

Beneficiated Fly Ash Versus Normal Fly Ash or Silica Fume

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Synopsis: The performance of a beneficiated fly ash (BFA), with very low carbon and very fine particle size ($5\text{ }\mu\text{m}$ as mean size) was compared with silica fume in superplasticized high strength concretes. The silica fume content of concrete was 40 kg/m^3 and the amount of BFA was 80 kg/m^3 to obtain approximately the same cost as that of silica fume (SF). When the two concrete mixtures are compared at the same water-binder ratio (0.39) and at a given slump (about 165 mm), the dosage of superplasticizer was much higher for the SF concrete (2.6 kg/m^3) than for the concrete with BFA (1.2 kg/m^3). The compressive strength of the SF-concrete was higher than that of the BFA-concrete, particularly at early ages, and this effect could be related to the better space filling capability of SF, as compared to BFA. However, when the same dosage of superplasticizer was used, the water-binder ratio of the BFA-concrete turned out to be lower (0.31) than with SF concrete (0.39). The strength increase in the BFA-concrete with respect to the SF-concrete has been recorded in specimens cured at room temperature as well as with steam-cured.

In a second set of tests, BFA was used to manufacture high strength self-compacting concrete (SCC) in comparison with a corresponding SCC where untreated fly ash (FA) was used. In such a case the most important advantage in using BFA rather than FA was the self-leveling properties needed particularly in placing slabs or ground- floors. Due to the higher cost of BFA with respect to FA, there is no significant advantage in using the former in manufacturing SCC when the above mentioned self-leveling properties are not needed.

Keywords: beneficiated fly ash; fly ash; self-compacting concrete; self-leveling concrete; silica fume; superplasticizer

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INTRODUCTION

Coal fly ash is a supplementary cementitious material widely used to replace part of portland cement by improving some properties of concrete such as reduction in heat of hydration and related risk of thermal cracking, durability and impermeability of the concrete in service. However, there is a strength loss at early ages with respect to the control concrete without fly ash.

Silica fume performs much better than fly ash in manufacturing high performance concrete without any strength loss at early ages. However, due to its low availability, silica fume is very expensive and used only for special concretes.

In the present paper a beneficiated fly ash (BFA) has been studied in comparison with untreated fly ash (FA) and silica fume (SF). BFA can be based on the separation technology for reducing the carbon content in coal FA through tribo-electrostatic technique [1] and/or air classification of FA to extract the more reactive fine fraction [2] to improve its performance in terms of higher workability of the fresh concrete and lower porosity in the hardened concrete.

EXPERIMENTAL: MATERIALS AND METHODS

BFA, FA and SF have been used in two sets of concrete mixtures:

- a first set of concretes has been devoted to the manufacturing of fluid ordinary mixtures at a slump level of about 170 mm, by using BAF (80 kg/m³) versus SF (40 kg/m³) in order to keep the same cost incidence of these cementitious materials;
- a second set of concretes was based on the comparative use of BFA versus FA in manufacturing self-compacting concrete (SCC) with 50% of replacement of ordinary concrete by BFA or FA.

Chemical composition of BFA, FA, SF are shown in Table 1; in particular the loss on ignition (LOI) of the BFA is only 0.8% and than much lower than that of SF (2%) and FA (6.6 %).

Figure 1 shows the micro-structural morphology of BFA, by scanning electronic microscopy and indicates that BFA particles are perfectly round grains. Figure 2 shows the particle size distribution results which indicate that BFA, with a mean particle of size of about 5 μm , is an intermediate material between SF (about 0.2 μm) and FA (about 25 μm).

Tables 2 and 3 show the composition of ordinary concrete (with SF or BFA) and SCC (with FA or BFA), respectively. High-strength portland cement (CEM I 52.5 R according to the European standard EN 197-1) has been adopted. The total amount of portland cement and SF or BFA or FA is indicated as “binder”, so that in Tables 2 and 3 are also shown the water-cement ratio (w/c) as well as the water-binder ratio (w/b).

Natural aggregates have been used with maximum size of 30 mm for the ordinary concretes and 16 mm for SCC mixtures.

A polycarboxylate-based superplasticizer (SP) has been used at an adequate dosage to manufacture ordinary concretes at a given slump (165 mm in Table 2) or at a given SP dosage (2.6 kg/m^3). Moreover, SCC mixtures with BFA or FA at a given slump-flow (about 740 mm in Table 3) were also studied through the rheological measurements [3] based on V-funnel, U-box and L-box (Table 4).

All concrete specimens were cured at room temperature (20°C). Moreover, the ordinary concrete has been also steam-cured at 55°C after a preliminary curing at room temperature of 3 hours and heated in 4 hours from 20°C to 55°C.

RESULTS

Ordinary concretes. Table 2 shows that the dosage of SP needed to produce fluid concretes at a given water-binder ratio of 0.39 with BFA is 1.2 kg/m^3 , 50% less with respect to the concrete with SF (2.6 kg/m^3). Moreover, when the same dosage of SP (2.6 kg/m^3) is used in the SF-mixture and in the BFA-concrete the water-binder ratio for the latter is much lower than in the former (0.31 vs. 0.39). These results are related to the larger size of BFA particles with respect to that of the SF particles (Fig.2), at the same spherical morphology as shown in Fig.1.

Figure 3 shows the compressive strength development of concretes cured at 20°C. When concretes are compared at the same water-binder ratio of 0.39, the compressive strength of the SF-mixture is significantly higher than that of the BFA-mixture, particularly at early ages (about 50% more at 1 day), whereas the difference decreases at longer ages (about 20% at 90 days). On the other hand, when the concretes are compared at the given slump of 165 mm and at the same superplasticizer dosage of 2.6 kg/m^3 , then the compressive strength of the BFA-mixture is higher than that of the SF-mixture: about 10% more at early and later ages.

Figure 4 shows the compressive strength values in the steam-cured concretes as a function of the time and confirms those obtained at room temperature (Fig.3).

All these results indicate that –at a given cost of the concrete mixtures– BFA (80 kg/m^3) performs better than SF (40 kg/m^3) in terms of compressive strength only if

the concrete mixtures are compared at a given slump and at the same dosage of superplasticizer (Table 2): due to the much lower mean size of SF with respect of BFA ($0.2\ \mu\text{m}$ vs. $5\ \mu\text{m}$), the water-cement ratio as well as the water-binder ratio is much higher in the SF-concrete than in the BFA-concrete.

Self-compacted concretes. Table 3 shows the composition of two SCCs, both with a portland cement content of about $245\ \text{kg/m}^3$, for which the same amount of FA or BFA ($245\ \text{kg/m}^3$) was adopted. At given w/c and slump-flow, the SCC mixture needs a significantly lower dosage of superplasticizer in the presence of BFA with respect to the corresponding concrete with FA (3 vs. $9\ \text{kg/m}^3$). This behavior is due to the most favorable spherical shape of BFA (Fig.1) with respect to that of the FA which includes some irregular particles specially in the range of the larger ones. Consequently, the BFA-SCC appears to be much more mobile than the corresponding FA-SCC (Table 4) in terms of much lower times needed in the V-funnel test (7 vs. 50 sec.) or in the U-box test (2 vs. 40 sec.) or in the L-box test (6 vs. 57 sec.). This property makes the BFA-SCC much more suitable, than the corresponding FA-SCC, to place self-leveling concretes for slabs and ground-floors or to fill more quickly the formworks specially if congested by steel reinforcements.

There is no significant differences in the compressive strength (Fig.5) at early ages, whereas at longer ages (≥ 14 days) the BFA-SCC performs slightly better than the corresponding FA-SCC probably for a more reactive behavior of the smaller particles of BFA.

All these results indicate that, except for some special applications where SCCs with particular flow-properties are needed for placing self-leveling mixtures, there is not a significant technical advantage in using BFA rather than FA. Moreover, the lower dosage of superplasticizer (3 vs. $9\ \text{kg/m}^3$) does not compensate for the higher cost of BFA with respect to FA.

CONCLUSIONS

The results of the present paper indicate that a beneficiated fly ash (BFA) with a low carbon content, very fine particle size (mean size of $5\ \mu\text{m}$), and a more favorable shape of the small spherical particles, could be used instead of silica fume (SF) for high performance concretes. In particular, the BFA-concrete mixture derives more advantage from the use of polycarboxylate-based superplasticizer (SP), when compared to the SF-concrete. Therefore, when the same SP dosage is adopted, the water-cement ratio as well the water-binder ratio were lower. The compressive strength tends to be higher when BFA ($80\ \text{kg/m}^3$) is used instead of SF ($40\ \text{kg/m}^3$) for the same cost.

On the other hand, when BFA is used instead of ordinary fly ash in the manufacture of self-compacting concrete (SCC), there is a significant technical advantage only where a self-leveling concrete is needed.

REFERENCES

- [1] Bittner, J.D., and Gasiorowski, “STI’s Commercial Beneficiation of high LOI Fly Ash”, Proceedings of 15th International Symposium on Management and Use of Coal Combustion Products, 2003, pp. 1-7.
- [2] Seedat, E.Y., “The new generation pozzolan for superior concrete”, private communication.
- [3] Uomoto, T., and Ozawa, K., “Recommendation for Self-Compacting Concrete”, Japanese Society of Civil Engineers, Shinjyuku, Tokyo, August 1999, pp. 1-70.

Table 1 – Chemical Analysis of Fly Ash (FA), Beneficiated Fly Ash (BFA), Silica Fume (SF) and Portland Cement (PC)

Chemical Analysis	Mass %			
	FA	BFA	SF	PC
SiO ₂	54.1	53.7	94.4	23.1
Al ₂ O ₃	28.9	34.2	0.2	5.1
CaO	2.0	4.1	0.8	62.9
Fe ₂ O ₃	3.7	3.8	0.7	0.9
K ₂ O	0.7	0.7	0.3	0.3
Na ₂ O	0.4	0.3	0.2	0.4
MgO	0.5	0.9	0.6	0.2
l.o.i.	6.6	0.8	2.0	2.0

Table 2 – Composition of concrete mixtures with SF or BFA at a given slump

Mix	Binder			Aggregate		Water kg/m ³	SP kg/m ³	w/b	w/c	Slump (mm)
	Portland cement kg/m ³	SF kg/m ³	BFA kg/m ³	Gravel 4-30 mm kg/m ³	Sand 0-4 mm kg/m ³					
SF	360	40	0	1074	795	157	2.6	0.39	0.43	160
BFA	320	0	80	1074	796	157	1.2	0.39	0.48	170
BFA*	325	0	81	1090	810	127	2.6	0.31	0.39	170

w/b = water/binder

Table 3 – Self-compacting concretes with FA or BFA

SCC mixture	Cement kg/m ³	Sand 0-4 mm kg/m ³	Gravel 4-30 mm kg/m ³	Water kg/m ³	SP kg/m ³	w/c	w/b	Slump flow mm	FA kg/m ³	BFA kg/m ³
FA-SCC	244	858	925	166	9.0	0.68	0.34	750	244	—
BFA-SCC	246	869	938	163	3.0	0.66	0.33	750	—	244

Table 4 – Rheological properties of FA-SCC and BFA-SCC
at the same slump-flow of 750 mm

SCC type	Times (sec) for the following test:			
	V-tunnel	U-box	L-box	Slump flow
FA-mix	50	40	57	50
BFA-mix	7	2	6	7

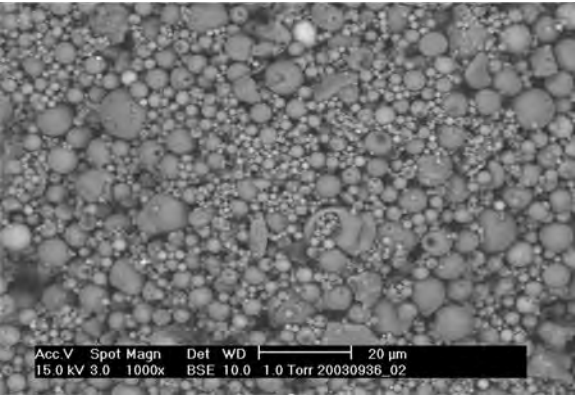


Fig. 1—SEM microscopy of BFA.

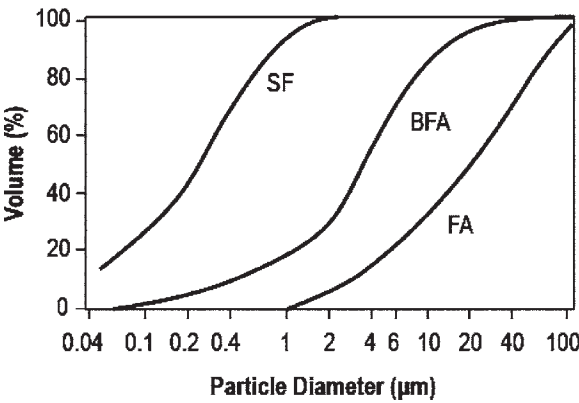


Fig. 2—Particle size distribution of SF, FA, and BFA.

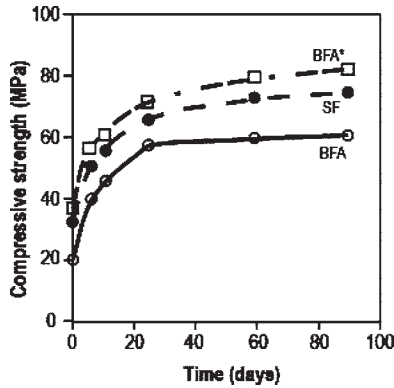


Fig. 3—Compressive strength versus time at 20 °C for concretes with silica fume (SF) and beneficiated fly ash at the same w/c (BFA) or at the same dosage of superplasticizer (BFA*).

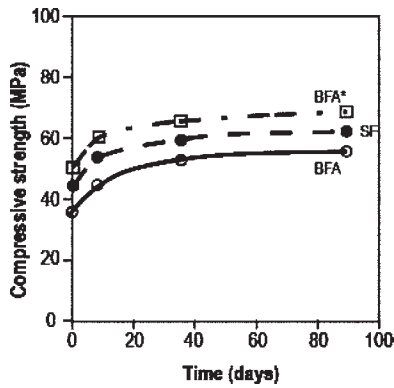


Fig. 4—Compressive strength versus time for steam-cured concretes with silica fume (SF) and beneficiated fly ash at the same w/c (BFA) or at the same dosage of superplasticizer (BFA*).

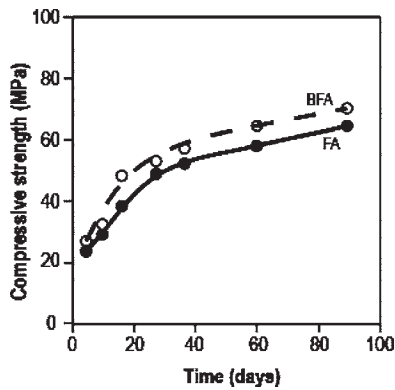


Fig. 5—Compressive strength versus time for SCC with fly ash (FA) or beneficiated fly ash (BFA).

Alkaline Activation of Synthetic Gehlenite Glasses

by J. Deja

Synopsis: Alkaline activation of granulated blast furnace slags by highly concentrated solutions of sodium or potassium ions has been a subject of numerous investigations for many decades. Irrespectively of the type of activator used, the so-called C-S-H phase formed is very compact, of low C/S ratio, rich in sodium, aluminium and magnesium and is predominant hydration product. Properties of AAS pastes, mortars and concretes strongly depend on the chemical and phase composition of the slag.

This paper presents the properties of the alkali activated pastes and mortars produced on the base of synthetic alumino-silicate glasses of gehlenite type. Setting time, mechanical properties and heat of hydration of the gehlenite-glass pastes are presented. Detailed studies of phase composition, microstructure and structure of alkali-activated gehlenite glasses are presented in the paper. Alkaline activation of gehlenite slag glasses is influenced by molar ratio $\text{Al}_2\text{O}_3/\text{SiO}_2$ of the slag and concentration of NaOH. The hydration process is much quicker in the case of gehlenite type glasses than for typical industrial melilite granulated blast furnace slags. The results of XRD and SEM/EDS examinations show that in gehlenite type pastes amorphous C-A-S-H phase containing high amount of sodium, silicon and aluminium are the dominating hydration product.

Keywords: alkaline activation; gehlenite glass; microstructure; paste; phase composition

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INTRODUCTION

The use of by-product and waste materials plays also a very important role in solving present ecological problems (for example: reduction of CO₂ emission) and could enlarge the range of building materials and partly replace high energy-consuming portland cement [1-7].

Alkaline activation of aluminosilicate glasses is one of the new methods of special low-energy binders production. Ground granulated blast furnace slag is the main material which is used in this technology, but other glassy materials can be also activated (for example: fly ash, incineration slags). Sodium carbonate, water glass or sodium hydroxide are typical alkaline activators. Depending on the initial raw materials used, the type of activator and the conditions of curing, the products of hydration in such binding systems can be minerals, such as C-S-H and C-A-S-H in very dense and amorphous form. Changes of A/S and C/S ratios in these systems make it possible to form calcium-alumino-silica gel (the main product of so-called geopolymers), which can be good precursor for zeolite formation. High strength, very high durability in aggressive solutions and very dense microstructure are the most interesting properties of these materials. It gives many opportunities for their application (immobilization of heavy metals and toxic substances, anti-filtration screens etc.) [8-17].

The objective of the study consisted in investigating the properties of alkali-activated pastes based on synthetic gehlenite glasses. Influence of activator content and curing conditions on the hydration process was determined. Mechanical properties as well as heat of hydration of the pastes were measured. The phase composition and microstructure observations of the hydrated pastes were done.

EXPERIMENTAL

Characteristics of starting materials

Ground granulated synthetic slags and alkaline activators were used for paste preparation.

Ground granulated synthetic glasses (slags)

Proper mixtures of chemically pure oxides, hydroxides and carbonates were melted in induction furnace. Melted slag was cooled in water and later on ground in a ball mill to the specific surface of 400±10 m²/kg. The chemical compositions of the synthetic slags are provided in Table 1 and their XRD patterns in Figure 1. The increased level of background (shoulder) at 25 - 35° 2Θ, visible in the XRD patterns, points to their high degree of vitrification. Content of crystalline phases (gehlenite and grossite) in the slags was very limited (1.5 - 2%).

Preparation of pastes

The amount of alkaline activator in the activated slag mixes differs according to the alkalinity and fineness of the slag and according to the required strengths of hardened concretes. Usually, 2% to 7% of Na₂O (calculated) of the slag content is necessary.

Małolepszy has found that Na_2CO_3 is especially suitable for slags rich in C_2MS , and NaOH is a good activator for slags rich in C_2AS [2].

In the presented study solution of sodium hydroxide was used as alkaline activator of the slags. 1.2, 3.5 and 7.0% Na_2O addition (calculated to the weight of the slag) was fully dissolved in additional water and after that mixed with slag in a laboratory mixer. After mixing the pastes were cast into 10x10x60 mm moulds. The composition of the pastes and their notation are presented in Table 2.

After the preliminary consolidation, all the samples were matured in the moulds for the first 24h, at 20°C and 100% RH.

Subsequently, after 24h the pastes were demoulded and stored in curing chamber (20°C, 100% RH). Part of the samples after 24 hours of preliminary curing was autoclaved for 12 hours in 230°C.

RESULTS AND DISCUSSION

Setting time and mechanical strength of pastes

The physical properties of the pastes are presented in Table 3.

Activity of gehlenite glasses is very high. The setting times of the pastes are extremely short (10 sec. to 2 minutes), but an interesting levels of strength was observed (compressive strength even over 110 MPa). Pastes containing synthetic glass with $\text{Al}_2\text{O}_3/\text{SiO}_2$ ratio 1.0 presented significantly higher strength than those with ratio 1.2. Positive influence of higher concentration of alkaline activator is also well visible. The significant reduction of the strength for the autoclaved samples was observed (over 40%). It is probably the effect of higher crystallization ratio in the autoclaved pastes.

Heat of hydration

The heat evolution on hydration of pastes prepared at water to solid ratio 0.5 was followed within the 24h by microcalorimetry (BMR differential microcalorimeter constructed in the Institute of Physical Chemistry, Polish Academy of Science). 1.2, 3.5 and 7.0% Na_2O was added as activator. The initial temperature was kept constant at 25°C. The calculated heat values are presented in Table 4.

On the heat evolution curves of the pastes, in all cases a strong initial peak appears within the first few minutes. It is commonly accepted that very quick hydration of slags activated by sodium hydroxide is an effect of high concentration of OH^- , which can break the bonds inside the glass structure [18]. It is clear that NaOH activated synthetic glasses do not show an appreciable induction period. Total heat evolution during 24 hours of hydration is very low (89 - 141 J/g) and typical for alkali-activated slag binders.

Powder X-Ray Diffraction (XRD) Analysis

Powder XRD analysis was performed on ground alkali-activated slag pastes with a Philips Diffractometer X $\&\#8217$ Pert. The phases were identified from the characteristic peak data base of the Powder Diffraction File of the Joint Committee on Powder Diffraction Standards. The XRD patterns are presented on Fig. 2.

The XRD patterns for the gehlenite glass containing pastes cured in natural conditions are significantly different from typical pastes based on industrial granulated blast furnace slags (melilite glass structure in such slags is dominating). Practically, no C-S-H phase is

present in tested pastes. Content of C-S-H phase is very low, but amount of C-A-S-H phase is much higher. Amount of this phase is significantly higher for synthetic glass with alumina/silica ratio 1.2. Crystalline form of the hydration products in this case is distinct better than for typical melilite slags. Partial carbonation of C-A-S-H phase and aragonite formation was also detected.

Hydrothermal conditions influenced the phase composition of the pastes. On Figs. 3, 4 the XRD patterns of pastes AN3 and BN3 cured at 20°C and autoclaved are presented. For paste AN3 (A/S=1.0) after autoclaving only slightly higher degree of crystallization and higher content of C-S-H phase was observed. For paste with higher content of Al_2O_3 (BN3) autoclaving leads to formation of better crystallized katoid (hydrogarnets). Presence of such mineral can reduce mechanical properties of the pastes (see the results of compressive strength tests).

Scanning Electron Microscope (SEM) Observations

The microstructure of the pastes was examined under a Jeol Scanning Electron Microscope. Micrographs were taken of represent-five areas at magnifications ranging from 1,000x to 5,000x. For all samples EDS analyses in various points of the microstructure were also done. In Figures 5 - 8 the typical examples of the microstructure are presented.

The SEM observations confirmed that under the influence of the activator solution an interesting microstructure is observed in pastes AN3 (synthetic glass with alumina/silica ratio 1.0). Very amorphous, dense microstructure of C-A-S-H phase rich in aluminium, silicon and sodium is easily detected. Such kind of product can be good precursor for zeolite formation [9, 19, 20].

Huge amounts of described C-A-S-H gel are also visible in gehlenite type samples with $\text{Al}_2\text{O}_3/\text{SiO}_2 = 1.2$ (paste BN3). Crystalline form of the hydration products (also zeolites) in this sample is better, which confirms the results of XRD measurements. In both cases autoclaving process gives better ordered microstructure with higher content of crystalline phases.

Nuclear Magnetic Resonance examinations

Using the NMR (Nuclear Magnetic Resonance) technique to analyse the ^{29}Si spectrum many authors confirmed that in slag-alkaline pastes C-S-H phase is characterised by the occurrence of a chain structure in which the arrangement of chain elements is repeated at every fourth tetrahedron [6]. This is a characteristic element of the structure of numerous calcium silicates and is known as dreierketten; it is linked to the cationic substructure, and thus to the octahedral oxygen-calcium layer. The two side elements are coordinated directly with the octahedral layer while the middle tetrahedron remains outside. It is precisely this tetrahedron that participates in establishing a bridge between neighbouring octahedral layers.

If we assume that a considerable part of Si^{4+} ions in bridging tetrahedrons is replaced by aluminium, the presence of a large quantity of Na^+ ions incorporated in the C-S-H phase is balancing the charge. This has been confirmed by the research carried out by Nocun-Wczelik [21].

MRJ impulse spectrometer of Bruker equipped with a 7T superconductive magnet and a probe, with the sample spinning at a magical angle (MAS) was used. MAS-MRJ spectra were measured of ^{29}Si (frequency of 59.517 MHz, sample spinning frequency of 4 kHz). The ^{29}Si MAS-NMR spectra of initial slag samples contain a broad line at (−80) ppm, which is characteristic for amorphous silicate, covering a range typical for silicon-oxygen

anions type Q^1 and Q^2 (end and middle tetrahedrons in the chain structure). Due to the amorphous (vitreous) nature of slags, the splitting of these bands does not occur. Neither do the spectra indicate any presence of a three-dimensional structure (Q^4), cross-linking (Q^3), or isolated tetrahedrons (Q^0).

For selected pastes described earlier the ^{29}Si MAS-NMR spectra were measured (Fig.9 and 10).

The paste spectra correlate very closely with the spectra of slag. There is no visible splitting into individual bands, nevertheless peaks become prominent and shift towards an increase in the share of Q^2 anions, which corresponds to the disintegration of silicate-type glass and the appearance of a larger number of chains, most probably constituting a part of the C-A-S-H structure. In the case of materials subjected to autoclaving or hydrated for 28 days, band peak becomes sharper and shifts towards a range corresponding to Q^2 ; this may be explained by the increase in the chain-like quantity of the product, and also by the increase in the length of chains. As in the case of the initial material, the presence of a three-dimensional structure (Q^4), cross-linking (Q^3) or isolated tetrahedrons (Q^0) was not found.

CONCLUSIONS

On the basis of the results of the research the following conclusions can be made:

- Alkaline activation of gehlenite slag glasses is influenced by the $\text{Al}_2\text{O}_3/\text{SiO}_2$ molar ratio of the slag and concentration of Na_2O .
- The hydration process is much quicker in the case of gehlenite type glasses than for typical industrial melilite granulated blast furnace slags. Total heat of hydration measured during 24 hours of the process is much lower than for typical portland cements.
- The highest strength has been found for pastes based on gehlenite type glass with alumina to silica ratio 1.0.
- The results of XRD and SEM/EDS examinations show that in gehlenite type pastes amorphous C-A-S-H phase containing high amount of sodium, silicon and aluminium are dominating hydration product.
- ^{29}Si MAS-NMR spectra shows that ordering corresponding to Q^2 structure with tetraedra rich in Al is dominating in the hydration products. This may be explained by the increase in the chain-like quantity of the product, and also by the increase in the length of chains.
- Calcium-alumino-silica gel observed in gehlenite pastes can be good precursor for zeolite formation. SEM and XRD examinations of autoclaved samples with A/S ratio 1.2 confirmed this.

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REFERENCES

- [1] Głuchowski W.D., Gruntosilikatni wirobi i konstrukcii Budiwilnik, Kijew 1967 (in Russian) .
- [2] Małolepszy J., Hydration and properties of alkali-activated slag binders. Zeszyty Naukowe AGH 53, 1989 (in Polish).
- [3] Krivienko P.V., Alkaline cements. First Int.Conference “Alkaline Cements and Concrete” Vol. I, No. 11, Kijew 1994.
- [4] Talling B., Brandsteter J., Present state and future of alkali activated slag concretes. 3rd Int. Conference “Fly Ash, Silica Fume, Slag and Natural Pozolans in Concrete” Vol. 2, s. 1514, Trondheim 1989, Norway, ACI SP114.
- [5] Roy D. M., Alkali-activated cements. Opportunities and challenges, Cem. Concr. Res. 29, 249 (1999).
- [6] Deja J., Durability of alkali-activated slag mortars and concretes. Ceramika 83, 2004 (in Polish).
- [7] Kurdowski W., Problems of cement application in radioactive wastes immobilization, CWG, 6, 1992 (in Polish).
- [8] Glasser F.P., Progress in the immobilization of radioactive wastes in cement. Cem. Concr. Res. 22 (1992), pp. 201-216.
- [9] Davidovits, J., Properties of Geopolymer Cements, Proceedings of the First International Conference Alkaline Cements and Concretes, Edited by P.V. Krivenko, vol. I, Kiev 1994, pp. 131-149.
- [10] Nocuń-Wczelik W., Immobilization of heavy metals, CWB 5.1997 (in Polish).
- [11] Małolepszy J., Kopia B., Immobilization of heavy metals in synthetic, melilite type alkali-activated slag binder, Ceramika 54, 1997 (in Polish).
- [12] Fernandez-Jimenez A., Palomo A., Activation of fly ashes: a general view, Proceedings of 8th CANMET/ACI International Conference on Fly Ash, Slag and Natural Pozzolans in Concrete, Las Vegas 2004, pp. 351-366.
- [13] Deja J., Immobilization of Cr^{6+} , Cd^{2+} , Zn^{2+} and Pb^{2+} in alkali-activated slag binders, Cem. Concr. Res. 32 (2002), pp. 1971-1979.
- [14] Spence R.D., Bostick W.D., McDaniel E.W., Gilliam T.M., Shoemaker J.L., Tallent O.K., Morgan I.L., Evans-Brown B.S., and Dodson K.E., Immobilization of Technetium in Blast Furnace Slag Grouts, [in:] Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Vol. 2, Proceedings of the 3rd International Conference, Trondheim, Norway, ACI SP 114, 1989, pp. 1579-1596.
- [15] Baś B., Bobrowski A., Deja J., Małolepszy J., Utylizacja of galvanic wastes from oxydation processes, 1st Conference “Biomaterials, Materials in Helth and Environmental Protection”, Zeszyty Prac Komisji Nauk Ceramicznych PAN, Kraków 1994, pp. 347-351 (in Polish).
- [16] Krivenko P., Skurchinskaya J., Lavrinenko L., Starkov O., Konalov E., Physico-chemical bases of radioactive wastes immobilization in a mineral-like solidified stone, [in:] P.V. Krivenko (Ed.), Alkaline Cements and Concretes Proceedings of the 1st International Conference, VIPOK Stock Co., Kiev, Ukraine (1994) 1095-1106.

- [17] Brylicki, W., Małolepszy, J., Stryczek, S., Alkali-activated slag cementitious material for drilling operations, 9th Congress on the Chemistry of Cement, New Delhi 1992, vol. III, Th. II, pp. 312-318.
- [18] Shi, C., Day, R.L., A Calorimetric Study of Early Hydration of Alkali-Slag Cements, Cem. Concr. Res., vol. 25, No. 6, 1995, pp. 1333-1346.
- [19] Palomo, A., Grutzeck, M.W., Blanco, M.T., Alkali-activated fly ashes. A cement for the future, Cem. Concr. Res., vol. 29, 1999, pp. 1323-1329.
- [20] Deja, J., influence of slag and metakaolin on the properties of mineral pastes, 2nd International Conference on Non-Traditional Cement and Concrete, Brno 2005.
- [21] Nocuń-Wczelik, W., Effect of Na and Al on the phase composition and morphology of autoclaved calcium silicate hydrates, Cem. Concr. Res 29 (1999) pp. 1759-1767.

Table 1. Composition of synthetic slag glasses.

Slag glass	Content of oxides[%]			Amount of moles [on 100g]			Moles ratio Al ₂ O ₃ /SiO ₂
	CaO	Al ₂ O ₃	SiO ₂	CaO	Al ₂ O ₃	SiO ₂	
A	40.91	37.38	21.91	0.73	0.37	0.37	1.0
B	38.08	41.53	20.39	0.68	0.41	0.34	1.2

Table 2. Composition of slag pastes.

Sample	Content of activator, % Na ₂ O	Water/Binder
AN1	1.2	0.40
AN3	3.5	0.40
AN7	7.0	0.40
BN1	1.2	0.38
BN3	3.5	0.38
BN7	7.0	0.38

Table 3. Setting time and mechanical strength of the pastes.

Sample	Setting time	Compressive strength – curing 28 d., [MPa]	Compressive strength – autoclaving, [MPa]
AN1	ca.10 sec.	78.5	n.d.
AN3	ca.10 sec.	94.5	50.4
AN7	ca.10 sec.	112.7	n.d.
BN1	2 min.	20.0	n.d.
BN3	30 sec.	48.5	27.3
BN7	15 sec.	69.8	n.d.

Table 4. Total heat evolution values.

Sample	Heat of hydration, J/g
AN1	89
AN3	103
AN7	140
BN1	98
BN3	124
BN7	141

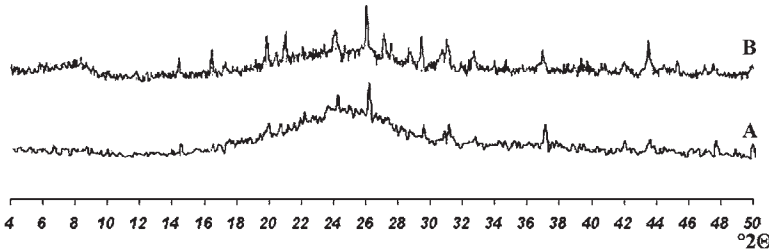


Fig. 1—XRD patterns of synthetic slag glasses.

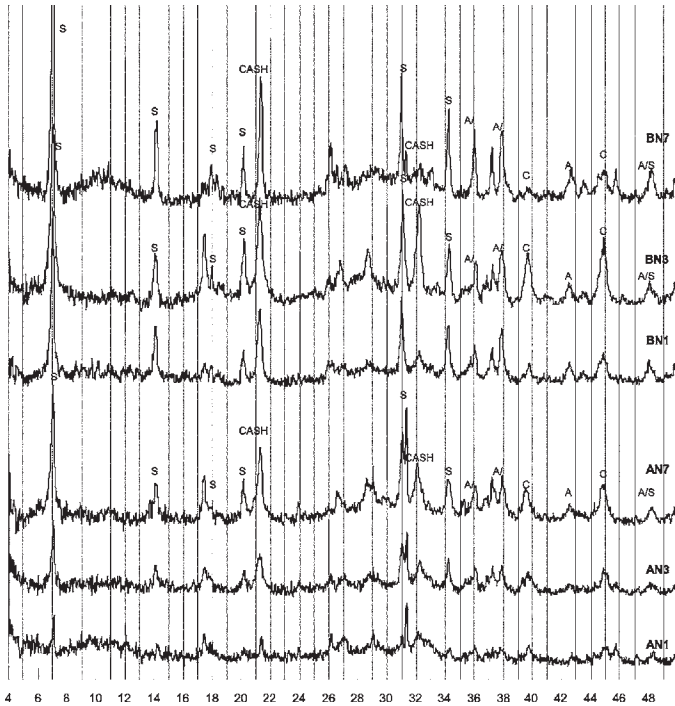
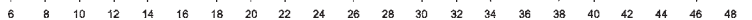


Fig. 2—XRD patterns of the pastes cured at 20°C. Notations: CASH = C-A-S-H; A = aragonite (CaCO₃), C = calcite (CaCO₃), S = stratlingite (Ca₂Al₂SiO₇·8H₂O).



T = tobermorite; G = gehlenite.



T = tobermorite.

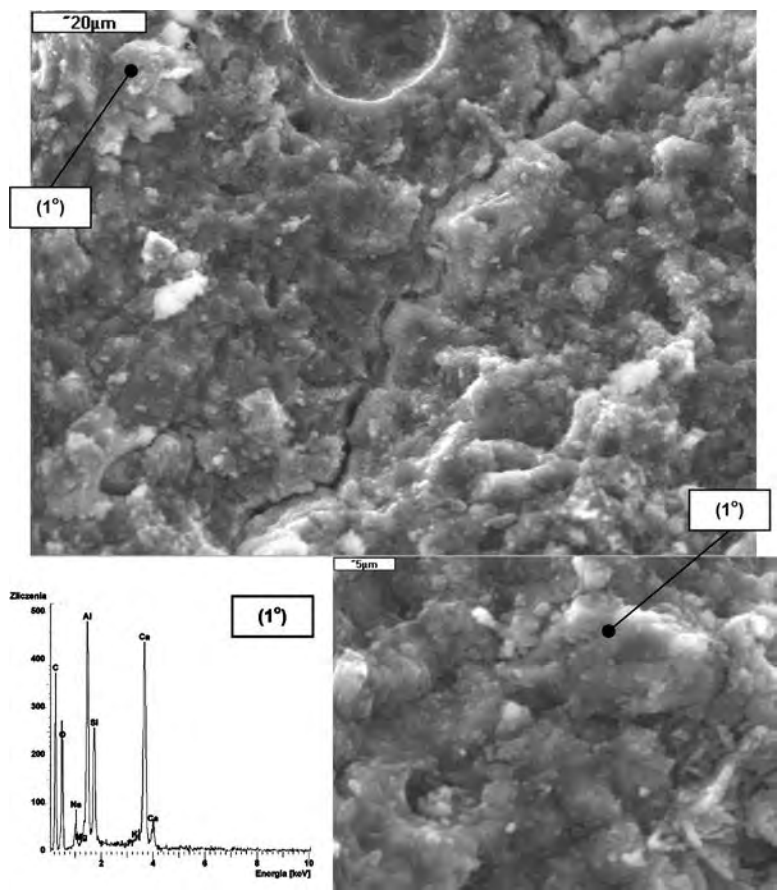


Fig. 5—SEM-EDS from slag glass paste A – typical microstructure of sample AN3 (magnification 1,000 and 3,500x).

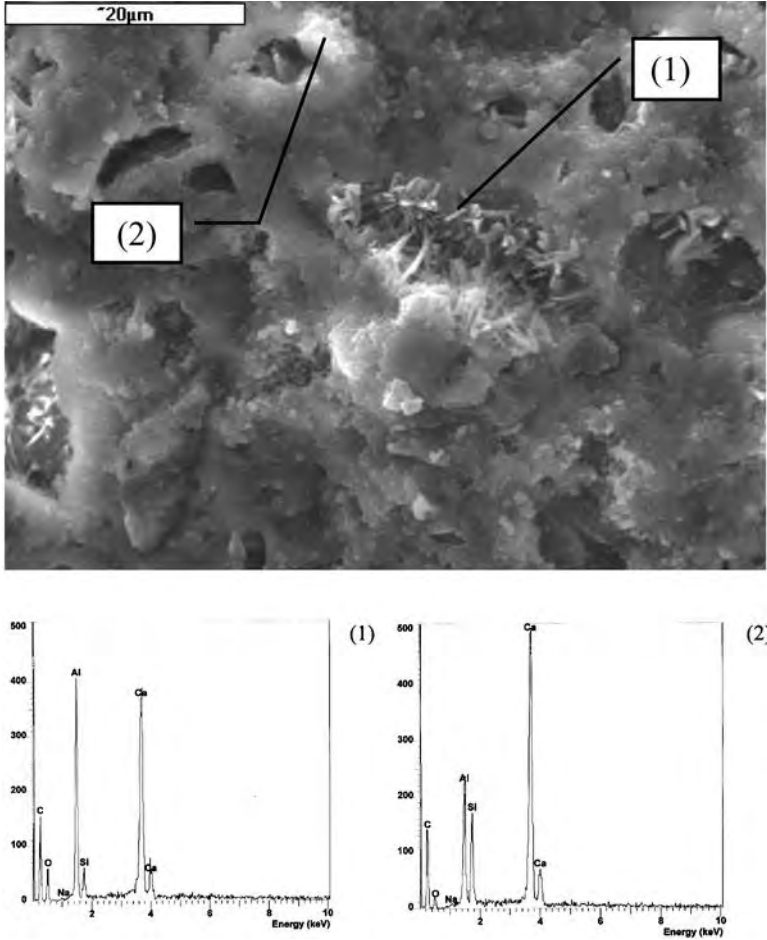


Fig. 6—Microstructure of hydration products and EDS analyse of AN₃ paste after autoclaving (magnification 2,000x).

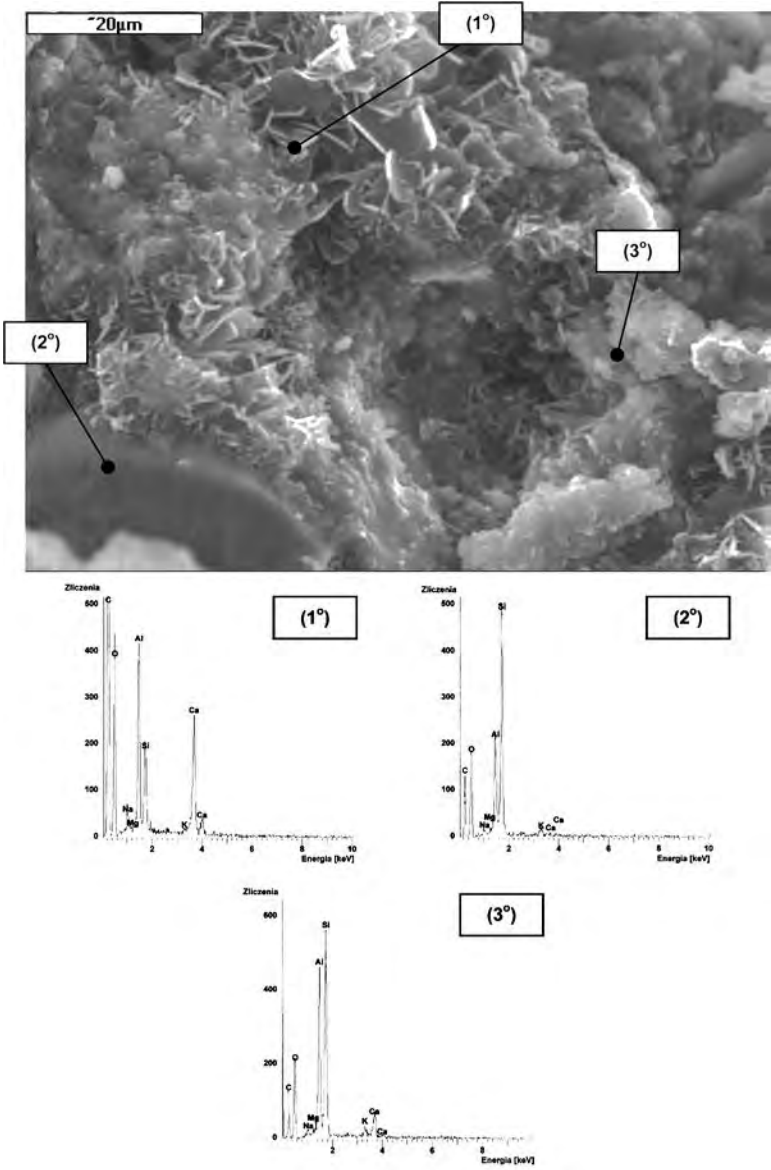


Fig. 7—SEM-EDS from slag glass paste BN3 (curing at 20°C) – typical microstructure (magnification 1,500x).

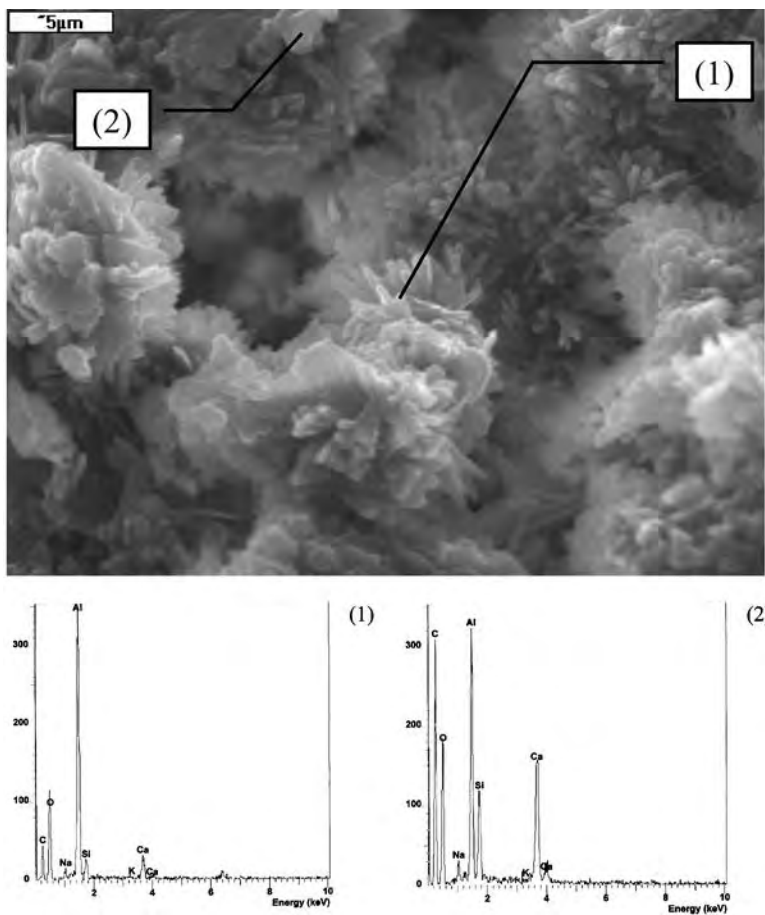


Fig. 8—Hydration products and EDS analysis of BN3 paste after autoclaving (magnification 1,000 and 3,500x).

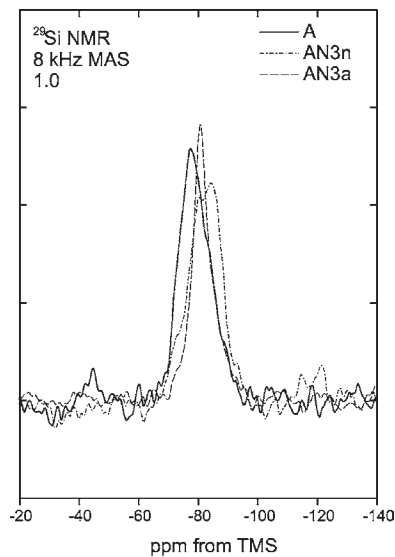


Fig. 9— ^{29}Si NMR spectra for synthetic gehlenite glass A and paste AN₃ (cured at 20°C and autoclaved).

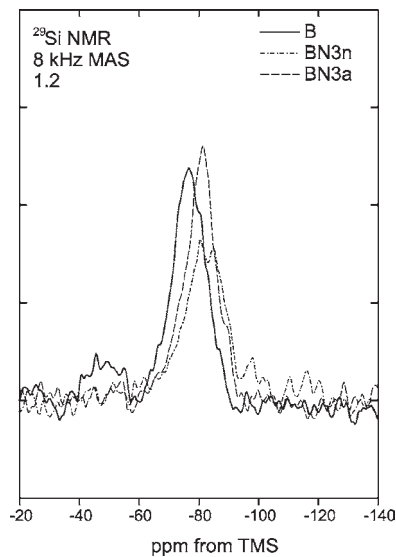


Fig. 10— ^{29}Si NMR spectra for synthetic gehlenite glass B and paste BN₃ (cured at 20 °C and autoclaved).

Bending Fatigue Strength of Roller-Compacted Concrete with Fly Ash as a Substitute for Fine Aggregate

by E. Matsuo, K. Hino, and S. Hamada

Synopsis: In Japan, sea sand is used frequently as fine aggregate for concrete. The mining of sea sand has been more difficult year by year for environmental reasons. This would be a serious problem in manufacturing concrete. The amount of fly ash has been increasing gradually in Japan, therefore the utilization of fly ash as the substitution of fine aggregate has a significant advantage. In this study, bending fatigue test of roller compacted concrete was carried out, in which all fine aggregate was replaced with fly ash. The obtained S-N curve was compared with that of normal concrete and general roller compacted concrete. Scatter of fatigue strength was also determined.

Keywords: bending strength; fatigue; fly ash; roller compacted concrete

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INTRODUCTION

In Japan, the quantity of fly ash discharged from coal-fired power plants shows the upward tendency with energy demand. It is estimated to exceed 10 million ton per year in 2010 [1], consequently, the shortage of the disposal ground, the sudden rise of disposal expense, and the problem of environmental load would be aggravated. In Japan especially in the western region, sea sand has occupied the great portion of fine aggregate for concrete. Sea sand mining has been limited because of the destruction of ocean environment on account of surplus mining. Therefore, the lack of fine aggregate has been a serious problem.

From such a background, research in which fly ash is used as a part of fine aggregate for concrete are carried out extensively in Japan. In USA, some papers were reported about the strength and the durability of roller compacted concrete (RCC) and concrete pavement in the past [2], [3], [4], [5]. For example, they were about the deice salt scaling resistance, the drying shrinkage, etc. In order to increase those durability, there are also studies in which fly ash and silica fume were used [6], [7], [8]. Moreover, various static strengths, drying shrinkage and freeze-thaw durability of RCC in which fly ash was used in large quantities were investigated [9], [10]. There are studies also for general concrete pavement in which fly ash was used, and the fatigue strength and the durability were verified [11], [12]. However, cement was replaced by fly ash in those cases. The new point of this study is that fine aggregate in RCC was replaced by fly ash, and which did not use fine aggregate at all. This is based on the aggregate situation of western part of Japan. Furthermore, it is also new knowledge to have clarified the fatigue property of this kind of RCC.

In this study, authors have developed RCC, in which all fine aggregate is replaced with fly ash. This RCC is abbreviated as NS-FA-RCC (No Sand Fly Ash RCC). The mix design method of [13] was used. Next, workability at site construction was confirmed for the concrete on the basis of the obtained optimum mix proportion in the laboratory [14]. The results, the static bending strength of the optimum mix proportion in the laboratory satisfied the design mix strength of 5.7MPa, which is the ordinary mix strength of general RCC at 28 days in Japan. At the site, the desired compacting ratio (96%), which is defined as “actual density / mix design density”, was obtained by increasing the water powder ratio by 3% in comparison to the optimum mix proportion in the laboratory. The target bending strength of 5.7MPa was also obtained in 28 days.

Ninety five percent of the pavements in Japan are constructed with asphalt concrete. The rest (5%) is concrete pavement. The ratio of concrete pavement is less than that of other western Europe and American countries. Japan is now under a long-term depression therefore, when constructing infrastructures, investigation of life cycle cost has become more important. In that case, authors are expecting that the demand for concrete pavement, which is superior in durability, would increase.

One of the design principles of concrete pavement in Japanese specification, including RCCP, is based on bending fatigue strength. The repeated stresses are developed by traffic loads, and the number is also obtained from the traffic volumes. Bending fatigue strength of concrete pavement is very important in order to estimate the life cycle. Especially, when the concrete is used as a new material, fatigue characteristic must be evaluated.

In this paper, bending fatigue test of NS-FA-RCC was carried out. Obtained S-N curves were compared with those of normal concrete and general RCC, which were carried out in other studies. Scatter of fatigue strength was also obtained.

TEST METHOD

Materials and Mix Proportion

In the laboratory, high early strength Portland cement was used in the concrete. Coarse aggregate was andesite macadam. The maximum size was 20mm, the density was 2.71g/cm^3 , and the percentage of solid volume was 59%. Fly ash was used from its source. Table 1 shows the physical properties of the fly ash. It corresponds to the 2nd class of JIS (Japanese Industrial Standards) except for ignition loss and the index of activity at 28days. Fig.1 shows the grading curve of the fly ash.

The target compacting ratio of NS-FA-RCC was 96%, and the desired bending strength at 28 days was 5.7MPa [15]. Table 2 shows the mix proportions and static bending strength. "The mass of unit volume of coarse aggregate" in this table means the volume ratio of coarse aggregate against the percentage of solid volume. Materials were mixed by a compulsion type mixer. At first, coarse aggregate and fly ash were mixed for 60 seconds. Then, the cement was added and mixed for 60 seconds, and the water added and mixed for another 60 seconds. The specimen size was $15 \times 15 \times 53\text{cm}$. The concrete was cast in two layers, and was compacted with vibration. Specimens were cured in water with temperature of 20°C until the start of the fatigue test. The fatigue test was initiated on and after the age of 180 days.

Test Method

The loading span was 45cm and divided in thirds (see Fig.2) in both the static and fatigue test. A sine wave was applied for fatigue testing with a frequency of 3Hz. When

expressing the stress of fatigue strength, the stress was evaluated by the stress amplitude or the maximum stress ratio. In this study, stress amplitude is employed in consideration that the minimum load cannot be disregarded.

Stress amplitudes were 0.4, 0.5, 0.6, 0.65 and 0.7. The minimum load was 10% of the static bending strength. When the specimen did not fail, the test was run out at 2 million repetitions.

TEST RESULTS

S-N Curve

Table 3 shows the results of the fatigue test. As shown in the table, data for each stress amplitude shows the characteristic scatter of fatigue phenomenon, which was then analyzed by the theory of an order statistic [16]. Probability of survival $P(N)$ in the table was obtained by a formula (1) using an order statistic.

$$P(N) = 1 - r / (n + 1) \quad (1)$$

where,

r : the number counted from the smallest

n : the total number of specimens under the same stress amplitude

When the data included the run-out specimens at 2 million repetitions, the data processing method was especially important. In this study, the method to estimate $P(N)$ was applied, which investigated the order statistic rationally by Hamada [17]. That is, when m of n specimens did not fail by 2 million repetitions, the experiment was presented with specimens of $(n+1)$, and it was considered that the $(n-m+1)$ th specimen failed at 2 million repetitions. In that case, $P(N)$ was obtained from a formula (2).

$$P(N) = 1 - r / (n + 2) \quad (2)$$

Various models have been proposed as a distribution type of the fatigue life of concrete, for example, the logarithmic normal distribution, the Weibull distribution and McCall distribution and so on [16]. In this study, logarithmic normal distribution was applied, which is often used generally in analyzing fatigue performance. The relationship between $P(N)$ and N could be expressed by formula (3). The average fatigue life $m(\log N)$ which corresponds $P(N)=0.5(t=0)$ and the standard deviation $\sigma(\log N)$ are expressed with formula (4) and formula (5), respectively.

$$t = A \log N + B \quad (3)$$

$$m(\log N) = -B / A \quad (4)$$

$$\sigma(\log N) = 1/A \quad (5)$$

Here, A and B are the constants which could be found from an experiment. Notation “t” is the distance from the symmetry axis of a normal distribution curve, which could be found in a standard regular integration table by the P(N). Fig. 3 plots the relation between P(N) and N on logarithmic normal probability paper. In this figure, data are generally in a straight line, therefore data of each stress amplitude can be considered to belong to a population of the logarithmic normal distribution.

Table 4 shows the regression formulas, in which the relationship between t and N at each stress amplitude by the method of least squares in the direction of N axis, the average of fatigue life $m(\log N)$ and the standard deviation $\sigma(\log N)$ obtained from formulas (4) and (5). By determining the relation of $m(\log N)$ and S in this Table by the method of least squares in direction of N axis, the S-N curve in $P(N)=0.5$ could be obtained by formula (6).

$$S = -0.140 \log N + 1.302 \quad (6)$$

Estimation of S-N Curve

Table 5 shows the formulas of S-N curves at a probability $P(N)=0.5$ obtained by Koyanagawa [18], Inoue [19], Kobayashi [20], [21], where the fatigue strength at 2 million repetition, range of the standard deviation $\sigma(\log N)$ of fatigue life at each stress amplitude and the standard deviation of scatter in the direction of N axis from the S-N curve of all measured points. Fig.4 shows S-N curves when the minimum stress is 10% of the static strength in Table 5. From Fig.3, the S-N curve of NS-FA-RCC has a large inclination as compared with that of general RCC and RCC using recycled aggregate (type 1). Larger differences of fatigue strength occur in the region of larger number of repetitions. That is, as compared with general RCC, the durability of NS-FA-RCC is lower.

If fatigue strength at 2 million repetitions is designed at a survival probability of 50%, half of the structures may reach the fatigue limit state within the service period. Therefore, the fatigue limit of a higher survival probability should be evaluated. In Fig.5, the ranges of $\pm 1 \sigma$ (S-N) and $\pm 2 \sigma$ (S-N) are shown, which were calculated by the standard deviation σ (S-N) (see Table 5) in the direction of the N axis. The line of -1σ (S-N) and -2σ (S-N) is equivalent to the S-N curve, in this case P(N) is 0.86 and 0.98, respectively. S (fatigue strength at 2 million repetitions) is 0.30 when P(N) is 0.86, and it is 0.19 when P(N) is 0.96, then the fatigue limit strength in design can be determined.

Estimation of Scatter

a) Static Bending Strength

Table 6 shows the coefficient of variation for static bending strength of normal concrete (using general aggregate), RCC (using recycled aggregate, type 1) by Kobayashi and the present NS-FA-RCC. The age at static bending test of RCC was longer than that of normal concrete. The strength of RCC may become stable, and the coefficient of variation of static bending strength of RCC was evidently large as compared with normal concrete. This seems to be caused by the scatter of concrete density, that resulted from the peculiar compacting property of RCC.

Fig. 6 shows the relationship between compacting ratio and static bending strength. Paying attention to the strength at the compacting ratio around 95~97%, scatter is very large even when compacting ratios are almost the same. From this, it is estimated that the scatter of bending strength of NS-FA-RCC would be influenced by other factors excluding the compacting ratio.

For example, the influence of mixing method will also be large in such a case where the water powder ratio is extremely small, and the powder was a blend of cement and fly ash. Even if materials are mixed sufficiently, the amount of water may not be necessary contributed for the hydration reaction with cement particles. In that case, the surface of fly ash particles first adsorbs water in mixing, and water, which do not contribute to hydration. This may cause the scatter of strength of mortar (or paste) matrix. It is an important and interesting problem to confirm and analyze this phenomenon quantitatively and examine the casting order of materials in mixing. Moreover, since the water powder ratio was small, autogenous shrinkage strain might increase, and the non-uniformity may increase in the internal structure of hardened.

As shown in Table 2, the unit volume of coarse aggregate of NA-FA-RCC is so large that it is almost the percentage of solid volume. Therefore, the boundary area of mortar and coarse aggregate is larger than that of general RCC. The location where coarse aggregate contacts directly, it increases the boundary area, and adhering area between coarse aggregate and paste decreases.

When the unit volume of coarse aggregate is large, like for this concrete, it can be estimated that the adhesion between mortar and coarse aggregate and the distribution at a location where coarse aggregates contact each other differ significantly between specimens. This may decrease the adhering strength, develop the locations leading to stress concentration, and cause the variation of static strength.

b) Repetitions at Failure in Fatigue Test

From Table 5, the standard deviation at failure in the fatigue test of NA-FA-RCC is the largest as compared with previous studies. Since the scatter of fatigue life of the concrete is closely related to the scatter of static strength, the above-mentioned scatter of static strength may result also in the scatter of the repetition at failure.

Fig.7 shows the relationship between each stress amplitude and the coefficient of variation of the repetition of failure. The larger the stress amplitude, the larger the scatter of the repetition at failure is. In NS-FA-RCC, since coarse aggregate exists in the state near contact, the internal structure of mortar becomes non-uniform. Therefore, when the load repetition is large, the adhering force at boundary of mortar and coarse aggregate changes easily. As a result, the weak points against bending stress also easily occur, which leads to a decrease of the resistance against the bending fatigue load. This is considered to also be the reason of the scatter of static strength.

CONCLUSIONS

In this study, authors have developed new RCC (NS-FA-RCC), in which all fine aggregate is replaced with fly ash. The principal results from the bending fatigue test of this RCC were as follows.

- (1) The S-N curve of NS-FA-RCC is lower than that of general RCC, when the stress amplitude is smaller.
- (2) Scatter of the strength of the NS-FA-RCC may result in a hardening property of mortar and non-uniformity of adhesion at boundary between mortar and coarse aggregate.
- (3) Weak points of NS-FA-RCC under bending fatigue stress increase, when the repetition load is larger.

REFERENCES

- [1] Concrete Committee, Subcommittee about Effective Use of Resources for Concrete: Concrete Library “The Present Condition and Problems of Effective Use of Resources”, Japan Society of Civil Engineers, 1999.
- [2] Haque, M.N., and Ward, M.A.: Marginal Materials in Roller Compacted Concrete for Pavement Construction, ACI Journal Proceedings Vol.83 No.4, pp.674-679, 1986.
- [3] Fukute, T., Nakano, K. and Ishii, M.: Improvement of Characteristics of Roller Compacted Concrete by Classified Fly Ash, ACI Special Publication Vol.153, pp.43-66, 1995.

- [4] Gao, Pei-Wei, Wu, Sheng-Xing, Lin, Ping-Hua, Wu, Zhong-Ru, and Tang, Ming-Shu: Effect of Fly Ash on Deformation of Roller Compacted Concrete, *ACI Materials Journal*, Vol.103, No.5, pp.336-339, 2006.
- [5] Bordonado, G. and Nissoux, J.L.: Road-Building Concrete Incorporating Fly-Ash or Slag, *ACI Special Publication Vol.79*, pp.471-494, 1983.
- [6] Marchand, J., Boisvert, J., Pigeon, M. and Isabelle, H. L.: Deicer Salt Scaling Resistance of Roller Compacted Concrete Pavements, *ACI Special Publication Vol.126*, pp.131-154, 1991.
- [7] Marchand, J., Pigeon, M., Boisvert, J., Isabelle, H.L., and Houdusse, O.: Deicer Salt Scaling Resistance of Roller Compacted Concrete Pavements Containing Fly Ash and Silica Fume, *ACI Special Publication*, Vol.132, pp.151-178, 1992.
- [8] Delagrave, Anik, Marchand, Jacques, Pigeon, Michel and Boisvert, Jean: Deicer Salt Scaling Resistance of Roller-Compacted Concrete Pavements, *ACI Material Journal*, Vol.94, No.2, pp.164-169, 1997.
- [9] Joshi, R.C. and Natt, G.S.: Roller Compacted High Fly Ash Concrete, *ACI Special Publication*, Vol.79, pp.347-366, 1983.
- [10] Haque, M.N., Langan, B.W., and Ward, M.A.: High Fly Ash Concrete, *ACI Journal Proceedings*, Vol.81, No.1, pp.54-60, 1984.
- [11] Tse, E.W., Lee, D.Y., and Klaiber, F.W.: Fatigue Behavior of Concrete Containing Fly Ash, *ACI Special Publication*, Vol.91, pp.273-290, 1986.
- [12] Vieira, M., de Almeida, I.R., and Goncalves, A.F.: Influence of Moisture Curing on Durability of Fly Ash Concrete for Road Pavements, *ACI Special Publication*, Vol.192, pp.91-102, 2000.
- [13] Okada, Ryota, Matsuo, Eiji, Saitoh, Tadashi, Hamada, Sumio: Property of RCC using fly ash instead of fine aggregate, *Proceedings of the Japan Concrete Institute*, Vol.24-1225, pp.1395-1400, 2002.
- [14] Okada, Ryota, Matsuo, Eiji, Hino, Kazutoshi, Hamada, Sumio: Site construction test of RCC in which all fine aggregate was replaced to coal ash, *The Proceeding of the 57th JSCE Annual Meeting*, the 5th department, pp.479-480, 2002.
- [15] Japan Road Association: Guideline of Design and Construction for Pavement, 2002.
- [16] Ishibashi, Tadayoshi, Kojima, Takayuki, Sakata, Kenji, Matsushita, Hiromichi: Series of durability of concrete structures – Fatigue, pp. 41, Gihodo publishing, 1987.
- [17] Hamada, Sumio, Nakagawa, Kenji, Naruoka, Masao: On the treatment of censored data in fatigue test, *Journal of Materials, Concrete Structures and Pavements*, No.189, pp.99-105, 1971.
- [18] Koyanagawa, Masashi, Kokubu, Katsuro, Fukuda, Tadashi: A basic study on the flexural fatigue of concrete pavement slab, *Journal of Materials, Concrete Structures and Pavements*, No.372, pp.131-137, 1986.
- [19] Inoue, T., and Omoto, S.: Fatigue of Roller Compacted Concrete, Review of the 42nd General Meeting-Technical Session-Held in Tokyo, pp.543-546, 1988.
- [20] Kobayashi, Ryota, Hamada, Sumio, Ueda, Mitsuru: Study on the bending fatigue properties of roller compacted concrete mixed with recycled aggregates, *Journal of Materials, Concrete Structures and Pavements*, No.546, pp.133-143, 1996.

[21] Kobayashi, Ryota, Hamada, Sumio, Kato, Yasuhiro: Study on the bending fatigue properties of concrete pavement with recycled aggregates, Journal of Materials, Concrete Structures and Pavements, No.581, pp.155-160, 1997.

Table 1 Physical Properties of Fly Ash

item		Japanese Industrial Standards				measure value
		class 1	class 2	class 3	class 4	
hygroscopic moisture (%)		= ≤ 1.0				0.1
ignition loss (%)		< 3.0	< 5.0	< 8.0	< 5.0	5.3
SiO ₂ (%)		45 $<$				55.4
density (g/cm ³)		1.95 $<$				2.12
ratio of flow value (%)		105 $<$	95 $<$	85 $<$	75 $<$	103
fineness	residue from the sieve 45×10^{-6} m (%)	< 10	< 40	< 40	< 70	21
	specific surface area (cm ² /g)	5000 $<$	2500 $<$	2500 $<$	1500 $<$	3560
index of activity	at 28 days (%)	90 $<$	80 $<$	80 $<$	60 $<$	75
	at 91 days (%)	100 $<$	90 $<$	90 $<$	70 $<$	91

Table 2 Mix proportion and Strength of Concrete

the mass of unit volume of coarse	W/(C+FA) (%)	W/C (%)	the unit content (kg/m ³)			
			W	C	FA	Coarse Aggregat
100	23	50	149	300	349	1599
Average of static bending strength: $f_{br}=6.47\text{N/mm}^2$ (Number of specimen: 8, Coefficient of variation: 8.9%)						

Table 3 Results of Fatigue Test

S	r	N	P(N)
0.70	1	1,096	0.857
	2	1,097	0.714
	3	2,258	0.571
	4	151,342	0.429
	5	184,661	0.286
	6	296,041	0.143
0.65	1	2,583	0.857
	2	3,809	0.714
	3	43,352	0.571
	4	145,090	0.429
	5	268,857	0.286
	6	293,985	0.143
0.60	1	19,091	0.857
	2	185,517	0.714
	3	199,659	0.571
	4	471,992	0.429
	5	846,016	0.286
	6	1,752,825	0.143
0.50	1	75,015	0.857
	2	164,728	0.714
	3	552,750	0.571
	4	578,420	0.429
	5	1,361,287	0.286
	6	1,367,656	0.143
0.40	1	1,186,366	0.750
	2	2,000,000	0.500

Table 4 Regression Formula of *t* and *N* in Each Stress Amplitude

S	regression formula	m(log <i>N</i>)	σ(log <i>N</i>)
0.70	$t = -0.630 \log N + 2.845$	4.518	1.588
0.65	$t = -0.895 \log N + 4.129$	4.615	1.118
0.60	$t = -1.177 \log N + 6.414$	5.449	0.850
0.50	$t = -1.597 \log N + 9.015$	5.644	0.626

Table 5 S-N Curve, σ (log*N*), σ (S-N) Obtained from Each Author's Test Results

kind of concrete	aggregate	author	S-N curve	fatigue strength at 2 million repetitions	σ(log <i>N</i>)	σ(S-N)
normal concrete	normal aggregate	Koyanagawa	$S = -0.111 \log N + 1.339$ ($S = -0.082 \log N + 1.133$)	0.64 (0.57)	0.70~1.14	0.732
RCC	normal aggregate	Inoue	$S = -0.105 \log N + 1.213$ ($S = -0.090 \log N + 1.078$)	0.55 (0.50)	0.65~0.89	0.574
	recycled aggregate:	Kobayashi	$S = -0.095 \log N + 1.105$	0.51	0.65~1.49	0.786
	fly ash	this study	$S = -0.095 \log N + 1.105$	0.42	0.34~1.70	0.822

*() means the equation which used the modified lower stress level by the modified Goodma

Table 6 Coefficient of Variation of Static Bending Strength

kind of concrete	aggregate	authors	coe. of variation (%)	aimed bending strength (MPa)	the number of specimens
normal concrete	normal aggregate	Koyanagawa	6.2	3.9	24
		Inoue	6.9	5.9	24
RCC	recycled agg. : Type 1	Kobayashi	8.0	5.7	13
	fly ash	this study	17.1	5.7	17

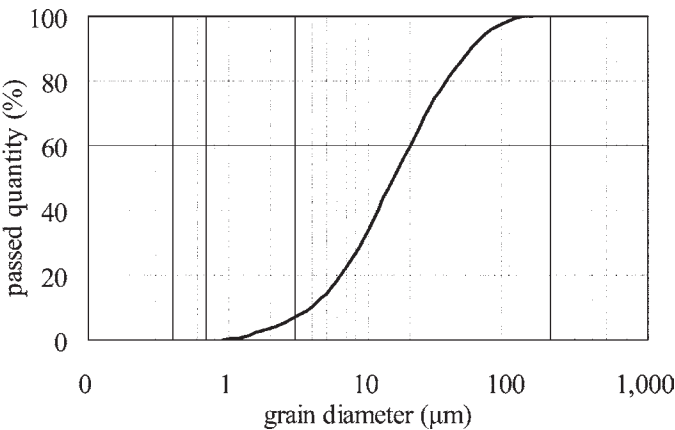


Fig.1—Grading curve of fly ash.

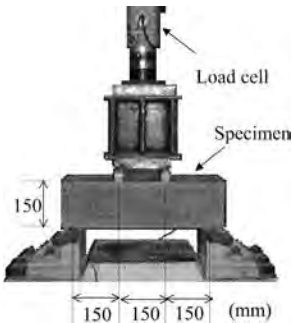


Fig.2—Tests set-up.

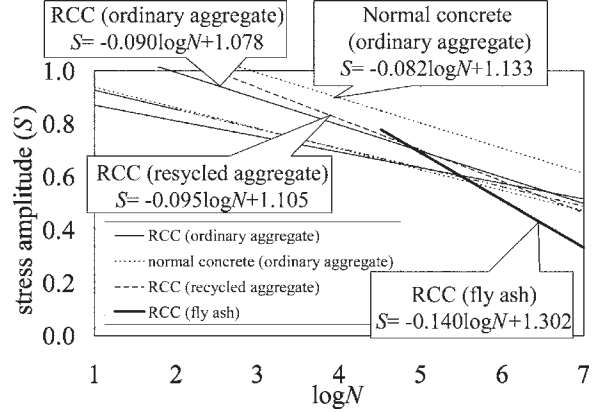


Fig.3—P(N)-N curve on logarithm normal probability paper.

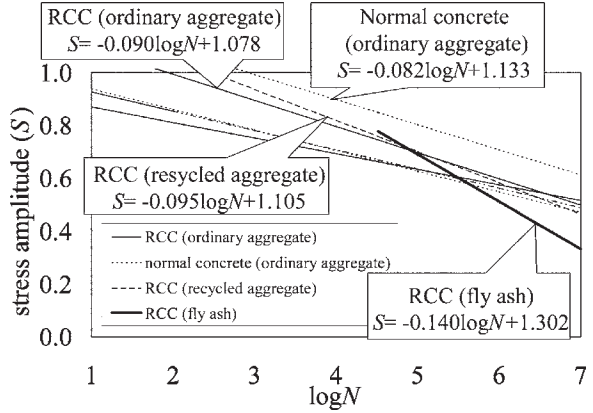


Fig.4—S-N curves of this study and other studies.

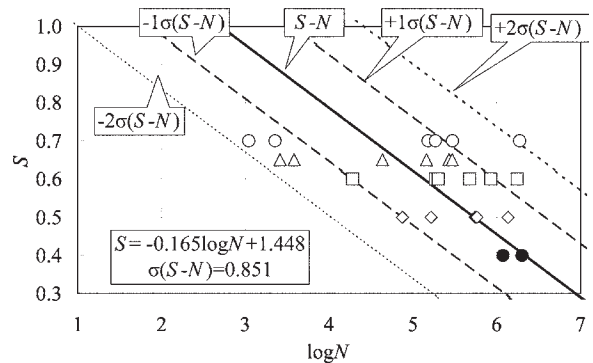


Fig.5—Scatter of fatigue life and S-N curve.

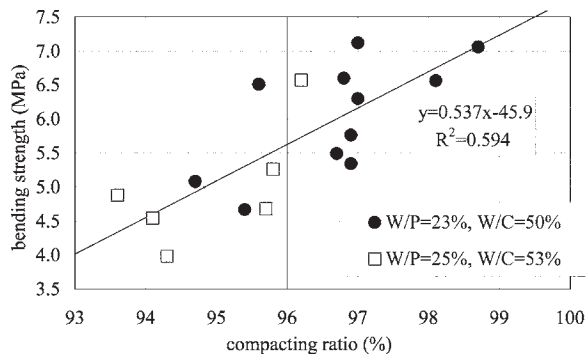


Fig.6—Relationship between compacting ratio and static bending strength.

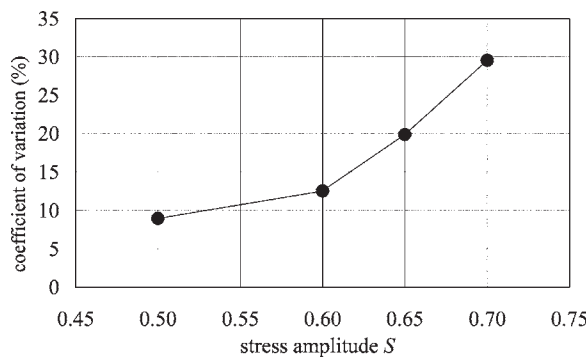


Fig.7—Relationship between each stress amplitude and the coefficient of variation of the repetition at failure.

Temperature Rise and Strength Development in Laboratory-Cast Structural Elements Containing Slag

by S.J. Barnett, M.N. Soutsos, S.G. Millard,
and J.H. Bungey

Synopsis: Sections of high strength concrete (target mean strength of 100 MPa) column elements were cast in the laboratory and cured in a temperature-controlled environmental room at conditions representing average summer or winter temperatures in the UK. In each case, the room was programmed to cycle with a 24-hour period between a minimum and a maximum temperature representing day and night variations. The elements were instrumented with embedded thermocouples and pull-out test inserts, for measurement of in situ strength. Temperatures were continuously recorded for at least seven days after casting. Strength development was assessed by means of both pull-out tests and compressive strength testing of drilled cores at ages up to 28 days. Temperature-matched curing of 100 mm cubes was also carried out using the thermocouple output measured from the column elements. In summer conditions, the temperature rise was observed to be lower in slag concretes than in portland cement concrete but despite this the measured strength development was significantly enhanced compared to standard cured (20 °C) control cubes. The in situ strength improvement was sufficient to allow slag concrete to be used in construction in summer without causing delays to fast-track construction schedules.

Keywords: fast-track construction; slag cement; strength development; temperature rise

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INTRODUCTION

Over the past two decades, the increasing demand for fast track construction has created a need for early-age in situ estimates of concrete strength. Fast track construction can have considerable economic benefits. Accelerated construction schedules that put a new, repaired or overlaid pavement into service require adequate concrete strength to withstand traffic loads. Typical applications include localised repairs, replacement of busy intersections and major slipform paving. In structural components, the early removal of forms and props or the application of post-tensioning, and the termination of curing in cold weather, have safety implications as well as resulting in cost savings¹⁻⁴. Since fast track construction requires high strengths in the first few days after concrete placement, the factors affecting strength at early ages must be considered. These factors include the composition of the concrete mixture, such as cement type and pozzolan addition. The strength development of concrete is also influenced by the temperature within the structural element. Strength gain is more rapid at higher temperatures and slower at lower temperatures. If the temperature is too low, strength gain will cease altogether. The temperature inside the concrete depends on the heat of hydration produced by reaction of the binder, the size and shape of the structural element, the type and removal time of formwork, the ambient temperature and the use of insulation.

Ground granulated blast furnace slag, along with other supplementary cementitious materials, has not gained popularity in fast track construction because its strength gain is slower at standard 20 °C curing temperatures²⁻⁴. Tables of striking times have been developed which apply only to portland cement and rapid hardening portland

cement. However the use of slag as a supplementary cementitious material in concrete provides both material cost savings and environmental benefits, through reduction of cement use and consequently reductions in energy use during production and CO₂ emissions. In addition there are a number of technical advantages provided by the use of slag (e.g. improved workability, increased ultimate strength, improved durability)⁵. There are indications that slag is heavily penalised by standard 20 °C curing regimes^{3, 4}. At the elevated temperatures often found within in situ concrete elements, there can be a significant improvement in the early-age strength of blended cement concrete containing fly ash or slag^{3, 4}. The rate of reaction of these pozzolanic materials has been observed to be enhanced by elevated temperatures. Once the enhanced rate of reaction is initiated it appears to continue even when the external source of heat is removed. This observation has been attributed to an 'activation effect'⁴. The high early-age temperatures occurring within in situ structural concrete elements, due to heat of hydration of the binder, can produce significant improvements in early-age strength development.

The work described here forms part of a three-year project funded by the Engineering and Physical Sciences Research Council (EPSRC), UK⁶⁻⁹. The main aim of the project was to investigate the early-age strength development of concrete made with composite cement under simulated in situ temperature histories in order to give guidance for their use in fast track concrete construction. Part of this work involved the casting of representative sections of structural elements in the laboratory. Columns were selected for this study due to their importance in fast track construction. This paper focuses upon high strength, thick column sections, which represent a situation where low heat loss would be expected.

MATERIALS AND EXPERIMENTAL PROCEDURES

Materials

Two concrete mixtures, both with a target mean strength of 100 MPa (assuming a design strength of 90 MPa) at 28 days, were investigated. One mixture was a slag cement concrete, containing a binder comprising 35% slag and 65% portland cement (pc) by mass. The other mixture was a portland cement concrete. Mixture proportions are given in Table 1.

Single batches of portland cement and slag were used throughout. The fine aggregate used in both mixtures was natural sand which had 42% passing a 600 µm sieve and a fineness modulus of 2.92. The moisture content of the sand was determined by the Speedy Test and allowance made for excess water in determining batch weights for mixing. The coarse aggregate used in both mixtures was a 50%:50% mixture of 10 mm and 20 mm single-sized crushed granite. A polycarboxylate polymer based superplasticising additive was used to maintain workability at the low w/b used.

Structural Elements

The structural laboratory test specimen studied in this investigation, Fig. 1, was designed to represent a short section of a thick tall column. The column had dimensions 750 x 750 x 300 mm high. It was instrumented with 10 embedded thermocouples positioned at various points along the lines AB and BC at the mid-height of the section,

as indicated in Fig. 1. Five pull-out test inserts were fixed to each of the four vertical faces on the inside of the plywood formwork prior to casting. Pull-out test inserts were separated by a distance of 200 mm. Core drilling positions were separated by a distance of 160 mm. The column specimen was insulated with 150 mm expanded polystyrene at the top and bottom to prevent vertical heat transfer and thus represent a short section of a longer column. Preliminary measurements showed that the vertical temperature variation within the height of the section was minimal, so it may be assumed that all heat loss occurred through the vertical faces of the column section. The vertical column formwork was removed after 1 day. However thermal insulation on the upper and lower faces remained in place throughout the curing regime, except the short duration when core cutting was taking place, during which time the upper insulation was removed.

Temperature Conditions

Publicly available information from the UK Meteorological Office¹⁰ was used to develop the temperature profiles for programming of the 2 m x 2 m x 2 m environmental room. The room was programmed to cycle between the appropriate minimum and maximum temperatures in a 24-hour period, simulating day and night temperature variations. The minimum and maximum temperatures used were the average minimum and maximum temperatures for the months of January (winter conditions) and July (summer conditions) over the period 1971-2000, as recorded at Manchester International Airport, UK. The equation used to describe the temperature variation is a sine function of the form:

$$T = T_0 + a \sin\left(\frac{2\pi t}{b}\right)$$

Where T = temperature, °C

t = time, hours

$$T_0 = (T_{\min} + T_{\max})/2, ^\circ\text{C}$$

$$a = (T_{\max} - T_{\min})/2, ^\circ\text{C}$$

$$b = 24 \text{ hours}$$

Details of the temperatures used and the corresponding values of T_0 and a are given in Table 2. The peak in the ambient temperature corresponded to approximately 4 hours after casting.

Mixing, Casting and Testing Specimens

The materials were weighed and placed in a 0.1m³ capacity horizontal pan mixer. For summer conditions, the materials were used at room temperature. When winter curing temperatures were required, the materials were cooled overnight in the environmental room in order to lower the casting temperature to that expected on site. Casting temperatures, measured after mixing was complete, were 23 ± 2 °C for elements cured in summer conditions and 15 ± 2 °C for elements cured in winter conditions. Materials were added in the order: coarse aggregate, cement, slag, sand and superplasticiser combined with the mix water. The concrete was mixed for 5 minutes and then transferred to the formwork by shovel and compacted using a vibrating poker. Each element was cast from 3 equal batches of concrete. After casting, the element was

transferred to the environmental room. In addition, a number of 100 mm companion cubes were cast from each batch and compacted on a vibrating table. These cubes were covered with damp Hessian and a polyethylene sheet. After 24 hours, they were demoulded and placed in a curing tank at 20 °C. The cubes were subsequently tested for compressive strength at the same ages as the relevant column was tested, to provide a comparison with strength development in the structural element and to check batch variability.

The temperatures within the column specimens were recorded for at least seven days. Beyond this age, the temperatures within the columns varied in a predictable way as the temperature was affected only by the ambient conditions. In general, strength testing was carried out at 1, 2, 3, 7 and 28 days. At each testing age, four pull-out tests were first performed and then 100 mm diameter cores were drilled. Cores were drilled vertically through the full height of the element, as indicated in Fig. 1. Two cores were drilled in the columns at each testing age. The cores were extracted and labelled and then cut to length. 50 mm was removed from the top of each core and 10 mm from the bottom. The remaining core was then cut in half to provide two specimens for testing. A total of four test specimens for each testing age were produced in this way, each having a length/diameter ratio of approximately 1.1-1.2. The ends of the specimens were smoothed by precision grinding prior to testing in compression. Companion cubes cured under standard conditions were tested at the same time.

Temperature Matched Curing

At a later date, a further batch of the concrete was prepared as above. 100 mm cubes were prepared for standard curing and for temperature-matched curing. The standard cured cubes were treated as above. For temperature matched curing, the cubes were transferred to an environmental chamber immediately after casting. The chamber maintained a relative humidity of 90% and the temperature of the chamber was computer-controlled to match the temperatures recorded at the centre of the relevant column section. After 7 days, the temperature matched cured cubes remaining in the chamber were removed and cured at a constant temperature of 16 °C (summer conditions) or 5 °C (winter conditions) until testing. Beyond this age, there was little variation with time in the temperature history used for temperature-matched curing and the temperature at the centre of the column section was constant to within ± 2 °C. Three standard cured cubes and three temperature-matched cured cubes were tested at ages of 1, 2, 3, 5, 7, 14 and 28 days.

RESULTS AND DISCUSSION

Temperature Rise of Structural Elements

Fig. 2 shows the temperature histories recorded at the centre of each column specimen for the first five days after casting. The peak temperatures and the respective times at which they occurred at various points within the columns are detailed in Table 3. A significant rise in temperature above the casting temperature occurred in all the column specimens. The peak temperature was higher when using portland cement concrete than with the concrete containing slag. The portland cement concrete reached a temperature of 56 °C in summer conditions, compared with 49 °C for the 35% slag concrete. The

columns cured in winter conditions reached peak temperatures of 39 °C for portland cement concrete and 25 °C for 35% slag concrete. Ambient conditions were observed to have a greater effect on the temperature rise of the slag concrete than on portland cement concrete. For 35% slag concrete, the rise above the casting temperature was 15 °C lower in winter conditions than in summer conditions. This compared with a difference of 11 °C for portland cement concrete.

The temperature of the concrete was also observed to decrease very quickly when the formwork was removed. In summer conditions, the temperature at the edge of a column dropped by 5 °C in the first hour after formwork removal. Leaving the formwork in place for a longer period of time would provide an advantage both in terms of improving strength development and preventing large temperature gradients between the centre and the edge, which can lead to thermal cracking.

Strength Development of Structural Elements

Fig. 3 shows the strength development of the concrete in the column elements cast in summer and winter conditions. Each graph consists of four plots:

- Compressive strength of core specimens: Each point is the average of four test results. Due to the geometry of the specimens the in situ cube strength calculated according to BS1881¹¹ was within ± 2 MPa of the core strength and hence no corrections were applied to the results.
- Compressive strength estimated from the pull-out test results: The manufacturer's recommended correlation between pull-out test result and compressive strength was applied to obtain the compressive strength estimate. This has been shown to provide an adequate estimate of compressive strength. Each point is the average of four tests.
- Temperature matched cured (TMC) cube strength: Each point is the average of three test results.
- Standard cured cube strength: Each point is the average of three tests. The results are for standard cured cubes prepared alongside the temperature-matched cured cubes. The control cubes prepared alongside the column elements had similar strengths, except for the 35% slag concrete column cured in winter ambient conditions where there was significant batch variability. In this case, the control cubes prepared alongside the column had higher strengths than the standard cured cube results shown in the graph (96 MPa Compared to 87 MPa).

The plotted error bars are based on the minimum and maximum values obtained at each testing age.

Core strengths were generally higher than pull-out test estimates of the compressive strength. Statistical tests are needed to determine the significance of this observation as the variability of the pull-out tests is high. However, strengths at the centre of the column would be expected to be greater than strengths at the edge, due to the different temperature histories and edge effects during compaction. Core strengths were also observed to be lower than the TMC cube strengths in general. This is due to effects such as differences in compaction, segregation and bleeding, which occur to a greater extent in the column element than in 100 mm cubes. This is supported by the observation that core specimens from the upper part of the column tended to have lower strength than those from the lower part. The core strength of the 35% slag concrete cured

in winter conditions (Fig. 3d) was higher than the strength of the TMC cube because of the batch variability mentioned above.

For portland cement concrete, the core specimens had lower strength than the standard cured cube specimens in both winter and summer conditions at all ages except for 1 day. The reaction of portland cement is less sensitive to temperature than that of slag, so any improvement in strength development as a result of the higher in situ temperatures is masked by bleeding/segregation effects. These effects are removed, when comparing TMC and standard cured cube strengths, and in this case an improvement in strength development was observed for the first few days after casting in both winter and summer conditions.

For the 35% slag concrete cured in summer conditions, there was a significant improvement in early-age strength development at ages up to 7 days. This was observed for both the core specimens and, to a greater extent, the TMC cubes. Beyond this age, the core and TMC strengths are approximately equal to the standard cured cube strength. For the same concrete cured in winter conditions, the strength development was similar to the strength obtained from standard 20 °C curing for the first few days, and beyond this age the strength was lower than the standard cured cube strength.

Fig. 4 shows the ratio of TMC cube strength to standard cured cube strength for each concrete. This figure illustrates the different sensitivity to temperature of the portland cement and the slag reaction. In the case of portland cement the ratio of the TMC cube strength to the standard cured cube strength was similar during both summer and winter temperature conditions. For the 35% slag concrete, there was a significant difference in the strength development during the different temperature regimes. While winter conditions caused the TMC strengths to be lower than when using standard 20 °C curing, summer conditions produced a great improvement in the strength development. The one day strength of the 35% slag concrete obtained from TMC cubes was 1.75 times higher than the strength obtained from standard 20 °C cured cubes. The equivalent value for portland cement was only 1.3.

Table 4 shows the time taken for each column specimen to reach 75% of the 90 MPa design strength. The time taken for the slag concrete to reach this level of strength development in the column was longer than for portland cement concrete. In summer conditions, the portland cement concrete reached this strength within 2 days in both the column and the control cubes. The 35% slag concrete took 1.9 days in the column and 4.4 days in the control specimens. Although, the portland cement concrete reaches this level of strength in a shorter time than the 35% slag concrete, the improvement in the strength development of the slag concrete compared to the control cubes is very significant and it reaches 75% of its design strength in less than half the time indicated by the standard cured control cubes. The improvement for portland cement concrete is only around 20%. This improvement means that, if the effect on strength development on in situ temperature rise is considered, 35% slag concrete can be used in summer conditions without resulting in delays in the construction schedule. In the results presented here, 35% slag concrete takes just 7 hours longer than portland cement concrete to achieve 75% of the design strength. In winter conditions for both concretes, the time taken to reach the required strength is much longer than indicated by the control cube strength, although 35% slag concrete is affected more than portland cement concrete.

In order to utilise the effects of in situ temperature on strength development when planning construction schedules, it is necessary to have a reliable model to predict strength development from a measured temperature history (or indeed a more sophisticated model capable of also predicting the temperature history). One of the simplest existing models was developed by Freiesleben Hansen and Pedersen¹² based on the Arrhenius rate equation. This model predicts the strength development of concrete for any temperature history from its strength development at a reference temperature (in this case 20 °C). This model defines an equivalent age, t_e , which is the age when concrete at the reference temperature will achieve the same strength as at time t :

$$t_e = \sum_t \left(\exp \left[\frac{E_a}{R} \left(\frac{1}{273 + T_{ref}} - \frac{1}{273 + T} \right) \right] \right) \Delta t$$

Where T_{ref} is the reference temperature = 20 °C
 T is the average temperature over time Δt
 E_a is the apparent activation energy, J/mol
 R is the gas constant = 8.314 J/mol.K

The apparent activation energy is a measure of the temperature sensitivity of concrete. The values for the concrete studied in this paper are 32.9 kJ/mol.K for the portland cement concrete and 46.8 kJ/mol.K for the 35% slag concrete⁶. The equivalent age was calculated from the actual age and temperature history using the above equation. The predicted strength development was then calculated on the basis of the strength development at the reference temperature. A modified hyperbolic equation was used to describe the strength development at 20 °C, with the equation parameters determined by regression analysis. Fig. 5 shows the strength estimated by this process at the centre and edge of the columns elements (positions B and A respectively in Fig.1). These estimates are compared in the plot with the core compressive strength and the TMC cube strength. The core strength cannot be directly compared with the estimates due to bleeding/segregation effects, which are not taken into account in the model. However, the estimated strength development based on the temperature history at the centre of each column can be directly compared with the measured TMC cube strength. In this case, there are no differences in compaction, bleeding and segregation and the TMC cubes were also prepared from the same batch of concrete as the reference temperature cubes. The model predicts the strength development of the TMC cubes very well for the portland cement and 35% slag concrete cured in summer and winter conditions, with only slight discrepancies between the measured and predicted values. However, it should be noted that at a higher slag level or with lower strength grade concrete, the accuracy of this model is reduced as it is not able to account for the decrease in later age strength caused by higher curing temperatures^{13, 14}.

CONCLUSIONS

This work has highlighted the beneficial effects of temperature on the strength development of slag concrete. The in situ temperature rise occurring inside thick structural elements has been shown to produce a significant improvement in early-age

strength development compared to standard curing. The slag concrete in this study had 1-day strength 1.75 times higher than standard cured cube strength in temperature matched curing tests. For summer curing conditions, in the high strength concretes studied here, the magnitude of this effect was sufficient for 35% slag concrete to have early-age strengths similar to those of portland cement concrete. In winter ambient conditions, the strength development of both concretes is slowed down, and the effect is greater for slag concrete than for portland cement concrete.

FURTHER RESEARCH

The work presented here formed part of a larger study, investigating the effects of element size, ambient conditions, concrete strength grade and slag ratio on the early-age strength of in situ concrete. Analysis of the data obtained is ongoing and will form the basis of future publications. Further research is needed to extend the findings to other supplementary cementitious materials.

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REFERENCES

1. Bickley, J. A., *Field manual for maturity and pullout testing on highway structures*, National Academy of Sciences, Washington DC.
2. Carino, N. J., and Tank, R. C., "Maturity functions for concretes made with various cements and admixtures," *ACI Materials Journal*, 1992, V. 89, No. 2, pp. 188-196.
3. Escalante-Garcia, J. I., and Sharp, J. H., "The microstructural and mechanical properties of blended cements hydrated at various temperatures," *Cement and Concrete Research*, 2001, V. 31, No. 5, pp. 695-702.
4. Roy, D. M., and Idorn, G. M., "Hydration, structure and properties of blast furnace slag cements, mortars and concrete," *ACI Journal*, 1982, V. 79, No. 6, pp. 444-457.
5. Domone, P. L. J., and Soutsos, M. N., "Properties of high strength concrete mixes containing pulverised fuel ash and ground granulated blast furnace slag," *Magazine of Concrete Research*, 1995, V. 47, No. 173, pp. 355-367.
6. Barnett, S. J., Soutsos, M. N., Bungey, J. H., and Millard, S. G., "Strength development of mortars containing ground granulated blast-furnace slag: Effect of curing

temperature and determination of apparent activation energies," *Cement and Concrete Research*, 2005, V. 36, No. 3, pp. 434-440.

7. Barnett, S. J., Soutsos, M. N., Bungey, J. H., and Millard, S. G., "Fast track concrete construction using cement replacement materials," *Proceedings, Eighth CANMET ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete* (Las Vegas, USA, 2004), American Concrete Institute, 2004, pp. 135-151.

8. Barnett, S. J., Soutsos, M. N., Bungey, J. H., and Millard, S. G., "The effect of the level of cement replacement with ground granulated blast furnace slag on the strength development and adiabatic temperature rise of concrete mixes," *Proceedings, Global Construction: Ultimate Concrete Opportunities* (Dundee, UK, 2005), 2005, V. 1, pp. 165-172.

9. Soutsos, M. N., Barnett, S. J., Bungey, J. H., and Millard, S. G. "Fast track construction with high strength concrete mixes containing ground granulated balst furnace slag," *Proceedings, ACI Seventh International Symposium on High Strength/High Performance Concrete* (Washington DC, USA, 2005), 2005, V. 1, pp. 255-270.

10. "1971-2000 averages, manchester airport," UK Meteorological Office, http://www.metoffice.gov.uk/climate/uk/averages/19712000/sites/manchester_airport.html.

11. "Bs 1881-120: 1983, testing concrete- part 120: Method for determination of the compressive strength of concrete cores," British Standards Institution.

12. Freiesleben Hansen, P. and Pedersen, E. J., "Maturity computer for controlled curing and hardening of concrete," *Journal of the Nordic Concrete Federation*, 1977, No. 1, pp. 21-25.

13. Chanvillard, G., and D'Aloia, L., "Concrete strength estimation at early ages: Modification of the method of equivalent age," *ACI Materials Journal*, 1997, V. 94, No. 6, pp. 520-530.

14. Kjellsen, K. O., and Detwiler, R. J., "Later-age strength prediction by a modified maturity model," *ACI Materials Journal*, 1993, V. 90, No. 3, pp. 220-227.

Table 1 — Concrete Mixture Proportions

% slag	kg/m ³						water/ binder
	portland cement	slag	granite	sand	superplasticiser	free water	
0	487	-	1142	762	11.9	127	0.26
35	322	174	1142	762	5.8	124	0.25

Table 2 — Minimum and Maximum Temperatures Used For Ambient Temperature Profiles (UK Met Office Data for 1971-2000, Manchester International Airport)

Month	Met Office Averages, 1971-2000		Values Used		T_0 , °C	a , °C
	Minimum, °C	Maximum, °C	Minimum, °C	Maximum, °C		
January	1.5	6.9	2	7	4.5	2.5
July	12.3	20.3	12	20	16	4

Table 3 — Peak Temperatures Observed in Column Sections

		Summer conditions		Winter conditions	
		0% slag	35% slag	0% slag	35% slag
Casting Temp., °C		23	23	17	14
Centre (B)	Temp., °C	56	49	39	25
	Time, hours	18	22	22	23
Edge (A)	Temp., °C	43	38	28	18
	Time, hours	16	22	22	22
Corner of column (C)	Temp., °C	36	31	21	14
	Time, hours	16	22	22	21

Table 4 — Time to Reach 75% Design Strength (Calculated by Linear Interpolation)

	Summer Conditions		Winter Conditions	
	0% slag	35% slag	0% slag	35% slag
Element, days	1.6	1.9	5.5	9.6
Control, days	~2	4.4	3.0	3.0
Element/control	~0.8	0.43	1.8	3.2

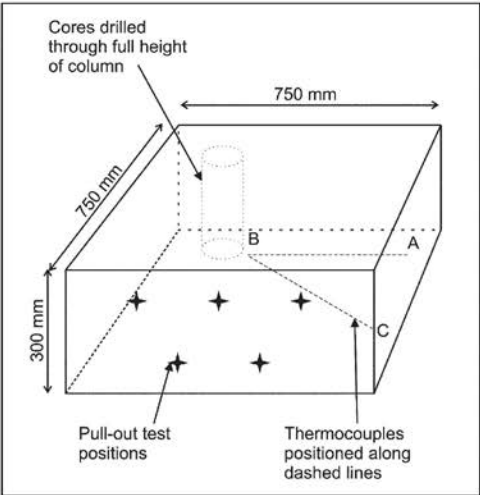


Fig. 1 — Schematic diagram of column specimen.

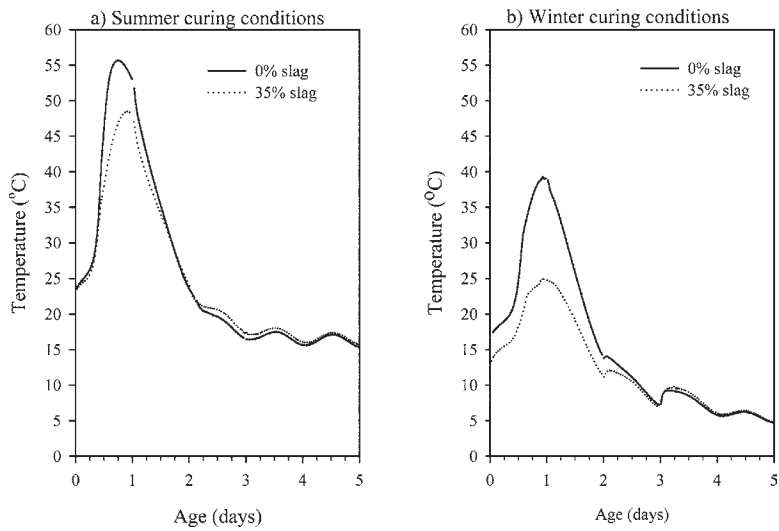


Fig. 2 — Temperatures recorded at the center of column elements.

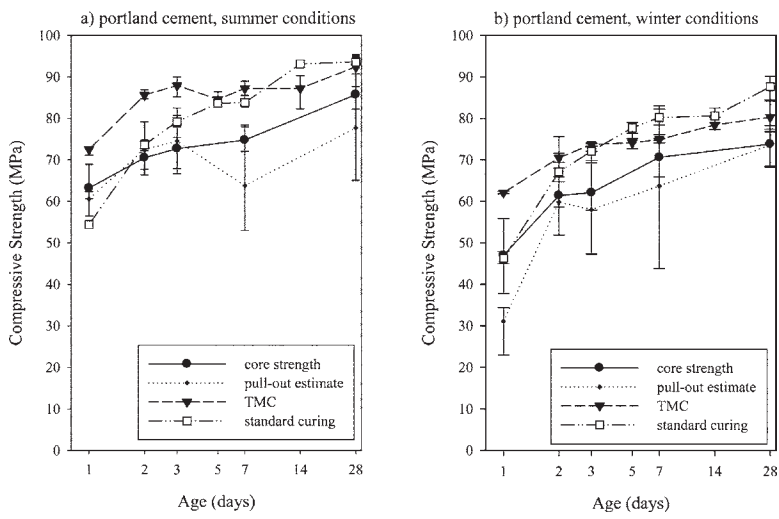


Fig. 3 — Compressive strength development of column elements.

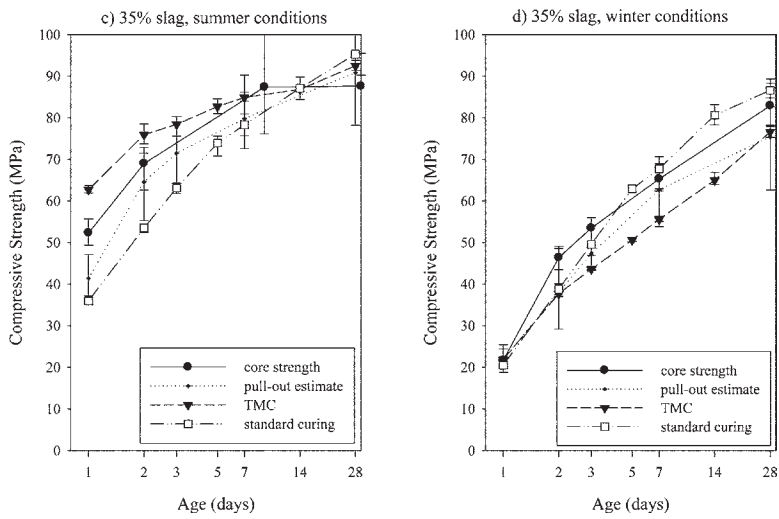


Fig. 3 (con't) — Compressive strength development of column elements.

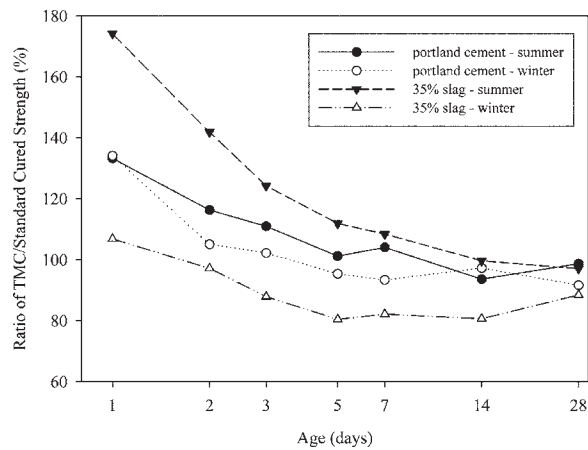


Fig. 4 — Comparison of strength development for temperature matched curing and standard curing.

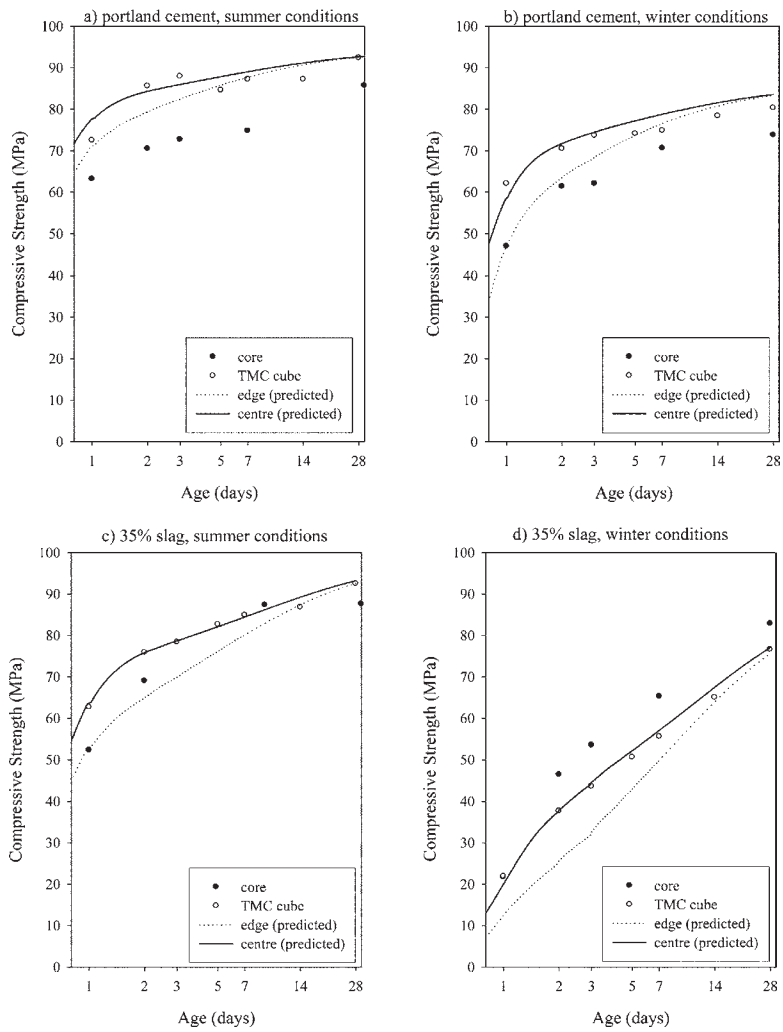


Fig. 5 — Estimated strength at center and edge of portland cement column in summer conditions based on equivalent age.