to the third axle by 30 cm (see Figure 1). The modification allows for an improved weight distribution and better utilizes the permitted weights on the front axles (front axles from 13.6 to 16.0 tons, rear axles from 18.4 to 19.0 tons).

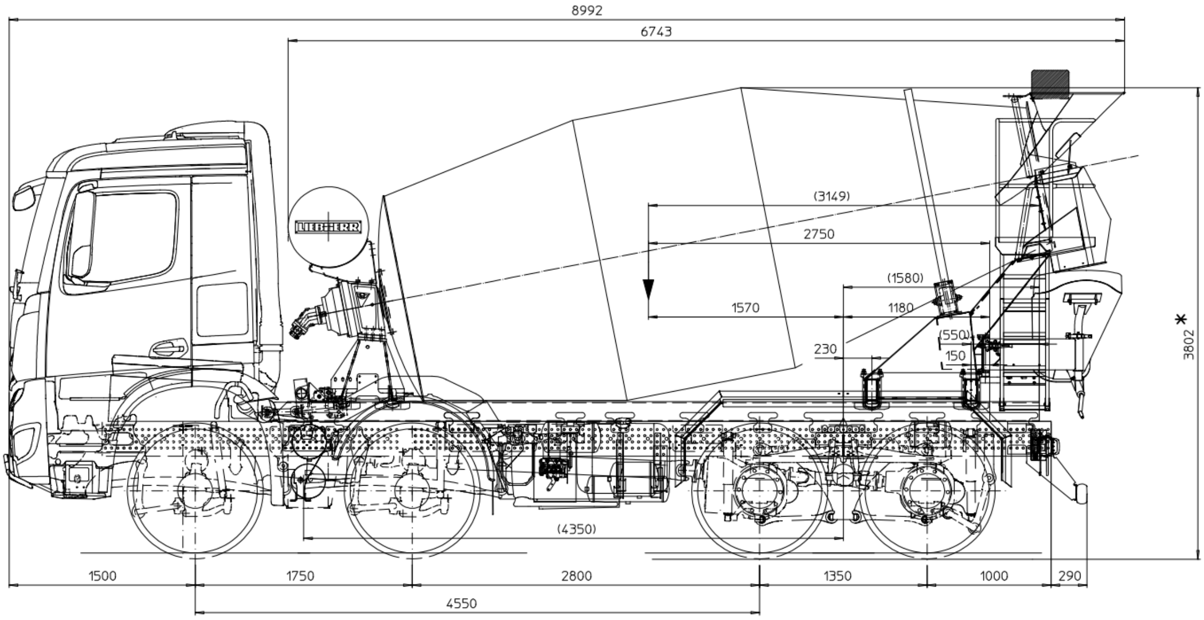


Figure 1: Model of a 35t four-axle truck mixer with modified wheelbase [1]

Numerous national associations such as the German Ready-Mixed Concrete Association (BTB) support the proposal of ERMCO. However, the European Parliament rejected the consideration of the proposal in the revision process of the EU-directive, as sensitive discussions about the negative impact of higher weights on both road safety and the condition of the European infrastructure, especially bridge structures, emerged. Based on the evaluation of related studies this paper analyses the validity of the objections and gives a brief overview about other arguments supporting the ERMCO proposal in ecological and economic means.

2. ASSESSING THE IMPACT ON THE INFRASTRUCTURE

2.1 Impact on bridge structures

In Germany, the temporary closure of some important interstate bridges recently caused a broad discussion about the condition of the German infrastructure. With respect to the ERMCO proposal, the German Ready-Mixed Concrete Association (BTB) decided to commission a study investigating the impact of the additional weight on existing bridge structures. The study was conducted by the engineering office Curbach/Bösche under supervision of Prof. Dr. Dr. h. c. Manfred Curbach, Director of the Institute for Solid Construction at the Dresden University of Technology [2]. The study compares already permitted truck mixer models with the proposed 35 tons four-axle vehicle and examines whether the vehicles are below the assessed limits of the bridge structures. Based on an analysis of the existing bridge infrastructure in Germany, the study examines the bridge classes BK 60 (built until 1985), BK 60/30 and BK 30/30 (both since 1985) as well as LM 1 (since 2003) as the most important in Germany. The bridge class indicates the maximum load for which the bridge is designed (BK 60, BK 60/30 and LM 1 mainly represent interstate and national bridges, BK 30/30 mainly rural and municipal bridges). Figure 2 shows the normative load model of the bridge classes BK 30/30 (SLW30), BK 60, BK 60/30 and LM 1 (SLW60) as well as the three investigated truck mixer models transformed into the comparative load models “32 tons” (permitted four-axle vehicle), “38 tons” (permitted four-axle truck-and-trailer model) and “35 tons” (proposed four-axle vehicle).

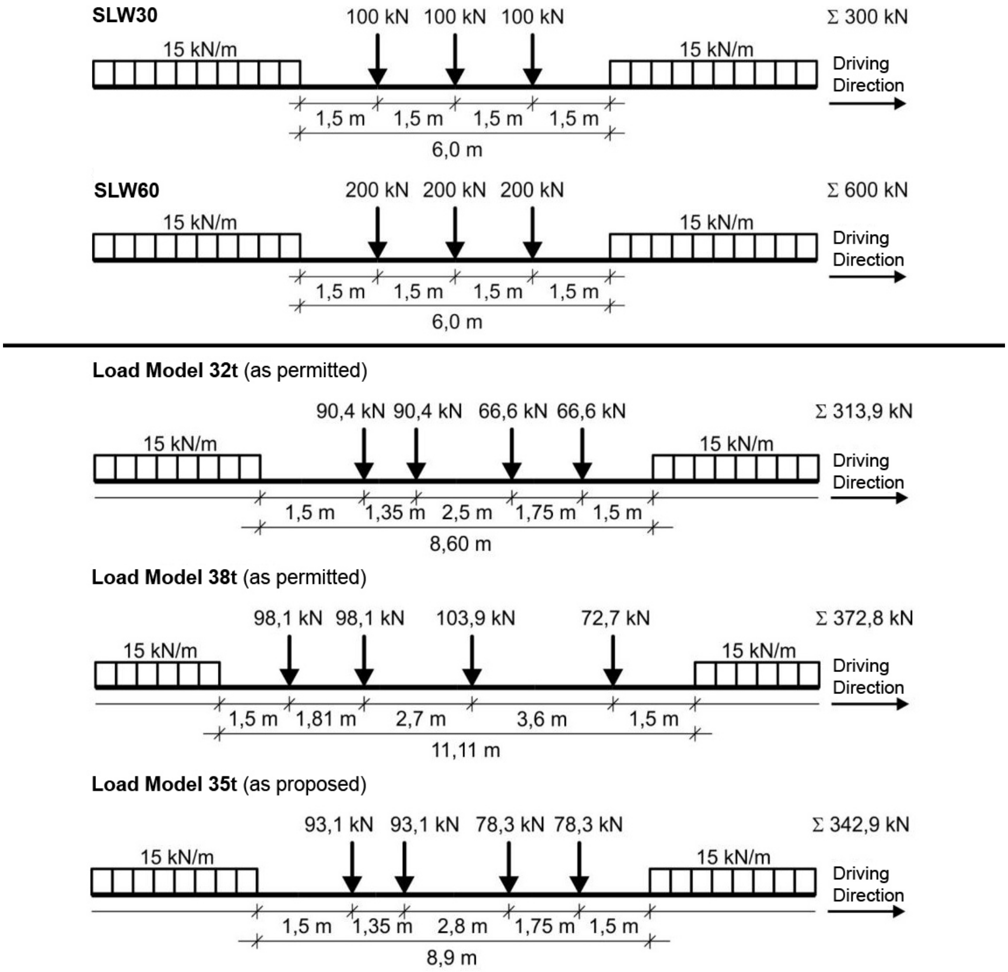


Figure 2: Normative load models of bridge structures and truck mixers [3]

The study clearly shows that the additional three tons are not relevant for bridge structures of the classes BK 60, BK 60/30 and LM 1, as the assessment of these classes already incorporates a significantly higher load. Hence, a derivation of negative impacts during the life span of these bridge structures due to the proposed 35 tons vehicle is inadmissible. Solely BK 30/30 bridge structures, which amount to less than 5 % of the national distance road network in Germany, require a further examination. For this reason, the following statements refer to the bridge class BK 30/30 only.

During the separate examination of the global longitudinal forces and the lateral forces of load bearing systems, different systems with variant span widths of the bridge segments as well as different cross-sections with variant dimensions are considered. As the modification of the vehicle's wheelbase lowers the impact on the load per axle of the vehicle, the statistical load of the bridges increases only marginally compared to the already permitted vehicles. Compared to the load in consequence of normative load models (bridge classes), the modified vehicle remains in almost every case below the assessed limit. Thus, the safety and load capacity of bridge structures assessed according to bridge class BK 30/30 is sufficient. In few cases, the traffic load marginally exceeds the limits of the longitudinal forces. However, the study considers the violation of the limit as negligible as it only amounts to a maximum of

2 % of all cases. Moreover, if all loads of the system instead of the traffic load only are considered (e.g. dead load and wind), the violation is even lowered. Further investigations regarding the decisive dimensioning of bridge structures, such as the fatigue and punching of structures, reveal no violation of the limits caused by the modified vehicle. Concerning the compliance with admissible frictions at service load levels, likewise the examination of the global longitudinal and lateral forces, marginal higher loads of the modified vehicles are possible. However, analogous to the longitudinal forces, they do not exceed a negligible level and are further lowered if all loads effecting the structure are considered.

In summary, the findings lead to the conclusion that even bridge structures assessed according to bridge class BK 30/30 are able to bear the additional load of the modified four-axle vehicle. The bridges will remain sustainable and safe. A negative impact during the life span of bridge structure is inadmissible. The study shows that a modest increase of the maximum permitted weight of modified four-axle vehicles with two twin axles while maintaining the maximum weight per axle has no negative impact on the existing bridge infrastructure in Germany. This is even more relevant for newer bridges designed according to the bridge class LM 1 (EN 1991-2), as they allow for even higher weights as suggested by the ERMCO proposal.

2.2 Impact on road pavements

A similar study, commissioned by the Austrian Ready-Mixed Concrete Association (GVTB) and conducted by the Vienna University of Technology, examines the impact of four-axle vehicles under consideration of different weight scenarios on road pavements [4]. Whereas the ERMCO proposal suggests an increase of the weight to 35 tons without exceeding the maximum permitted weight per axle of 9.5 tons, the Austrian study compares the existing 32 tons four-axle vehicle with three different scenarios in which the weight increases linearly to 36, 39 and 41 tons. As the wheelbase of the vehicle remains unchanged, all three scenarios exceed the maximum permitted weight per axle. This paper focuses on the 36 tons scenario, as it is most similar to the suggested 35 tons model.

Under consideration of the average load per truck mixer and the average distance to the building site, the study computes a decrease of mileage by 15.4 % in the 36 tons scenario. This leads to a decrease in fuel consumption and CO_2-emissions by 5.4 % (numbers do not match as the fuel consumption for loading and unloading remains constant). Assuming a Diesel price of € 1.40 per liter, the fuel savings in the Austrian ready-mixed concrete industry amount to € 1.74 million per year (data base 2007-2010).

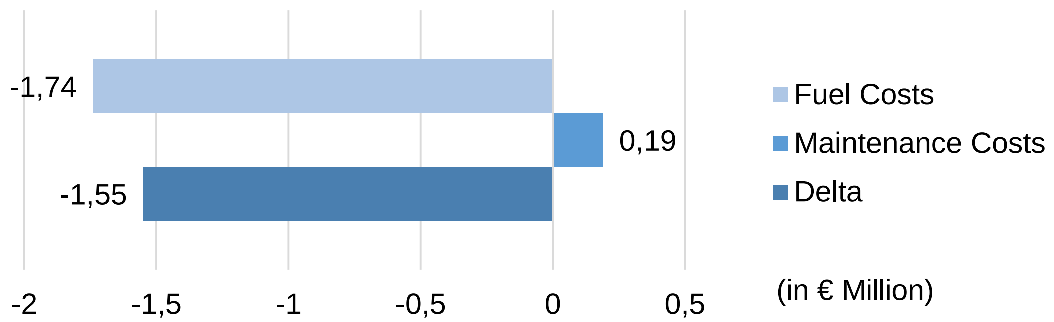


Figure 3: Cost projection of the 36t four-axle truck mixer scenario in Austria [5]

On the other side, the impact of the additional weight would reduce the technical life span of road pavements by -0.3 %. Evaluating the average costs for road pavements in Austria, this would increase the annual maintenance costs by 0.06 % or € 0.19 million. As shown in Figure 3, the additional costs for maintenance are clearly overcompensated by the savings of the enhanced transport efficiency, leading to positive net effects for the national economy. These positive effects would even be furthered considering a 35 tons truck mixer as suggested by ERMCO, which does not exceed the maximum weight per axle, thus, lowers the negative impact on road pavements (as shown in Section 2.1).

3. BENEFITS OF TRANSPORT EFFICIENCY

3.1 Impact on traffic and road safety

In 2013, the total ready-mixed concrete production of ERMCO members in the European Union amounted to 217.7 million m³ [6]. Considering the maximum load per truck mixer of 7.5 m³, this implies a required number of turns for delivery of approximately 29.0 million. With an average distance to the building site of 20 km, truck mixers in the European Union cover a total distance of approximately 1.16 billion km per year. The allowance for an additional weight of 3 tons raises the capacity of truck mixers to 8.5 m³, lowering the required number of turns for delivery by 12 % to approximately 25.6 million. Simply put, every eighth turn of a truck mixer is no longer required if the ERMCO proposal is accepted. Especially in traffic systems, which already reach its capacity limits, the projected reduction of the driving performance is of great value: it lowers the traffic density and reduces the burdens on public roads. Accordingly, this also implies benefits for road safety, as “the safest traffic is the traffic which does not exist”.

Besides, the improvements in vehicle construction make sure that the additional weight has no negative impact on safety issues such as the braking distance or stability of the vehicle. Prof. Dr. Ronald Blab from the Vienna University of Technology examined the impact of the additional weight of truck mixers on road safety and proofed that the braking distance, the dynamic axle load distribution, the cornering stability as well as the requirements on restraint systems remain unchanged even if the maximum permitted weight increases to 36 tons [7]. Further objections regarding a positive effect of the additional weight on the attractiveness of the road transport in general, leading to more instead of less vehicles on the road, are unfounded. The vast majority of four-axle vehicles is utilized by the building materials industry, which is exclusively driven by demand and the actual building activity – nobody will pursue a new building project just because the costs of transport decrease by a few percent.

3.2 Ecological benefits

Naturally, the mileage reduction also carries some important advantages for the environment. A truck mixer roughly consumes 35 liters of diesel per 100 km. Considering the projected saving of 3.5 million turns or 140 million km per year (this equals 3,500 turns around the earth), the reduction of the annual fuel consumption in the European ready-mixed concrete industry amounts to almost 50 million liters per year. According to EN 16258, one liter of Diesel equals a CO_2-emission of 3.15 kg [8]. Thus, increasing the weight of four-axle vehicles from 32 to 35 tons allows for a CO_2 reduction of more than 150,000 tons per year – in the European ready-mixed concrete industry only. Concerning the goal of the European Union to reduce the CO_2 emissions by 40 % until 2030, the ERMCO proposal provides a simple but effective opportunity to support this undertaking.

3.3 Economic benefits

Finally yet importantly, the advances in transport efficiency achieved by a modest increase of the maximum permitted weight of four-axle vehicles allows for some benefits on the entrepreneurial side. In an industry drawn by intense competition and regional overcapacity the reduction of transportation costs and turnaround times provides the concrete companies with opportunities for a better utilization of the existing concrete plants, truck mixer fleet and staff. Furthermore, an increase of the maximum permitted weight would result in incentives to replace the existing, partly outdated transport fleet with more modern, fuel-efficient and environment friendly EURO-6 vehicles, leading to a higher demand in the truck construction industry and a generally more modern and efficient transport fleet. Lastly, some regional markets such as Germany recently experienced a shortage in available truck driver staff. The reduced driving performance could help to lower the consequences of such shortages in the particular areas.

4. SUMMARY AND OUTLOOK

The studies investigating the impact of four-axle truck mixers with a total weight of 35 tons proved that (1) a negative impact on the life span, safety and durability of bridge structures is inadmissible and (2) that the impact on road pavements is clearly overcompensated by the improvements in transport efficiency. Additionally, the maintenance of the maximum permitted weight per axle of 9.5 tons (as suggested by ERMCO) further lowers or even neutralizes the negative impact on road pavements. Lastly, the decrease of traffic density and improvements in vehicle construction encounter objections regarding the road safety of the proposed vehicle. In summary, the benefits of the ERMCO proposal regarding transport efficiency, modernization of the transport fleet, fuel and cost savings as well as environmental friendliness are free of counter arguments and thus deserve a second chance in the international and national parliaments.

Unfortunately, the recent ERMCO initiative at the European level was not successful. Despite all arguments, the increase of weights in the international traffic is a very difficult and sensitive topic. However, national associations are encouraged to aspire an exception at the national level as foreseen by Council Directive 96/53/EC (12). The evaluation of the two studies and the collection of arguments might help to convince the national parliaments of the suggested proposal. Some countries, such as the Netherlands, already make use of such weight exemptions with benefits for both the environment and the economy.

REFERENCES

[1] Liebherr Mischtechnik GmbH (2014): Project drawing of a 35 tons four-axle truck mixer, Drawing No. 9494 213 05.

[2] Ritter, Robert; Bösche, Thomas (2014): Modifizierte vierachsige Straßenfahrzeuge mit zwei Doppelachsen – Auswirkungen auf die Beanspruchung von Brückenbauwerken, Study, Curbach Bösche Ingenieurpartner, Project Number G0026/01.

[3] Ibid., pp. 11 and 16.

[4] Blab, Ronald; Kluger-Eigl, Wolfgang (2012): Technische und ökologische Auswirkungen von Achslasterhöhungen von 4-Achsfahrmischern, Study, Vienna University of Technology, Project Number 11510.
[5] Ibid., p. 20.
[6] ERMCO (2014), Ready-Mixed Concrete Industry Statistics, p. 3.
[7] Blab, Ronald (2015): Verkehrssicherheit von 4-Achsfahrmischern mit einem höchstzulässigen Gesamtgewicht von 36 Tonnen, Advisory Opinion, Vienna University of Technology.
[8] Deutscher Speditions- und Logistik Verband e. V. (2013), Berechnung von Treibhausgasemissionen in Spedition und Logistik gemäß DIN EN 16258, Guideline, p. 12.

INDUSTRIALISATION OF CONCRETE INNOVATIONS IN LATIN AMERICA AND THE CARIBBEAN

Diaz Jorge C. (1), Zampini D. (1)

(1) Cemex Research Group AG, Switzerland

Abstract

In a geographical region represented by more than 20 nations and counting a population of almost 600 million, Latin America and the Caribbean represents the regions with the greatest growth, especially when referring to urbanization - approximately 80% of its population is living in cities. Its wide range of economic segments such as agriculture, minerals and oil, manufactory and services such as tourism and transport has increased infrastructure and housing investments in a manner that the construction industry is challenged to offer solutions to be faster as well as of increase quality. Consequently, cement based materials such as concrete will continue to be the most important material in construction industry. Meanwhile, it should be noted that recently concrete technology has undergone important advances as a result of important research and development activities. Nevertheless, many of the cutting-edge technologies are in most of the cases limited to laboratory and prototype production, and never industrialised massively in the marketplace. One of the main challenges to go from laboratory to the construction site is easy and practical industrialisation. The concrete industry represented in this report reflects the experience of CEMEX as construction materials company that has been able to demonstrate that important innovations can not only be developed efficiently and effectively , but more importantly they can be adopted into the marketplace rapidly, affordably and above all can be produced everywhere in at a massive scale. This success showcased by concrete has not only been the impetus to help the construction industry overcome its challenges, but has also been accompanied by initiatives of relevant social impact, resulting in the offering of solutions that have been able to equilibrate the socio-economic fabric of communities that initially manifested inequality.

1. INTRODUCTION

Although technically speaking Latin America and the Caribbean (Latam) belongs to the so called “third world” or “emerging economies” which is small compared to the main group of economies as US, Western Europe or Japan, its inside dynamics and diversity brings an exciting potential of growth for the upcoming years despite the fact that every country has to confront a lot of challenges, and must fashion its own response based on its own peculiarity in order to achieve a sustainable growth.

One of the best expressions of this potential is seen among its cities that today represents almost the 80% of its population, giving the real sense to be the most urbanised region in the world. In 1930, Latam had just over 100 million inhabitants. Today the region is over 500 million living in cities; it has an urban/rural ratio similar to that in the industrialised countries. Some countries are particularly high like Argentina (89.8%), the Bahamas (88.5%), Uruguay (91.3%) and Venezuela (86.9%). Moreover, metro areas of Buenos Aires, São Paulo, Rio de Janeiro, Mexico City and Lima are already among the 30 largest in the world. The region is expected to have an urban population of 665 million (81% of total) by 2020. Urbanization footprint, with Brazil being a distinguishable exception, typically involves a single very large city per country, for example Lima’s metropolitan area has over 7.4 million inhabitants which represents almost the 30 percent of the total population in Peru. The second largest city is Arequipa with only 700,000 inhabitants. Today more than 50 cities have surpassed the one million inhabitants, of which 14 are in Brazil alone, and represent the current trend of growth and push back of the megacities.

Despite this economic growth, the weakness of the political and social situation has brought difficulties such as poverty and inequalities which are concentrated for example in urban areas like Mexico City or São Paulo with roughly 40 percent of people living below the poverty level. This means that most of the poor urban dwellers live in substandard housing within informal settlements and with limited or no access to basic services.

All this environment brings a real challenge for planners and professional sectors like the construction materials industry by raising the requirements for higher quality products, but yet affordable in order to enable the construction of massive projects like low income housing, water management or transportation (like BRT’s - Bus Rapid Transit) among others that are intended to support properly population growth. In many construction projects the material costs can represent a big portion of the budget, the innovation plays an important role in order to be able to come up with ideas that can be achievable with local materials, but more importantly accessible, meaning being able to be industrialized elsewhere.

Despite the conservative nature of the construction industry, as demonstrated by only incremental changes and evolution of concrete technology – e.g. automation of batching systems, increase in truck sizes, admixtures developments, advances in pumping technologies and improvements in logistical coordination, there are experiences that show progress around the region in the use of newer technologies.

The Ready-Mix Concrete industry Latam shows an annual production of 120 million of cubic meters, nevertheless its penetration of the so called “special products,” which can represent the latest technologies is barely reaching 10 to 15% among the region, nevertheless big cities can show numbers over 40% of its total production related special products. The normal housing in many cases represents more than 80% of the consumption and uses standard concretes, and typically infrastructure projects or rarely high rise buildings are the ones that push for introducing the most challenged concretes into the market.

In order to make a differentiation between “traditional” and “new concrete technologies” the following criteria has been established as a possible classification technique:

Table 1:

Variable	**Dimensión**	**Traditional Concrete**	**New Technology**	**Cutting-Edge Technologies**
Compressive Strength (MPa)	Concrete performance	<= 42 MPa	40-80 MPa	>= 150 MPa
Consistency	Concrete performance	5 to 20 cm slump test	> 20 cm slump test Self Compacting Concrete (SCC >= 60 cm flow)	Maintain SCC or high slump mixes by minimum of 90 min with no loss of workability
Admixtures Technology 1) Rheology 2) Workability 3) Setting time (Accelerate/ Retard)	Mix design	LNS (Lignosulfonates) Sugar Based Naphtalenes	PCE (Policarboxilates) VMF (Viscosity modifiers)	
Fibers Technology	Mix design	Plastic shrinkage control (<=2kg/m3 Polypropilene)	Toughness increase (>=20 kg/m3 steel fibers)	Ductile concrete – Modulus of Elasticity >= 35 GPa. Elastoplastic behavior
Early Strength	Concrete performance		4.0 MPa @ 1d	>=4.0 MPa @ 4h
Shrinkage	Concrete perfomance		Drying shrinkage < 400 microns @ 28d	Non-shrinkage concrete

The examples to be shown as representatives of “new” and “cutting-edge” technologies are referred to a set of solutions that have been developed at the CEMEX Research Centres in

Switzerland and Mexico, and today are deployed in different regions around the world including Latam. The technologies to be described are:

- Hidratium. Technology that renders concrete tolerant to poor curing practices and eliminate the use of an external curing, meaning it print a self-curing property to the concrete. Its performance is achieved through a combination of distinctive mix design principles and proprietary admixtures to eliminate the plastic shrinkage and reduce the long term drying shrinkage. On top of that, the final appearance of the concrete surface shows excellent results for architectural purposes, and gives an extraordinary “anti-dust” property that brings a cleaner job site meanwhile it reduces the cleaning activities and more important for example can be used as final floor for low income housing, which in some cases like Colombia low income housing projects the concrete slab does not have a cover like tiles or carpet. In this case the new dwellers have to live with the concrete surface for a long time meanwhile its affordable a nicer topping. In this case, “anti-dust” properties are important in order to avoid pulmonary diseases which are very common among the poor families. During 2014 for example, in Colombia more than 14.000 m3 of Hidratium concrete (50% out of the total) was poured for building more than 4.000 homes for Low Income Housing using “cast in place” construction system.
  In the case of Mexico, around 1 million of cubic meters with Hidratium technology are spread yearly into the marketplace for different applications were we can extract Low Income Housing projects in the Yucatan Peninsula, each one by building between 2.000 to 8.000 new homes.

Photo 1:Low Income Housing Project in Mexico

Photo 2: Low Income Housing Project in Colombia

- Promptis. Ultra rapid hardening concrete is able to reach early compressive strength in only 4 hours, compared with an average of 18 hours in conventional concrete. Despite its rapid-hardening properties, Promptis® has been designed to retain workability for over 90 minutes, thus allowing the material to be easily handled without the fear of sudden hardening, even under extremely hot weather. Therefore, only a couple of hours after delivery, one is able to remove the formwork, and prepare for the next cast on the same day. Strength develops in a progressive manner as the concrete ages, resulting in a highly durable material that also exhibits a very good resistance to shrinkage cracking. Evolution. A highly flowable concrete that can spread into place under its own weight and achieve excellent consolidation without internal or external vibration.

An interesting example was done in Colombia for a massive low income house project (3,000 homes) were more than 3,400 m3 of Promptis concrete were used out of the 17,300 m3 in total the project consumed during a period of 13 months (2013-2014), this means around 20% of the project. The ultra-rapid hardening technology allowed to recuperate time lost by burocratic paper work that stopped the construction works by allowing to cast twice a day with the same formwork. In this case construction production was increased by 100%.

Photo 3: Low Income Housing Project in Colombia

- Evolution. A highly flowable concrete that can spread into place under its own weight and achieve excellent consolidation without internal or external vibration. Although self-compacting concrete is not new inside the advanced concrete industry, one of the main challenges has been to introduce it massively. An interesting example is the deployment in Dominican Republic were Low Income Housing projects have been transformed from standard concretes (20 cm slump) to SCC, but more interesting being able to match the material cost budget even if using a high tech technology. Final results on the houses have shown excellent finishing walls, very impermeable slabs and diminishing significantly labour costs up to 30%.

Photo 4–5: Self Compacting Concrete applied in Dominican Republic.

Out of these examples, there are more technologies on the roadmap that will keep overcoming the construction industry challenges, as follows:

- Insularis. A 100% ready-mix concrete construction solution for thermal insulation and improving energy efficiency in buildings.
- Resilia. A concrete technology engineered to achieve unprecedented mechanical performance: ultra-high strength & ductility.
- Pervia. Concrete technology that provides optimal management of water drainage in conventional and structural pavements.

Finally in order to link the success in the field with the success in the lab, it is important to show the process applied for industrialisation, which involves in a very simple manner different aspects as follows:

- Strong collaboration among experts from different areas (commercial, technical, operative, R&D).
- Global networking to speed the leverage of different experiences among countries and markets.
- Customer involvement to properly get feedback.
- Plant certification for achieving operative excellence.

2. CONCLUSIONS

As are stated the challenges shown by the concrete industry inside the regional economic and social situation of Latin America and the Caribbean, and by showcasing some of the most important developments in concrete technology, it is demonstrated that innovations can be deployed among regions and inside the most constraint projects like Low Income Housing. The success is not only for the concrete industry but also for the society in general, where concrete can be shown not only as an excellent material for construction but also as part of the solution to build more adaptable, resilient and sustainable buildings and urban infrastructure.

SESSION FOUR

Marketing and management

MARKET DEVELOPMENT: THE UK EXPERIENCE

Andrew J. Minson

Executive Director of the Concrete Centre, UK

Abstract

Concrete is the most widely used material in the world but there is no room for complacency. The UK experience demonstrates that alternative materials can be chosen to replace concrete for large percentages of the market place. In addition as new markets develop there is no automatic right that concrete is chosen.

There is a need to invest in activity to ensure concrete can compete with alternative materials - that concrete is “on the playing field”. This activity requires technical expertise and understanding of designing projects in concrete to be able to influence standards, regulations and industry guidance. It is an extension of license to operate activity and expertise in the material or concrete products themselves. These activities are essential but not sufficient.

There are further opportunities for specific target market development, and in the UK this has focussed on multi storey buildings, housing, offshore wind foundations and more recently roads. In the first two markets, market share statistics show a recovery in the fortunes of concrete and concrete products following investment in activity. The new market of offshore wind foundations is yet to show a return on investment.

Keywords: Marketing, sustainability, steel, timber

1. INTRODUCTION

Concrete is the most widely used material in the world but there is no room for complacency. The UK experience demonstrates that alternative materials can be chosen to replace concrete for large percentages of the market place. In addition as new markets develop there is no automatic right that concrete is chosen.

2. THREAT FROM ALTERNATIVE MATERIALS

Steel has become the dominant structural material for long span steel single storey buildings with a UK market share in excess of 95%. This contrasts markedly with the UK in the 1960s, when concrete dominated this market. The multi storey buildings market is also dominated by steel with a two-thirds market share. A doubling in market share was achieved in a 20 year period from 1980.

In housing timber frame construction in the UK almost trebled its share from 8% in a 9 year period at the beginning of this century. In all building markets, the cross laminated timber (CLT) product, currently imported into the UK from Austria, poses a new threat. It is a flat panel made up of layers of poor grade timber to form structural grade wall and floor panels of typically 180mm to 250mm thickness. This product together with glulam beams (which in the UK are more a threat to long span steel beams), offer a new palette of products to those wishing to construct using timber for its perceived sustainability advantages.

A third alternative material is plastics. The term plastic covers a whole spectrum of properties and performance but the common features of lightness and corrosion resistance make plastics a potential risk to concrete in many applications. Currently the most significant impact into concrete market share is in pipeline systems. Market share figures are not available but qualitative evidence is undeniable.

Finally, asphalt dominates as the material of choice in the UK for pavement construction and wearing surfaces. Concrete roads, particularly with concrete as the running surface fell out of favour in the UK because of joints and road noise.

The threat from alternative materials exists in new markets as well. For example, offshore wind is a huge potential market for concrete with approximately 1,500 towers to be built by 2020. The foundations of these could be 5,000 tonne of concrete each – or they could be founded on steel bases.

3. COST

It is a given that concrete products need to be available and cost effective in order that market share can be defended, market share grown or new markets entered into. Availability and cost effectiveness can be, and is supported by trade associations and this license to operate activity.

However, cost effectiveness is also a function of how designers can use the products. For examples of how this impacts market share we can learn from history. How did the UK steel industry win the multi storey buildings market? They made it cheaper to construct by standardizing connection details, refining calculation procedures so smaller beams were needed and reducing costs of fire protection by successfully lobbying for changes in regulations. The outcome was cheaper steel frames. The steel tonnage per building may have been reduced, but more steel buildings were built. The economy was not due to lower cost of the product per tonne, but lower cost of the solution.

In the UK we have responded by lobbying regulations and building standards to be advantageous to concrete and masonry. We have provided the guidance documents, computer software and training to equip engineers to develop the most economical concrete solutions so that these are more likely to successfully compete against steel. As a consequence we have halted the growth in steel market share in multi storey buildings and begun to make inroads.

Looking forward, there is extensive work to be done with design standards to ensure that concrete construction is not more expensive than it needs to be. Ten issues are listed in Table 1 which need to be addressed by the concrete industry during the the current revision process to of Eurocode 2 to ensure that concrete design remains as competitive as possible. Such a simple list does not do justuce to the extensive work over many years in understanding these issues and the opportunities an dthreats that lie within them. The issues have been brought to the attention of the European Concrete Platform together with analysis of the potential impact on market share and investment required to address each of them

Table 1: Issues with EN 1992, Eurocode 2: Design of concrete structures, which offer potential commercial market advantage to the concrete sector

Deflection: client specified limits	Sway sensitivity
Deflection: codified calculation procedures	Punching shear
Concrete partial safety factor	Fire – to gain parity with other sectors
Design for say 91- day strengths	Compression anchorages
Bond and low cover	Detailing

4. SUSTAINABILITY

Sustainability is perhaps more significant in the UK than most countries. The perception of timber's sustainability credentials, together with its perceived speed of construction and offsite manufacture, helped it nearly treble its market share in housing from 2000-2009. The new product of CLT is also marketed extensively on its sustainability credentials.

The UK concrete industry's response on sustainability is now well established and widely respected. A concrete industry strategy bringing together cement, ggbs, fly ash, admixtures, aggregates, ready-mixed, precast and reinforcement is now in its 8th year. A single voice across all these sectors has enabled a more credible, consistent and effective voice. The message is improved sustainable production, unparalleled in-use performance benefits that enable whole-life sustainability and end of life recycling. We promote, local, low whole life carbon and long life'. The Concrete Centre has published extensively for the audience than make material choices or those that in turn influence them. A textbook "Sustainable Concrete Solutions"[1] brings together into one publication the sustainable story that can be told.

In general, sustainability credentials of concrete may not win much market share, but it can ensure concrete remains an option to be considered. The exceptions to this relate to performance characteristics such as thermal mass, fire performance and longevity, all of which can rightly be attributed as sustainability and do sometimes, in themselves win projects.

A specific market of note is offshore wind foundations. Some steel solutions are ruled out on environmental grounds, namely the vibration resulting from percussive piling which adversely affects porpoises, seals and some fish species. This makes concrete foundations more likely to be chosen.

Another specific example is that of highway central reservation barriers. Concrete barriers do not need repairing whereas the steel alternative had the adverse social sustainability consequence of 5 or 6 highway workers being killed per year whilst repairing steel barriers during night time lane closures. This was a key aspect in transforming concrete to be the overwhelmingly dominant solution for central barriers.

5. BEING ON THE PLAYING FIELD

"Being on the playing field" describes our ambition that concrete can be chosen - that concrete is able to be considered an option because regulations, standards, voluntary assessment schemes, planning restrictions etc permit concrete to be chosen without undue restrictions. The issues addressed above - cost and sustainability - are very much a case of being on the playing field. A further example from recent years in the UK involved local planning rules for which there was a proposal for timber construction to be the default material unless good reason could be provided in local planning was successfully countered. This is a good example of seeking to ensure concrete is on the playing field. It is of note that the key arguments that won this argument were that concrete is local, offer low whole life carbon solutions, durable and fire safe and that designers are best placed to choose materials.

Educational work in universities to ensure future designers know why to choose concrete and how to design with concrete has obvious merits but in the UK over the recession this work has been significantly curtailed. An effective activity that has been retained, maquerades as a competition. Concrete building design is embedded as a design project into the course work in the final year of engineering courses by providing a design brief and a caompetition format where the best entry from each university is entered for national judging. More information on this, including the design problem set each year and summary video can be found at www.concretecentre.com.

6. TARGET MARKETS

Specific work on market development in the UK is focussed into target markets – the resource cannot stretch to try and grow or defend market share in every possible market. For several years The Concrete Centre, over and above the work already highlighted in this paper, has concentrated on target markets of multi storey buildings, housing and offshore wind. More recently concrete roads with asphalt wearing surface has also been the focus of attention. These markets are chosen for special efforts because of the potential return on investment in them.

The multi storey market has been a target market since The Concrete Centre was formed in 2003 and there has been established an extensive library of documents on:

- Performance (e.g. thermal mass, fire)
- Construction method (e.g. post tensioning, hybrid, precast)
- Analysis Methods (e.g. Eurocode 2, finite element analysis, vibration analysis)
- Sectors (e.g. offices, schools, tall buildings)

For this market there are many professionals who influence material choice. A study of this and what impacted their decisions was the subject of research funded through The Concrete Centre. An example output is presented in figure 1. Such research findings helps target efforts to ensure efficient use of resources.

The benefits of work in this market has been seen in the reversal of some of the market share gains by steel from 1980 through to 2005.

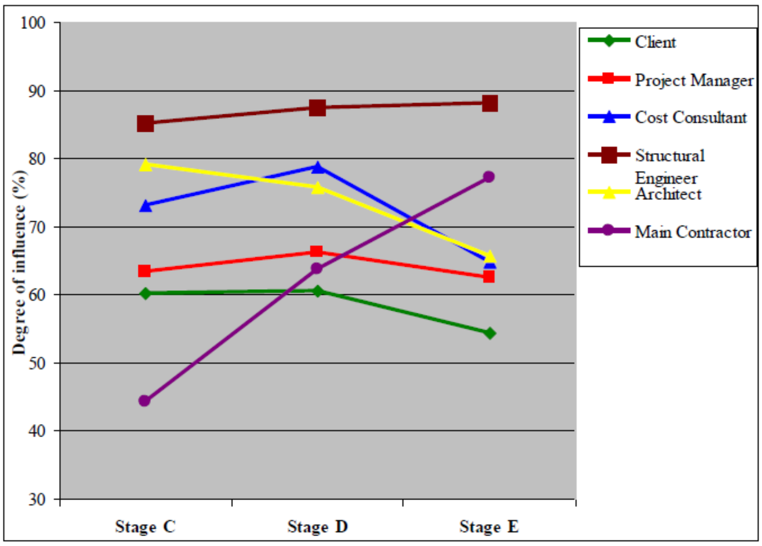


Figure 1: Respondents' view of the influence of the project team members during concept (stage C), scheme (stage D) and detailed (stage E) design stages showing that the structural engineer has the most significant degree of influence on material choice[2]

The Housing market has been addressed in a similar manner as above with architects being the dominant decision maker and structural engineers rarely featuring. There are three further additional features of note. A significant amount of the innovative end of the market is self-build, and these offer a particular client audience that do heavily determine material choice. Social housing (or public housing) is also worthy of note in that it can affect associated private housing developments where planning requires a mixed tenancy provision. And finally major house builders dominate the UK market and even though architects, whether working in- house or as consultants determine material choice, the build repetition can influence commercial decisions even to the extent of purchasing timber frame manufacturing capacity.

The benefits of work in this market has been seen in stemming the rise in timber frame market share as demonstrated in Figure 2.

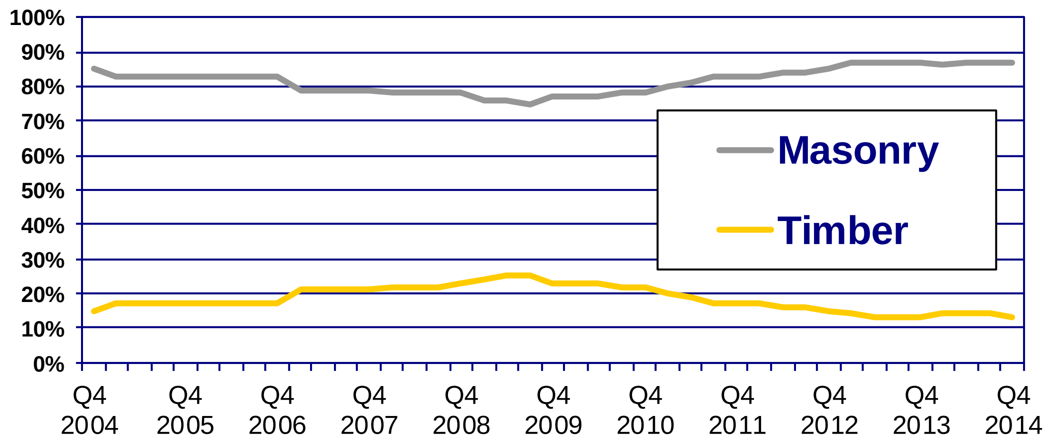


Figure 2: Low rise housing timber market share data from NHBC[3] (Masonry is assumed as 100% - timber frame %)

The Offshore wind foundations market has offered different opportunities to influence market share. These have arisen because:

- Design teams have committed to concrete as a solution and have become co-workers in championing concrete solutions
- A single landlord exists for the UK seabed and hence for all wind farm development
- There is no established solution for deep wind turbine foundations because it is new.

As a consequence, the mix of activity and the funding for this market this has been distinctive from other markets with an Offshore Wind Interest Group being formed with companies beyond the concrete industry. It is still too early to determine a return on investment, as the deeper wind farms are only now receiving planning permission.

Finally the roads market has recently opened up because of rising bitumen prices and a focus on whole life low maintenance pavements. This has arisen because of a change in ownership/funding of UK highways which places it at more distance from government. In private hands, the appeal of low maintenance solutions increases and concrete is considered more favourably. However, this does not include concrete as a running surface as it is still considered to be unacceptably noisy.

7. CONCLUSIONS

The experience in the UK is that there is both a need and an opportunity to address market share of concrete. Alternative materials will be used at the expense of concrete unless action is taken, but action can change perceptions, decisions and ultimately market share.

REFERENCES

[1] Georgopolous, C and Minson A. Sustainable Concrete Solutions. Wiley 2014

[2] Haroglu H. Investigating the structural frame decision making process. Loughborough University December 2009
[3] NHBC. Housing Market Report. March 2015

QUALITY SCHEME FOR READY MIXED CONCRETE: FROM SELF-REGULATION TO THIRD-PARTY CONTROL

Vijay R. Kulkarni (1) and Ravishankar Mahadevappa (2)

(1) Principal Consultant, Ready Mixed Concrete Manufacturers' Association, India

(2) President, Ready Mixed Concrete Manufacturers' Association, India

Abstract

In a country like India having a long and continuing history in the use of labour-intensive site mixed concrete, quality of concrete has indeed been one of the major concerns of customers. The paper highlights the efforts made by the Ready Mixed Concrete Manufacturers' Association (RMCMA), India, in evolving and implementing a quality framework for ready-mixed concrete in India. Initially, RMCMA developed a self-regulatory framework of the quality scheme and commenced its voluntary implementation from December 2008. After operating this scheme successfully for nearly four years, RMCMA decided to upgrade it. For this purpose, RMCMA signed a Memorandum of Understanding (MoU) with the Quality Council of India (QCI) - a non-profit and autonomous national apex organisation involved in quality facilitation, accreditation and surveillance. For making the scheme broad-based, QCI created three-tier, multi-stake holder committees, which evolved three comprehensive documents, dealing with technical requirements for production control of concrete, certification process and requirements for certifying bodies. Two types of certification are offered under the scheme – one on RMC Capability Certification (which is mandatory) and the other on RMC 9000+ certification (which is optional). While the technical requirements of the scheme are based on the relevant Indian Standards on concrete and its ingredients, the certification process and requirements of certifying bodies are based on different ISO Standards such ISO 9001, 17065, etc. The paper describes salient features of the QCI quality scheme. In many respect, the provisions of QCI scheme are more stringent than the erstwhile self-regulatory scheme. Thus, the journey of the quality scheme from self-regulation to third-party control is an advancement which will immensely benefit the customers.

Keywords: Quality assurance, ready-mixed concrete, self regulation, quality council of India, third-party certification

1. CONCRETE INDUSTRY SCENARIO IN INDIA

Ready-mixed concrete industry was born in India in early 1990s. After overcoming the initial teething troubles, the real growth commenced since late 1990s. By the year 2010, the industry spread its wings to nearly 50 cities of India. A brief account of the concrete industry scenario in India was provided in an earlier paper by the first author published in the proceedings of the XVIth ERMCO Congress[1]. The paper also highlighted the major hurdles to the growth of ready-mixed concrete in India. Based on certain assumptions, the paper estimated that the volume produced by the organized concrete industry in India will reach 87 million m^3 in 2012-13 and 147 million m^3 in 2017.

In absence of authentic data, it is difficult to verify the predictions made earlier. However, it is a fact that the general economic slowdown witnessed by the Indian economy during the period 2011-2014 adversely affected the concrete industry also. The commercial ready-mixed concrete in India mainly caters to the real estate sector, which witnessed a slump during the period 2009-13. The growth in this sector came down from a peak of 30% to 8% during 2009-11 and then to 6.5% in 2012-13. As a result, major ready-mixed concrete companies were not only constrained to withhold expansion plans but were forced to close a few unviable units. Many companies experienced reduction in their business or a negligible growth and their profitability was adversely affected.

However, the overall economic situation in India has improved recently. With the reduction in oil prices, inflation has slowed down and the fiscal deficit has narrowed. Further, with the politically stable government at the national level, the business sentiments have vastly improved and the outlook for the Indian economy has now turned distinctly positive. With the acceleration of economy and emphasis on public infrastructure development, the concrete industry in India is now hopefully looking forward to a better future.

2. QUALITY SCHEME FOR READY-MIXED CONCRETE

Even though ready-mixed concrete has now taken roots in India, with its presence extending to nearly 70 cities in the country, a substantially high proportion of concrete is still produced by using labour-intensive, volume batched site mixed concrete. In view of the "primitive" techniques used, the quality of such concrete has always been a major concern amongst the users and public authorities. The Ready Mixed Concrete Manufacturers' Association (RMCMA), India, therefore took the decision of creating a quality platform for winning over the customers using site-mixed concrete.

In view of the absence of any national quality framework for concrete, RMCMA took the initiative of developing the quality scheme for ready-mixed concrete in India. It also decided that the scheme would be self-regulatory and that it will rest on two strong pillars, namely, best international practices that are suitable for Indian conditions and strict observance of the standards of the Bureau of Indian Standards. Two quality manuals were published by RMCMA, incorporating the details of the scheme[2,3]. The paper by the first author presented during the XVth ERMCO Congress describes the salient features of this scheme.

RMCMA operated its self-regulatory quality scheme from December 2008 till March 2012. Based on the audits conducted by expert auditors, around 250 RMC facilities throughout the length and breadth of country (mostly belonging to RMCMA member companies) were thoroughly audited and certified by RMCMA. Figure 1 shows the city-wise plants certified by RMCMA.

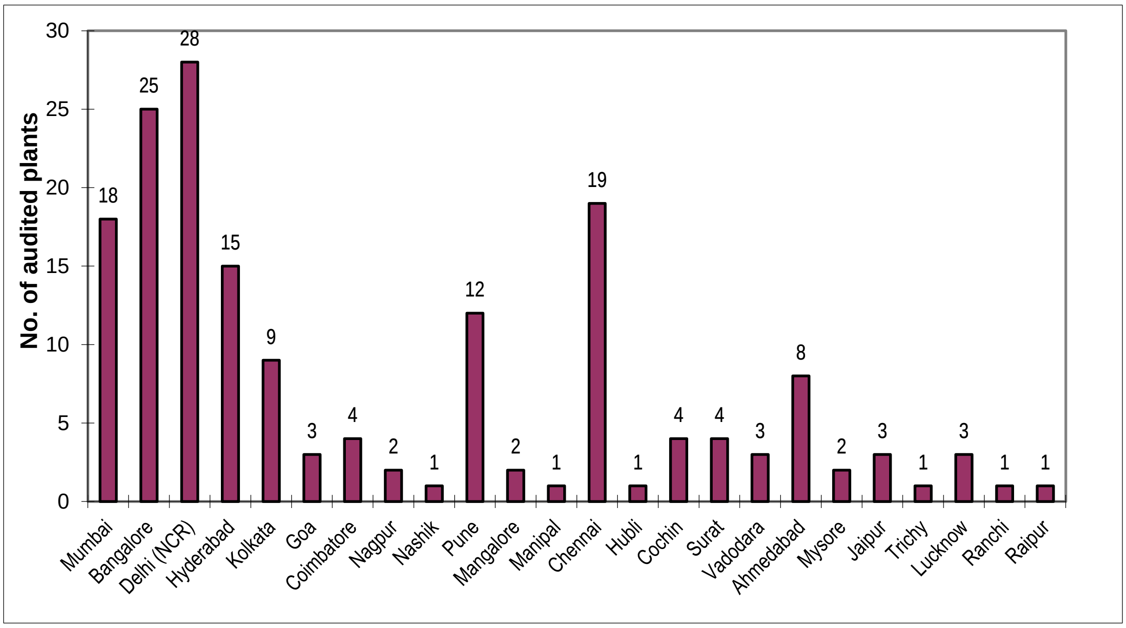


Figure 1: City-wise status of certified ready-mixed concrete plants by RMCMA

3. UPGRADATION OF QUALITY SCHEME

After operating the quality scheme for ready-mixed concrete successfully for nearly four years, RMCMA decided to raise the quality scheme to a higher pedestal. For this purpose, RMCMA signed a Memorandum of Understanding (MoU) with the Quality Council of India (QCI). The latter organization was set up in 1997 jointly by the Government of India and the Indian Industry represented by the three premier industry associations i.e. Associated Chambers of Commerce and Industry of India (ASSOCHAM), Confederation of Indian Industry (CII) and Federation of Indian Chambers of Commerce and Industry (FICCI). QCI is registered as a non-profit, autonomous society, governed by a Council with representatives from the government, industry, technical institutions and consumers. The Chairperson of the organization is appointed by the Prime Minister.

For upgrading the quality scheme for ready-mixed concrete, QCI took the initiative of setting up three multi-stakeholder committees, namely, the Steering Committee, Technical Committee and Certification Committee, *Figure* 2.

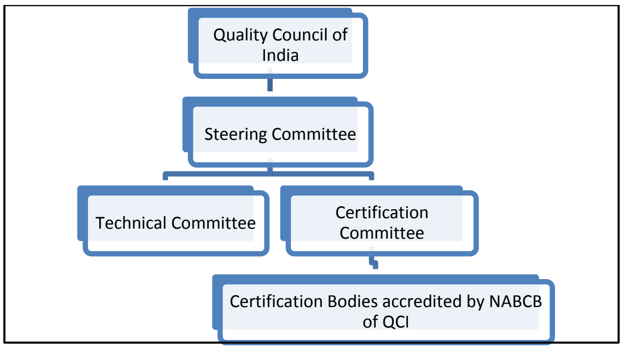


Figure 2: Multi-stakeholder committees set up by QCI

The QCI scheme has truly an independent character and multi-stakeholder ownership. This is evident from the composition of three committees, comprising of representatives from the following main organizations:

– Central Government Ministries, e.g. Housing, Ministry of Road Transport & Highways (MORT&H)
– Central Public Works Department
– Central Public Sector Undertakings e.g. National Highway Authority of India (NHAI), Airport Authority of India (AAI), etc.
– User bodies, e.g. Builders Association of India (BAI), Construction Federation of India (CFI), Confederation of Real Estate Developers' Association of India (CREDAI)
– Professional bodies, e.g. Indian Concrete Institute (ICI), Association of Consulting Civil Engineers (ACCE)
– Leading consulting engineering organizations
– Manufacturers, e.g. Ready-mixed Concrete Manufacturers' Association (RMCMA), Cement Manufacturers' Association (CMA)
– Certifying bodies, e.g. Bureau Veritas Certification India Pvt Ltd, and ICMQ India.

Thus, the scheme is owned by elite and diverse groups from the construction industry. In fact, an attempt has been made to involve representatives from all those who matter in the sphere of structural concrete in India.

The evolution of the QCI Scheme took more than one-and-a-half year because of the need to accommodate views and concerns of different stakeholders/owners of the scheme. More than a dozen meetings of the committees were required to finalize the following painstakingly-prepared manuals of the scheme (Soft copies available from http://qcin.org/CAS/RMCPC/) :

- Criteria for Production Control of Ready Mixed Concrete[4]
- Certification Process for Ready Mixed Concrete Production Control Scheme (RMCPCS)[5]
- Provisional Approval for CBs for RMCPCS[6].

The QCI Scheme conforms to the requirements of the Bureau of Indian Standards (BIS), Indian Roads Congress (IRC), Indian Railway Standards (IRS), and is applicable for plants supplying concrete commercially, those supplying concrete for specific project, and plants supplying concrete partly on commercial basis and partly for captive consumption. Scheme excludes operations of placing, compaction, finishing and curing of concrete.

The QCI Scheme offers two certifications, namely,

- RMC Capability Certification, and
- RMC 9000^+ Certification.

The RMC Capability Certification is plant-specific. For getting this certificate, it is mandatory for each ready-mixed concrete plant to successfully undergo a thorough audit based on an extensive check list included in the Criteria for Production Control of RMC[4]. The second certification is optional and it includes most of the features of the quality management system of ISO 9001, and of course, all features of the Capability Certification. Thus, to get certified under the second option, each plant must fulfil all requirements under the first option and also follow the quality management system.

4. BROAD FEATURES OF RMC CAPABILITY CERTIFICATION

RMC Capability Certification is the crux of the QCI Scheme. Besides the audit of the plant and machinery and the control mechanism adopted by the producer, the criteria also embody laboratory testing facilities, technical skills of the human resources, the controls exercised on the quality of different concrete ingredients, mix design and the final product.

The production control criteria mainly include following features:

- Resource management
- Control on quality of incoming materials
- Concrete design
- Production and delivery
- Control on process control equipments and maintenance
- Complaints and Feedback.

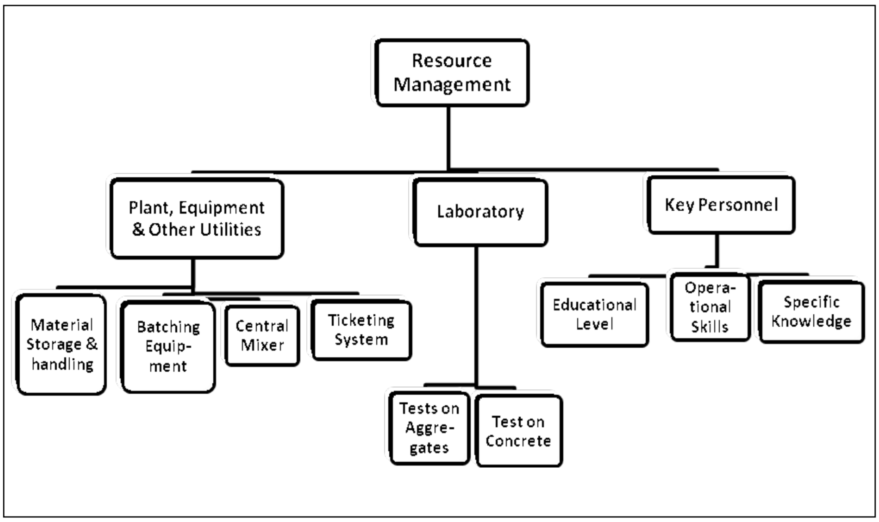


Figure 3: Resource management under production control criteria

Resource management: It covers three main areas, namely, plant, equipment and other utilities, laboratory and key personnel (See *Figure* 3). Most of the check list items for plant, equipment and other utilities of the erstwhile RMCMA Scheme have been retained. Laboratory testing and knowledge and skills of key personnel have a large bearing in determining final quality of the product. Hence, these additional inputs are included in the new scheme.

As mentioned earlier, provision of basic laboratory testing facility is made mandatory in the QCI Scheme. The minimum testing equipment for the laboratory and the calibration frequency of the equipment are specified. (Table 4 of Criteria document)[4].

Control on Quality of Incoming Materials: The ready-mixed concrete producer needs to verify quality of all ingredients on a regular basis. Various standards of the BIS specify tests to be performed on different ingredients and their frequencies of testing. The QCI scheme has made it mandatory to perform these tests at BIS specified frequencies (*Table* 6-A and 6-B of Criteria document)[4]. It is also specified that the physical and chemical properties of the basic ingredients should be tested in NABL-accredited laboratory, at least once in six months or when there is a change in the source of materials. Such rigorous testing regime would certainly ensure that ingredients having good quality are used in production of ready-mixed concrete.

Concrete Design: The QCI scheme requires that the ready-mixed concrete producer should have in-house capability to carry out mix proportioning and adjustments in the mix to cater to the variability in the incoming materials – for example, the variations in the moisture contents in aggregates, etc. The competence of the key personnel who carry out mix design will be judged by the auditors by going through the old records, interviewing some key personnel and witnessing few tests in the laboratory.

Production and Delivery: The auditor has been given the freedom to choose any five orders received by the producer during the past 3 months and verify from the autographic records as to whether the supplies have been made exactly as per the orders of the customers. Incidentally, the tests to be conducted and their frequencies in controlling the quality of the final product have also been specified in the QCI Scheme (*Table* 9 of Criteria document)[4].

Control on Process Control Equipment and Maintenance: Upkeep of plant and equipment, their calibration, etc. play an important role in determining the quality of the final concrete. The QCI scheme specifies the frequency of inspection, calibration tolerances, etc. The scheme specifies that scale calibration for all weighing and measuring equipment using electrical/ load cell system needs to be done at least once in a month. Stringent tolerances are specified for measurement of all constituent materials; e.g. ± 2 % for cement and supplementary cementitious materials and ± 3% for water, chemical admixtures and aggregates. (*Table* 10 and 11 of Criteria document)[4].

5. CERTIFICATION PROCESS

The Certification Committee set up by QCI evolved detailed procedures for carrying out certification under the two schemes[5]. The broad aspects covered under the procedures are briefly described below.

1. **Application for certification:** It covers details such as application form, information for applicant and registration of applicant. The information to be furnished by the applicant is also included.
2. **Audit program:** The audit program is divided into stage 1, stage 2 and surveillance audits. While each plant needs to be under RMCPCS, the ISO 9001 audits should be carried out on sampling basis as allowed under ISO 9001 certification.
3. **Audit mandays:** Audit mandays under certification and surveillance audits for both schemes have been fixed by the Certification Committee.
4. **Certification audit planning:** This includes the information to be provided to the certifying body, constitution of audit team and audit plan.
5. **Certification audit:** It provides guidance on how RMC 9001^{+} QMS based certification and RMC capability certification audits should be performed. Safety measures to be adopted during the audit are also highlighted.

Three types of non conformity (NC) have been specified, namely, critical, major and minor. The detailed description of each of the NCs and the time for closure has also been specified in para 5.3.1 and 5.3.2 of the Certification Process document[5]. The CBs need to send the audit report within 7 days from the date of the completion of the audit.

6. **Certification decision:** Certificate will be issued by the CB, only when all raised NCs are closed (critical and major after on-site verification and minor after off-site verification).
7. **Surveillance:** Surveillance audits shall be carried out by the CB every six months, with at least one surprise audit in a year. The surprise audit will be conducted with a short notice of 3 days.
8. **Complaints:** This section covers the complaint handling process. The need to have a documented process to receive, evaluate and make decisions on complaints is highlighted. The CB should audit the complaints received by the plant from its customers. The manner in which a complaint received about a certified plant should be handled is described.
9. **Certificate:** The information to be included in the certificate as well as its validity is described.
10. **Suspension and withdrawal of certificate**: It provides detailed guidance when the certificate will be suspended and withdrawn and the suspension will be revoked.
11. **Change of location/ownership/name:** The certified plant needs to inform the CB changes in location, ownership and name of company. With the change in location, audit of the new facility becomes essential. In case there is a change in the name of the company or its ownership, appropriate documents need to be submitted to the CB.
12. **Fees:** the fee structure of the CB shall be publically accessible and also provided on request.

6. APPROVAL SYSTEM FOR CERTIFICATION BODIES

The Certification Bodies (CBs) which would be auditing and certifying the ready-mixed concrete plants under the RMCPCS need to primarily comply with the requirements specified in ISO 17065 and additional requirements specified by the Quality Council of India. With a view to commence the operation of the scheme immediately, a provisional system for approval of the CBs was evolved, documented and approved by the Certification Committee[6]. Stringent requirements have been specified the committee for the CBs operating the scheme. For example, the auditors attached to the CBs need to have the following minimum competence:

- Minimum Bachelor's degree in engineering in related field(s) with at least 5 years of relevant experience in RMC/Batching plant; or Diploma in engineering in related field(s) with 7 years of relevant working experience in RMC/batching Plants
- Experience in core technical processes like QA/QC or production and process control
- Training and experience in auditing.

Further, the CBs will be subjected to a through yearly audit. They need to obtain accreditation from National Accreditation Board for Certification Bodies (NABCB) under the QCI.

7. BENEFITS OF QCI QUALITY SCHEME

The main advantages of the QCI Scheme to different entities are as below.

- For Owners and Specifiers (architects, consultants)

- Third-party quality assurance from an independent agency, based on well-defined quality norms evolved by experts
- Reliable Tool for short-listing of concrete producers

- For RMC/Concrete Producers
  - Competitive advantage over non-certified producers
  - Top management gets audited data on their plants
- Small Customer (e.g. individual house builder)
  - Assurance on QA&QC of concrete, without employing experts
- Concrete Industry
  - Raise the industry standard and bring it on par with those from advanced countries.

8. URRENT STATUS

Currently, nearly 125 RMC facilities throughout the length and breadth of India have been certified under the QCI Scheme and around 50 more facilities have applied for certification. Some government and semi-government organizations like the Mumbai Municipal Corporation, CIDCO, etc have made the certification under this scheme mandatory.

9. CONCLUSION

The QCI Quality Scheme provides today a comprehensive framework of ensuring quality of concrete from ready-mixed concrete plants. In many aspects, the QCI Scheme is made more stringent than the erstwhile RMCMA Scheme. The independent, third-party character of the scheme accompanied with six-monthly audits of the production facility has helped in instilling higher level of confidence amongst customers. Thus, the quality scheme's journey from journey from self-regulation to third-party control is an advancement which has greatly benefited the customer. Although the scheme is voluntary, looking at the benefits, specifiers and owners can make it mandatory for their projects/jobs.

REFERENCES

[1] Kulkarni, Vijay, 'Evolution of RMC in India', XVIth ERMCO Congress, Verona, Italy, June 21-22, 2012.
[2] Quality Manual Part – I 'Check List for Certification of Ready Mixed Concrete Production Facilities', Ready Mixed Concrete Manufacturers' Association, India, 2008.
[3] Quality Manual Part – II 'Guidelines on Quality Control and Quality Assurance of Ready Mixed Concrete', Ready Mixed Concrete Manufacturers' Association, India, 2008.
[4] 'Criteria for Production Control of Ready Mixed Concrete under Ready Mixed Concrete Plant certification Scheme' (QCI), Building Materials & Technology Promotion Council, Ministry of Housing & Urban Poverty Alleviation, Government of India, New Delhi, 2013.
[5] Ready Mixed Concrete Plant Certification Scheme (RMCPCS) – 'Certification Process', Quality Council of India, New Delhi, 2013.
[6] Ready Mixed Concrete Plant Certification Scheme (RMCPCS) – 'Provisional Approval System for Certification Bodies', Quality Council of India, New Delhi, 2013.

MARKET CHALLENGES FOR CONCRETE MACHINERY

Renato Biavati (1) and Marco Brambati (2)

(1) Head of Area in Europe, CIFA and ZOOMLION

(2) Strategic Marketing Director, CIFA and ZOOMLION

Abstract

In the last recent years big extraordinary events radically changed the world wide scenario of concrete machinery industry. Liebherr completed its Concrete equipment offer with the acquisition of Waitzinger, Zoomlion bought CIFA, Sany bought Putzmeister and Intermix. These events have pushed in the last four years to an aggregation between majors Concrete Equipments Manufacturers.

The effect of this industrial aggregation on “Sustainability and Competitiveness” is positive. Due to “Financial and Industrial” capability of these big groups, machinery manufacturers did new investments for innovation, after sales, financial services, and marketing activities, and a new innovating and safer product and service offer must be generated for final customers and users.

Today we forecast a more and more selective European demand, quality rather then quantity with attention to financial services. Characteristics that only big industrial groups with huge investments capacity and articulated organizations, can be offered.

Keywords: Concrete machinery, (truck) pumps, (truck) mixers, innovation

1. INTRODUCTION: CONSTRUCTION EQUIPMENT MARKET

The ranking of TOP 20 construction equipment manufacturers had a total sales amount growing from 83.428 Mln/USD in the year 2004 to 131.181 Mln/USD in the year 2014 (+ 57,2 %).

1.2 Construction equipment manufacturers - M&A in top 20 enterprises

- In September 2008, the Italian CIFA (Compagnia Italiana Forme Acciaio) one of the leading companies among concrete machineries manufacturers has been acquired by a consortium lead by the Chinese Group Zoomlion.
- In January 2012, the German Putzmeister one of the leading companies among concrete machineries manufacturers, announced the signing on the merger, lead by SANY together with a Chinese Private Equity as minority share holder
- In April 2012, has been announced that the German Schwing Group one of the leading companies among concrete machinery manufacturers, signed an agreement with XCMG which has acquired a majority stake.
- In October 2012, the German Liebherr one of the leading companies among concrete machineries manufacturers, announced the acquisition of concrete pump manufacturer Waitzinger

1.3 Construction equipment manufacturers - Top 20 in sales value

The following table reports a ranking with an estimation of annual Sales values comparing year 2004 to year 2014. Main enterprises with concrete equipments in portfolio are highlighted in yellow.

		Total USD	83.428.000.000
YEAR 2004			
Rank	Country	YEAR 2004 - TOP 20	Construction Equipments
2004	Head Qt	Company	Sales 2004 (USD)
1	US	Caterpillar	22.931.000.000
2	JP	Komatzu	9.470.000.000
3	US	Terex	6.400.000.000
4	US	John Deere	5.229.000.000
5	SE	Volvo CE	4.582.000.000
6	DE	Liebherr **	4.151.000.000
7	JP	Hitachi CM	3.998.000.000
8	US	CNH CE	3.963.000.000
9	US	Ingersoll Rand**	3.850.000.000
10	SE	Sandvik Construction **	2.706.000.000
11	UK	JCB	2.230.000.000
12	FIN	Metso Mining & Constr	2.118.000.000
13	SE	Atlas Copco CE	1.995.000.000
14	US	Oshkosh Access Eq (JLG)	1.735.000.000
15	US	Manitowoc Crane Group	1.629.000.000
16	JP	Kobelco CM	1.557.000.000
17	KR	Doosan Infracore	1.358.000.000
18	KR	Hunday Heavy Ind **	1.288.000.000
19	FR	Manitou	1.207.000.000
20	FIN	Hiab	1.031.000.000

		Total USD	131.181.000.000
YEAR 2014			
Rank	Country	YEAR 2014 - TOP 20	Construction Equipments
2014	Head Qt	Company	Sales 2014 (USD)
1	US	Caterpillar	28.283.000.000
2	JP	Komatsu	16.877.000.000
3	JP	Hitachi Construction Machinery	7.790.000.000
4	SE	Volvo Construction Equipment	7.785.000.000
5	US	Terex	7.309.000.000
6	DE	Liebherr (Waitzinger)	7.129.000.000
7	US	John Deere	6.581.000.000
8	CN	XCMG (Schwing-Stetter)	6.151.000.000
9	CN	Sany (Putzmeister-Intermix)	5.424.000.000
10	KR	Doosan Infracore	5.414.000.000
11	CN	ZOOMLION (Cifa)	4.376.000.000
12	UK	JCB	4.117.000.000
13	JP	Kobelco Construction Machinery	3.689.000.000
14	FIN	Metso	3.550.000.000
15	US	Oshkosh Access Equipment (JLG)	3.507.000.000
16	IT	CNH Industrial	3.346.000.000
17	KR	Hunday Heavy Industries	2.711.000.000
18	DE	Wirtgen Group	2.666.000.000
19	US	Manitowoc Crane Group	2.305.000.000
20	SE	Atlas Copco Construction Technique	2.171.000.000

Figure 1: Table Sales values. Data source Yellow Table International Construction

1.4 Demography development – Trend by geography

The following table shows the forecast of population growth: Asian markets since years have a constant growth and the trend for future is impressive. Africa will double the population. American market has a constant population growth

WW POPULATION - US CENSUS BUREAU - FORECAST 2011	6.846.072.206	9.440.000.000	37,9%
GEO - REGIONS	Population 2011	Population 2050	Trend 2011-2050
Asia & Oceania	4.241.226.077	5.254.000.000	23,9%
Africa	995.060.892	2.270.000.000	128,13%
America - Central & South	583.893.273	773.000.000	32,39%
Europe + Russia	678.464.618	679.000.000	0,08%
America - North	347.427.346	464.000.000	33,55%

Figure 2: Population Trend. Table Data source US Census Bureau

The population growth trend shows where there will be more demand of construction equipment as well as concrete equipment such as Batching Plants, Truck Mixers and Pumps.

1.5 Our market vision for the future

The effect of this industrial aggregation on “Sustainability and Competitiveness” of the sector will be without any doubt positive. The combination of financial and Industrial capabilities, with, in some cases already generated, in other cases will generate, new investments for marketing researches, engineering research and development, quality, trainings, after sales, financial services, and the customers will benefit of new innovative products, services..

A more and more competitive scenario will urge the machinery manufacturers to invest constantly and offer new solutions to improve human conditions in the job sites, safe and quality for operations: this means more added value for Ready mix and Construction enterprises.

The big manufacturer Groups have to be more and more efficient to answer promptly to new market needs through additional services. So flexibility in big organizations will be another very important characteristic.

Another aspect of concrete machinery manufacturers competitive selection, is that the improved competition will identify the most “Global and Flexible” or reactive company among concrete machinery manufacturers: the consequence will be even more benefits for our customers and meanwhile, a new selection in the industry of concrete machinery manufacturers.

2. ENERGY SAVING – SAFETY AND ENVIRONMENTAL RESPECT

2.1 Energy saving and low environmental impact for European markets

The technical evolution on concrete machinery, especially in the present economic downturn scenario, must be focalized on all those technologies that have been proven to be effective in the energy saving field. Hybrid technology (Diesel engine + Electric engine) decreases the energy wastes in concrete machinery, bringing at the same time a positive impact on the environment. Less fuel consumption. Noise reduction up to 10 dB. Zero emissions in closed areas

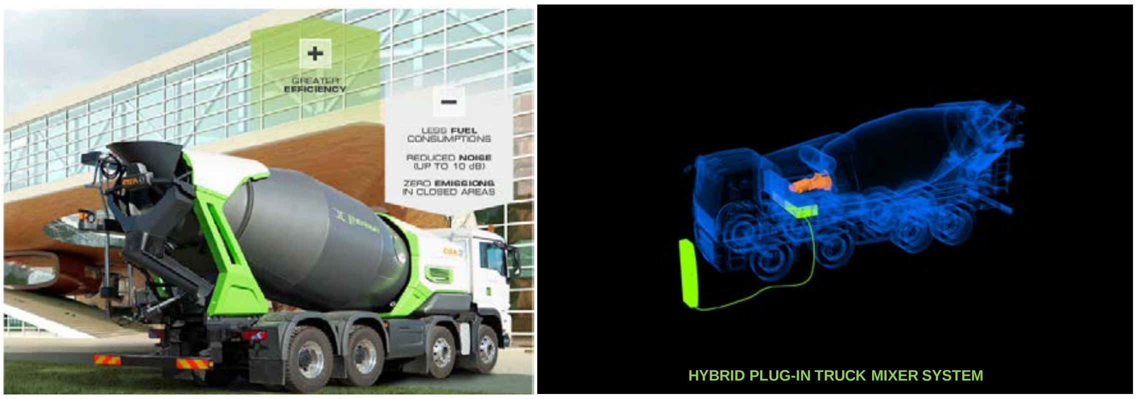


Figure 3: – Hybrid Truck Mixer

2.2 Safety & Maintenance for Truck Mixers

Electronics can play a major role in Truck mixers, in order to make the operation and maintenance of such machines totally under control, increase easier operations and minimize the risk of injuries and accidents.

Figure 4: Electronic devices for total machine control

2.3 Safety & Maintenance for Truck Pumps

The control of the stability is an example of an available technology applied to this field addressing one of the main risks of these machines working in jobsite.

Figure 5: Truck pump

The total control of the full Truck Pump performance is another example of an available technology applied to this field addressing for the best use of machines, in order to facilitate the operator job and reduce the main risks of these machines working in jobsite.

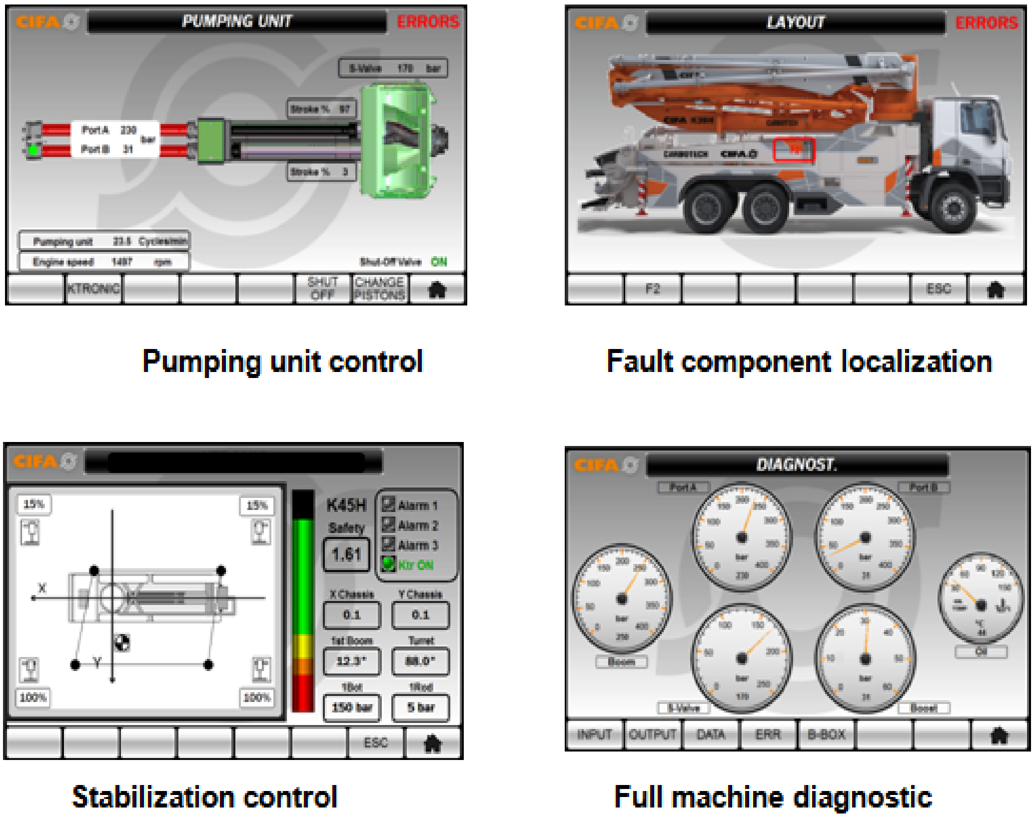


Figure 6: Electronic control management application for Truck pumps

3. NEW MATERIAL TECHNOLOGY

3.1 The advantages of actual new materials.

The use of new composite materials, open new frontiers. The target is to get safer, lighter, stronger products, high wear resistance, allowing cost saving.

Carbon fiber applied to placing booms of concrete machinery, gives advantages and more safety: no cracks, no welding, no rust, light weight. Booms are also more rigid.

Figure 7: Carbon fiber fields of application for Truck pumps

Figure 8: Carbon fiber versus steel. No cracks on placing booms

3.2 The advantages of others new composite materials.

The use of new composite materials with also different composition can open new frontiers, in many other parts on machines and plants for ready mix, allowing cost saving.

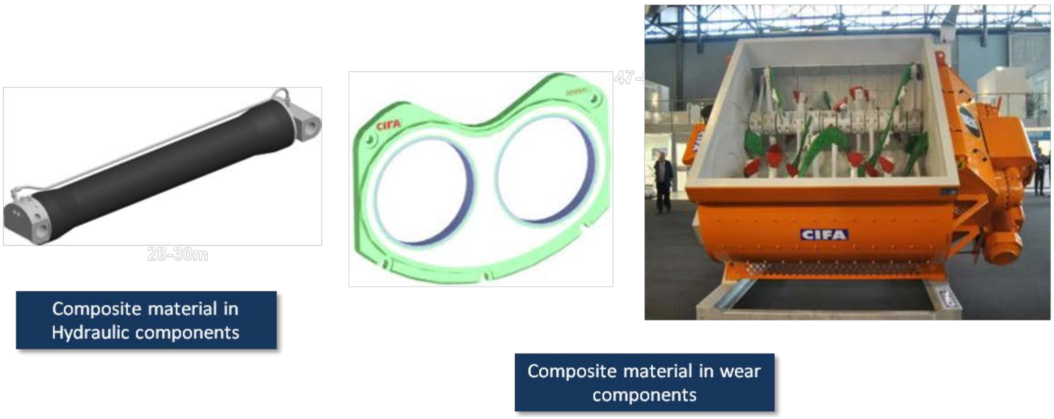


Figure 9: Other possible field of application for composite materials

4. CONCLUSIONS

The effect of industrial aggregations on “Sustainability and Competitiveness” of the sector will be without any doubt positive.

Due to “Financial and Industrial” capability of big manufacturing groups, a new innovating and safer product and service offer is generated constantly for final customers and users.

The technical evolution in the concrete machinery is mainly oriented to meet the market requirements in order to get products more and more reliable, efficient and safe.

The use of electronic systems can support the operators safety, reliability, diagnostics as well the hybrid technology can help in the energy saving and environmental respect.