USE OF PREPLACED AGGREGATE CONCRETE FOR MASS CONCRETE APPLICATIONS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

JUNE 2004

Approval of the Graduate School of Natural and Applied Sciences

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ABSTRACT

USE OF PREPLACED AGGREGATE CONCRETE FOR MASS CONCRETE APPLICATIONS

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June 2004, 170 pages

Heat of hydration is a source of problem in mass concrete since it causes the difference between the inner and the outer temperatures increase excessively, which leads thermal cracks. The first step in fighting against this problem is to keep the initial temperature of concrete as low as possible. From this point of view, Preplaced Aggregate Concrete (in short PAC) is quite advantageous, because the friction taking place among the coarse aggregates during the mixing operation causes the initial temperature of concrete increase. However, since coarse aggregates are not subjected to the mixing operation in PAC method, comparatively lower initial temperatures can be achieved. On the other hand, making PAC by the conventional injection method is quite troublesome, since it requires special equipment and experienced workmanship. Because of this, it would be very useful to investigate alternative methods for making PAC.

In this research, a new method for making PAC has been investigated. The new method is briefly based on increasing the fluidity of the grout by new generation superplasticizers to such an extent that, it fills all the voids in the preplaced coarse aggregate mass when it is poured over, without requiring any injection. In the scope of the study, twelve concrete cube specimens, each with 1 m^3 volume, have been prepared; one of which as conventional concrete, seven of which as PAC by injection method, and four of which as PAC by the new method mentioned above. In order to examine the specimens that have been prepared by three different methods from thermal properties point of view, the difference between the central and the surface temperatures of the specimens have been followed by the thermocouples located in the specimens during preparation. Also, in order to examine the mechanical properties of the specimens, three core specimens have been taken from each specimen at certain ages, compressive strength and modulus of elasticity tests have been carried out on these core specimens.

As a result of the experiments it has been observed that, the PAC specimens prepared by injection method performed better from thermal properties point of view, but worse from mechanical properties point of view than conventional concrete. On the other hand, the PAC specimens prepared by the new method have performed both as well as the other PAC specimens from thermal properties point of view, and as well as conventional concrete from mechanical properties point of view.

Key words: Preplaced Aggregate Concrete, Mass Concrete, Heat of Hydration, Concrete Temperature, Thermal Cracks.

ÖZ

AGREGASI ÖNCEDEN YERLEŞTİRİLMİŞ BETONUN KÜTLE BETONU UYGULAMALARINDA KULLANILMASI

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Haziran 2004, 170 sayfa

Hidratasyon ısısı kütle betonunda iç ve dış sıcaklık farklarının aşırı artmasına sebep olduğu ve bu da termal çatlaklara yol açtığı için bir sorun kaynağıdır. Bu sorunla mücadeledeki ilk adım, betonun ilk sıcaklığını mümkün olduğunca düşük tutmaktır. Bu açıdan, Agregası Önceden Yerleştirilmiş Beton (kısaca AÖYB) oldukça avantajlıdır, çünkü karışım işlemi sırasında kaba agregalar arasında meydana gelen sürtünme betonun ilk sıcaklığının yükselmesine sebep olmaktadır. Oysa, AÖYB yönteminde kaba agregalar karışım işlemine tabi tutulmadığından, nispeten daha düşük ilk sıcaklıklar elde edilebilmektedir. Ancak diğer yandan, geleneksel enjeksiyon yöntemiyle AÖYB yapımı, özel ekipman ve tecrübeli işçilik gerektirdiğinden biraz zahmetlidir. Bu nedenle, alternatif AÖYB yapım yöntemlerinin araştırılmasında büyük fayda vardır.

Bu araştırmada, yeni bir AÖYB yapım yöntemi incelenmiştir. Kısaca bu yeni yöntem, harç akışkanlığının yeni nesil süperakışkanlaştırıcılarla, harcın enjeksiyona gerek kalmadan, önceden yerleştirilmiş agregalar arasındaki boşlukları doldurmasına yetecek seviyeye yükseltilmesi prensibine dayanmaktadır. Çalışma kapsamında, herbiri 1 m^3 hacminde, bir tanesi geleneksel beton, yedi tanesi enjeksiyon yöntemiyle hazırlanmış AÖYB ve dört tanesi yukarıda bahsedilen yeni yöntemle hazırlanmış AÖYB olmak üzere toplam oniki adet küp şeklinde beton numuneler hazırlanmıştır. Bu üç farklı yöntemle hazırlanmış numuneleri termal özelikleri açısından inceleyebilmek amacıyla, döküm sırasında numunelere yerleştirilen ısı algılayıcı kablolar vasıtasıyla numunelerin merkez ve yüzey sıcaklıkları arasındaki farklar izlenmiştir. Ayrıca, numunelerin mekanik özeliklerini inceleyebilmek amacıyla da, çeşitli yaşlarda her numuneden üçer adet karot numune alınarak, bu karot numuneler üzerinde basınç dayanımı ve elastiklik modülü testleri yapılmıştır.

Deney sonuçlarına göre, enjeksiyon yöntemiyle hazırlanmış AÖYB numunelerin geleneksel betona kıyasla, termal özelikler açısından daha iyi, fakat mekanik özelikler açısından daha kötü performans gösterdikleri gözlemlenmiştir. Öte yandan, araştırma konusu olan yeni yöntemle hazırlanan AÖYB numuneler, termal özelikler açısından enjeksiyon yöntemiyle hazırlanmış AÖYB kadar, mekanik özelikler açısından da geleneksel beton kadar iyi performans göstermiştir.

Anahtar kelimeler: Agregası Önceden Yerleştirilmiş Beton, Kütle Betonu, Hidratasyon Isısı, Beton Sıcaklığı, Termal Çatlaklar.

If I could provide a small drop of knowledge
to the big ocean of science
by this thesis,
what a pride for me!

ACKNOWLEDGEMENTS

First of all, I am very glad to have finished the first thesis which is supervised by Dr. Lutfullah Turanlı with the title of 'Assoc.Prof.Dr.'. I would like to express my sincere gratitude to my supervisor for his great tolerance towards me throughout the long period passed for the preparation of this thesis, and I would like to congratulate him once more here for his new title. Also I would like to acknowledge Professor P. Kumar Mehta, Civil Engineering Department, University of California at Berkeley, for his valuable suggestions and helpful comments during this research.

I am also thankful to Mr. Osman Özkan, MSCE, president of TEKAR Teknik Araştırma Ltd.Şti. and Mrs. Selda Gedik, MSCE, project manager of TEKAR Teknik Araştırma Ltd.Şti. for supplying of the equipment, and to ERG İnş. Tic. & San. A.Ş. for supplying of the materials used in this research.

Thanks are extended to the laboratory staff who helped me physically during the experimental study.

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LIST OF ABBREVIATIONS

ACI American Concrete Institute

AÖY Agregası Önceden Yerleştirilmiş Beton

ASTM American Society for Testing and Materials

CEB European Concrete Committee

GGBFS Ground Granulated Blast Furnace Slag

PAC Preplaced Aggregate Concrete

PC Portland Cement

PKÇ Portland Composite Cement

SCC Self Compacting Concrete

TS Turkish Standards

CHAPTER 1

INTRODUCTION

1.1. General

Concrete gains strength as a result of the chemical reactions taking place between the cement and water in it, which is called hydration. Hydration of cement is an exothermic reaction, which means that heat is generated as a result of this reaction. For concretes with normal sections, this generated heat can easily be released to the atmosphere, and it does not cause any significant temperature rise in the concrete. However in mass concrete, while the outer portions can easily release the generated heat to the atmosphere, the heat generated at the inner portions can not be released so easily; and this leads a significant temperature difference between the inner and the outer portions of the concrete, which may cause thermal cracks. Because of this, control of the heat of hydration carries vital importance in mass concrete.

The precautions to be taken fighting against this cliched problem can be grouped under two main stages; the precautions to be taken before the concrete is placed, and the ones to be taken after the concrete is placed. The temperature of the concrete at a specified time is a result of its initial temperature when it was placed, and the amount of temperature rise that occurred afterwards until that time. Thus the precautions to be taken before the concrete is placed aim to reduce the initial temperature of the concrete, and the ones to be taken after the concrete is placed aim the control of temperature rise.

The initial temperature of concrete is a result of the initial temperatures of the concrete making materials to be used, and the temperature rise that occurs during the mixing of these materials due to the friction. Thus, in order to reduce the initial temperature of concrete, first of all the concrete making materials should be cooled before they are mixed. For this purpose, cold water, or even sometimes ice is used as mixing water, and the aggregates are washed with cold water. This procedure is called 'precooling'. Also, it is important to store the cementitious material to be used at low temperatures for the same purpose. Coming to the temperature rise that occurs during the mixing of these materials, if conventional concrete is used, there is nothing to do about this. However, use of PAC is a good solution to this problem; because, the temperature rise that occurs during mixing is mainly caused by the friction taking place among the coarse aggregates, and since the coarse aggregates are not subjected to the mixing operation in PAC method, this problem is eliminated.

On the other hand, the temperature rise that occurs after the concrete is placed is a result of the amount and rate of the heat of hydration evolution. Thus, the first precaution to be taken to control this temperature rise is the selection of cementitious material with low heat of hydration potential. For this purpose, use of pozzolans at high volumes is an effective way, but usually it is not enough alone to prevent thermal cracks. Because of this, as an additional precaution, a pipe network is installed in the formwork before the concrete is placed, later, by circulating cold water in this pipe network, the heat generated in the concrete as a result of hydration is absorbed and the excessive rise of concrete temperature is prevented. This procedure is called 'postcooling'. Postcooling is quite an effective way in fighting against thermal cracks, but it is quite troublesome and costly as well. Application of PAC method in combination with the use of high volume pozzolan can eliminate the requirement for postcooling operation. And this provides both a convenience in the construction and a reduction in the cost of mass concrete. Because of this, the subject of PAC should be studied more comprehensively, and it should be tried to make the application of the PAC method more widespread.

1.2. Object and Scope

The object of this study is to investigate a new method for making PAC, and to compare the thermal and mechanical properties of the PAC made by this new method with those of the PAC made by the conventional method and the conventional concrete.

For this purpose, in the scope of the study, twelve cubes of concrete specimens, each having a volume of 1 m^3, with different cementitious material compositions have been prepared. Eleven of these specimens have been prepared as PAC, and one of them as conventional concrete. Seven of the PAC specimens have been prepared by the conventional injection method, and four of them have been prepared by the new method, the details of which are explained in Chapter 2.

In order to investigate the thermal properties of the specimens, during the preparation of the specimens, four thermocouples have been located in each specimen, two of them at the surface and two of them at the centre of the specimen. By using these thermocouples, the difference between the central and the surface temperatures of the specimens have been followed until the temperatures reached a steady-state.

In order to investigate the mechanical properties of the specimens, three core specimens have been taken from each specimen at the ages of 28 days, 90 days, 6 months and 1 year. Compressive strength and the modulus of elasticity tests have been carried out on these core specimens. Also, the non-destructive tests have been carried out on the actual specimens in order to compare the results with the compressive strength values obtained from the core specimens.

CHAPTER 2

LITERATURE SURVEY

2.1. Definition

Preplaced aggregate concrete (PAC) is defined as concrete produced by placing coarse aggregate in a form and later injecting a portland cement-sand grout, usually with admixtures, to fill the voids [1].

Grouted-aggregate, injected-aggregate, Prepakt, Colcrete, Naturbeton, and Arbeton are some other terms used internationally to describe the same method.

2.2. Fields of Application

PAC is particularly useful for underwater construction, placement in areas with closely spaced reinforcement and in cavities where overhead contact is necessary, repairs to concrete and masonry where the replacement is to participate in stress distribution, heavyweight (high-density) concrete, high-lift monolithic sections, and in general, where concrete of low volume change is required.

Need for vibration can not be avoided even when high-range water reducing admixtures are used with conventional concrete; thus, when the reinforcement is too closely spaced to permit the use of vibrators, the PAC procedure is very helpful [2].

When heavyweight coarse aggregate (such as barite) is used to produce high-density concrete, there is a risk of segragation. This can be avoided by preplacing the heavyweight coarse aggregate. Such an application is illustrated at Figure 2.1 [3,4].

Figure 2.1. *Preplacing of heavyweight coarse aggregate (barite) by hand during the construction of a biological shield at Materials Testing Reactor (Arco, Idaho)*

Leaving exposed aggregate surfaces for architectural purposes is another field of application of PAC method. The percentage of coarse aggregate in PAC is roughly 70% greater than the percentage of coarse aggregate in conventional concrete, and if the surface grout is green cut or sandblasted after removal of forms, approximately 25% more aggregates can be exposed, which provides an attractive architectural finishing [2].

2.3. History of Development

PAC was first discovered in 1937 by Louis S. Wertz and Lee Turzillo during a rehabilitation work in Santa Fe railroad tunnel near Martinez, California. They were grouting the voids in the concrete at crown areas. As the work progressed, in order to reduce the consumption of grout, they thought of filling larger spaces with coarse aggregate before grouting. Then, they started to use formwork before placing the coarse aggregate. By this way, PAC was born. The resulting concrete showed such a good performance that it managed to attract the attention of academic researchers. Professor Raymond E. Davis was the first one who concentrated on it. He tried to develop grout mixtures and basic procedures to make this new method more widely used. As a result of his studies, Professor Davis determined most of the unique properties of PAC.

At the beginning, many patents were applied on the method (with trade-name 'Prepakt'), and on the admixtures used as grout fluidifier. However by 1940, all patents have been granted, with some exception on admixtures.

After the method was applied successfully in many big projects through 1940s and 1950s (as it is discussed in details at Section 2.4), in 1970s the Honshu-Shikoku Bridge Authority made a great contribution to the development of the method by carrying out an extensive research program on the method during the construction of a large bridge complex.

Lately, the method also found wide use in placing biological shields around nuclear reactors and x-ray equipment, as high-density concrete application. B.A. Lamberton and H.L. Davis are the two names who are mostly responsible for the development of high-density (heavyweight) PAC [2].

2.4. Important Examples of Application

At the beginning, because of the lack of any performance history, the use of PAC was limited to repair works only (especially, repair of bridges and tunnel linings). However after an extensive laboratory testing, the Bureau of Reclamation backfilled a large eroded area (34 m long by 10 m wide and 11 m deep) in the spillway at Hoover Dam by this method [Figure 2.2]. This is known as the first important application of PAC method [5,6].

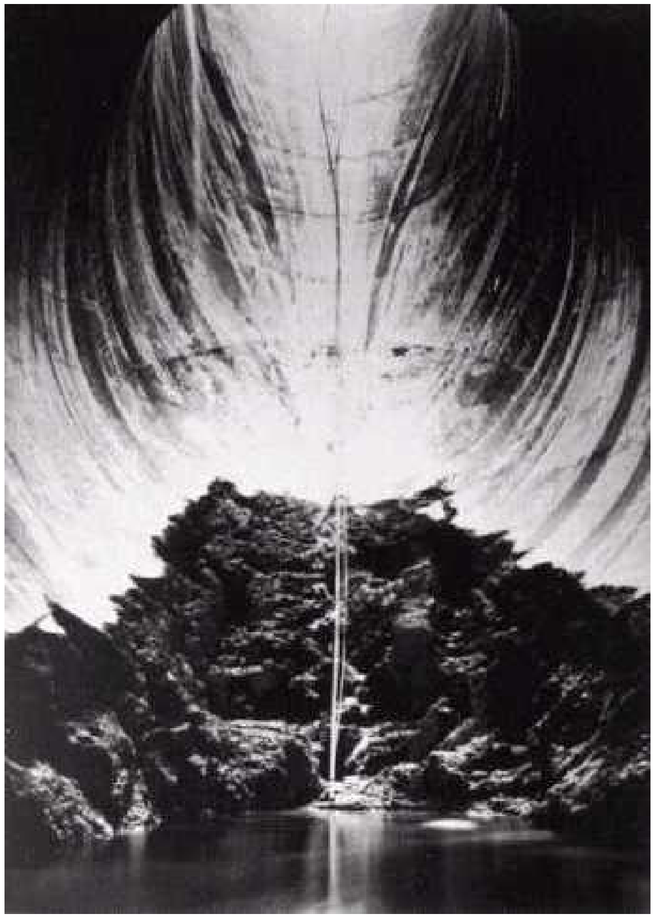

Figure 2.2. *The eroded part of the spillway tunnel at Hoover Dam, which is located 1.5 km below the crest, before it was repaired by PAC method*

The second major application was the addition to the upstream face of Barker Dam at Nederland, Colorado, in 1946 [7]. In this application, 1.8 m concrete slabs were anchored to the 52 m high dam surface, and the space between them were filled with coarse aggregate [Figure 2.3]. Of course this operation was done during winter when the reservoir was empty, and then through the end of the spring the aggregate was grouted in ten days with a continuous pumping operation when the reservoir was full. By this application, it was proved that PAC method was not limited by only small repair works, but it is also suitable for major projects.

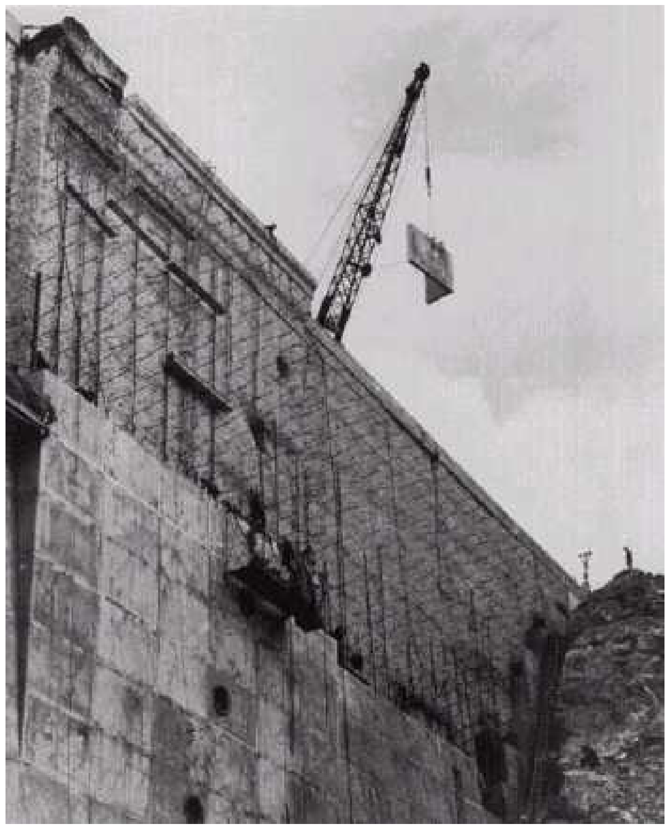

Figure 2.3. *The use of precast concrete slabs as formwork during the refacing of Barker Dam (Nederland, Colorado) by PAC method in 1946*

After the U.S. Army Corps of Engineers permitted the use of PAC method for the embedment of turbine scroll cases in 1951, the turbine scroll case at Bull Shoals Dam powerhouse was built by two lifts of PAC, each 3 m [Figure 2.4].

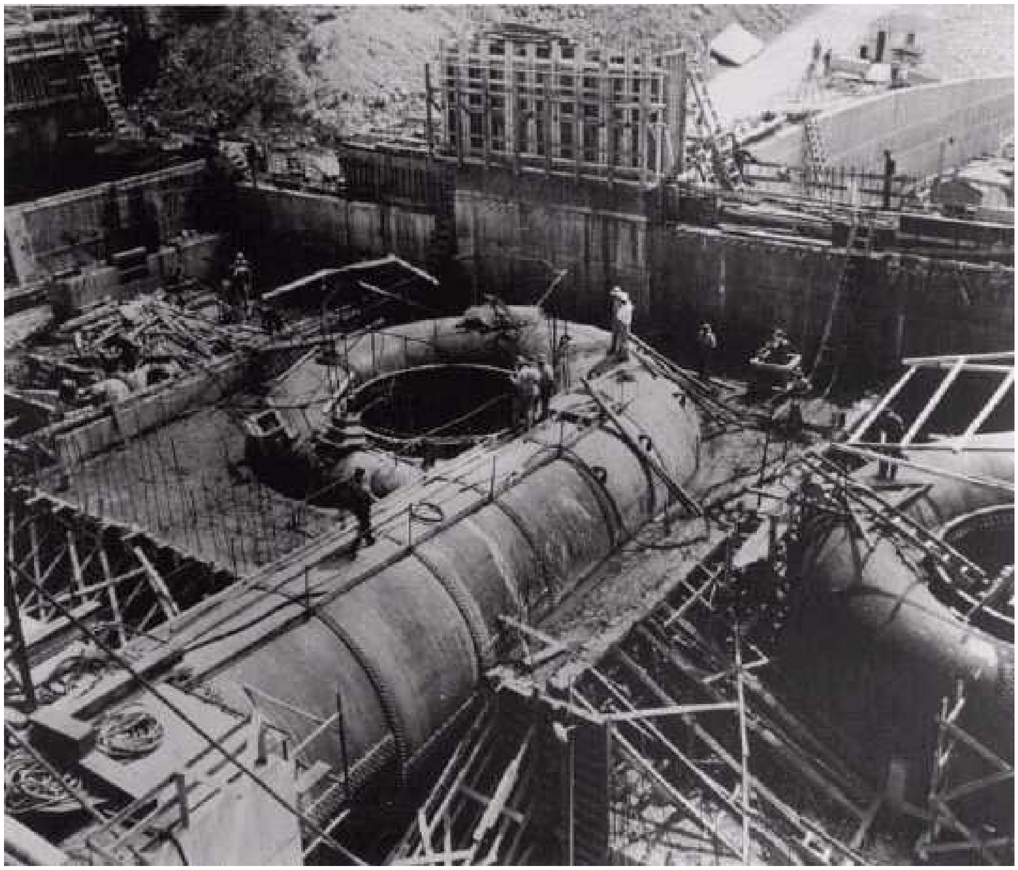

Figure 2.4. *During the construction of the turbine scroll case at Bull Shoal Dam powerhouse after the completion of the first lift of PAC*

Also the Snowy Mountains Authority, in Australia, used PAC method for embedding turbine scroll cases and draft tubes in their hydroelectric power projects.

After the Japan construction companies bought rights to the method in 1950, and built several bridge piers by this method, a new field of application for the PAC method was born: construction of bridge piers. An important example to this field of application is the construction of the 34 piers of the Mackinac Bridge in 1954-1955. Approximately 380,000 m^3 of PAC was used for this project [8].

2.5. Special Properties

The most significant difference of PAC from conventional concrete is that PAC contains a higher percentage of coarse aggregate compared to conventional concrete; because PAC is produced by depositing coarse aggregate directly into the forms, where there is a point-to-point contact, instead of being contained in a flowable plastic mixture as in conventional concrete. Because of this, the properties of PAC depend on the coarse aggregate, more than its other constituents. As a result of this, the modulus of elasticity of PAC is found to be slightly higher, and its drying shrinkage is less than half that of conventional concrete [9,10,11].

On the other hand, the most important property of concrete is no doubt its compressive strength. The strength of PAC depends on the quality, proportioning and handling of the materials, like in conventional concrete. Today in PAC technology, compressive strengths upto 41 MPa at 28 days are attainable. Furthermore, compressive strengths of 62 MPa at 90 days and 90 MPa at 1 year have been reported [7,12]. It seems that the strength of PAC can be increased by using high-range water-reducing admixtures, silica fume, and/or other admixtures, but there is a lack of information about this subject in the literature.

One of the other special properties of PAC is its excellent bonding ability when it is added to an existing roughened concrete. This ability of PAC comes from two reasons:

1. The grout can penetrate through the surface irregularities and pores on the existing concrete surface, and establish an initial bond.

2. The low drying shrinkage of PAC minimizes the interfacial stresses taking place upon drying.

The results of a test show that, the modulus of rupture of beams, which are made by placing PAC on conventional concrete, are more than 80% of that belongs to a monolithic beam, which is made of the same conventional concrete. In another test, again a concrete beam is made by placing PAC over conventional concrete, and several cores are taken from the bonding portion. When these cores are tested under bending, it has been seen that almost all of the specimens broke on either one side of the interface or the other, but not on the interface [2].

As it is mentioned above, when the subject is the properties of concrete, strength has always had priority; however today durability, as a property of concrete, attracts attention at least as much as strength, or in some cases even more [13]. Coming to the durability of PAC, although PAC was produced for many years without air entrainment (except that is introduced by the grout fluidifiers), it is observed that PAC used for repairs, which are normally exposed to severe weathering, has shown excellent durability. For example, a column in the West 6th street Viaduct (Erie, Pennsylvania) still looks like as if recently-repaired, 26 years after being repaired by PAC [Figure 2.5].

Another typical example is the lock wall on the Monongahela River above Pittsburgh, Pennsylvania. From far below the low pool level to the top of the wall, it was found to be visibly in sound condition 35 years after being refaced by PAC [14]. However, despite these successful examples, the durability tests conducted on PAC by U.S. Army Corps of Engineers Waterways Experiment Station laboratory show that air-entrainment is necessary to provide a sufficient durability which is comparable to that of air-entrained conventional concrete [15]; and currently, the specifications given by U.S. Army Corps of Engineers for PAC require an air-entrainment of 9±1 %, which is measured according to ASTM C 231, 15 min. after the completion of the mixing of the grout [16].

Another advantage of PAC method appears in heat of hydration control. Because it is much more feasible to drop the temperature of the concrete by cooling the preplaced aggregates in the forms and then intruding chilled grout, instead of applying expensive precooling and postcooling systems. By this method, initial temperatures as low as 4.5 to 7 ^{0}C are readily obtainable. The details of this subject are discussed at Section 2.6.

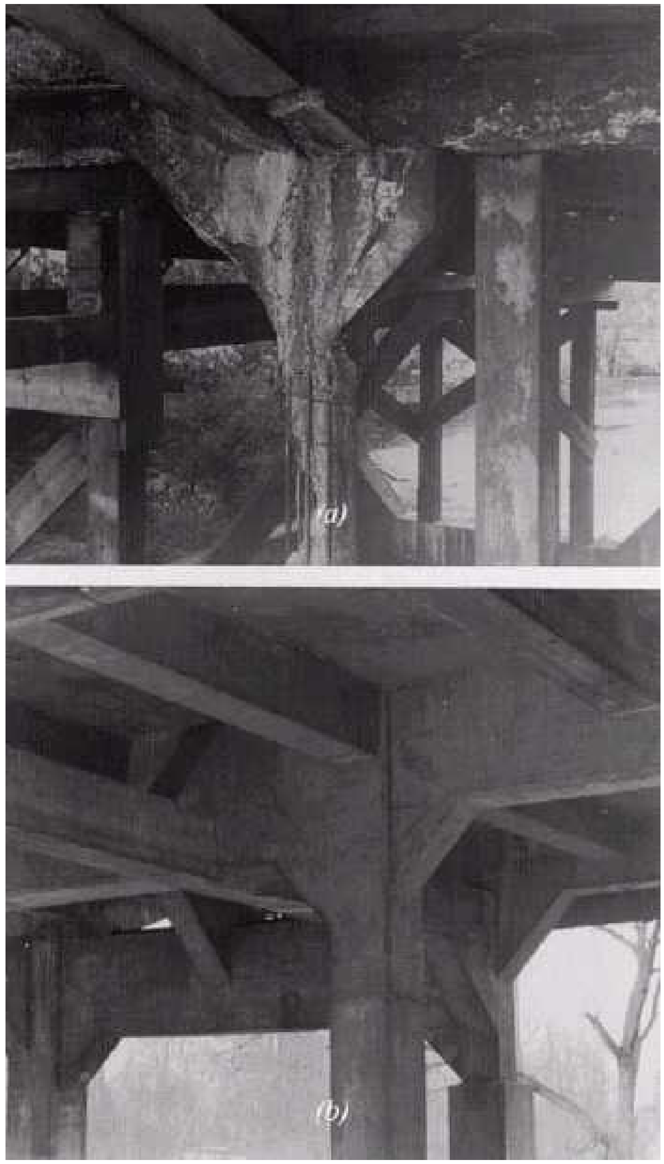

Figure 2.5. *A reinforced concrete viaduct column (West 6th street, Erie, Pennsylvania)*
(a) before repair and
(b) 26 years after being repaired by PAC method

2.6. Temperature Control

As in conventional concrete, the peak temperature of PAC should not exceed certain limits in order to avoid thermal cracks. The peak temperature depends on two factors:

1. the initial temperature of the concrete
2. the temperature rise due to hydration

PAC is already advantageous from initial temperature point of view, because there is no mixing of coarse aggregates with the other ingredients, which means there is no temperature rise due to the friction of the coarse aggregates. However the initial temperature of PAC can be reduced furthermore following one or more of the procedures explained below.

Coming to the second factor, the temperature rise depends on the amount and rate of heat released upon hydration [17]. The amount of heat can be reduced by reducing the amount of cementitious materials used in the mixture; in other words, by using a mixture as lean as possible in the limitations of design requirements. Using low-heat cement also helps in minimizing the amount of heat released [18]. On the other hand, the rate of heat evolution can be reduced by using high proportions of pozzolans (such as fly ash) in the cementitious materials content. This may also cause a reduction in the rate of strength development, but it is welcome, because in massive structures, where heat of hydration control has a priority, early strengths are not required; 90 days strengths are acceptable. The slower rate of strength gain results in slower rate of heat release, and that provides an additional time for heat dissipation [19].

As explained above, controlling the temperature of PAC means controlling the two factors simultaneously, the first of which is the initial temperature. Cooling the preplaced coarse aggregates is a procedure applied to reduce the initial temperature of PAC. The aggregates can be cooled either before placement or in place, but because of the time delay between the placement of aggregates and injection of the grout, cooling of the aggregates before placing in the forms is not recommended. In marine installations like bridge piers, there is no need to spend extra effort to cool the aggregates, because the aggregates get cooled automatically when they are deposited in cold water. For structures above water, the aggregates placed in the forms can be cooled by introducing cold water from bottom of the forms and drawn off at the top, until the desired aggregate temperature is obtained. Another way of cooling the aggregates in place is spreading crushed ice on the top [Figure 2.6].

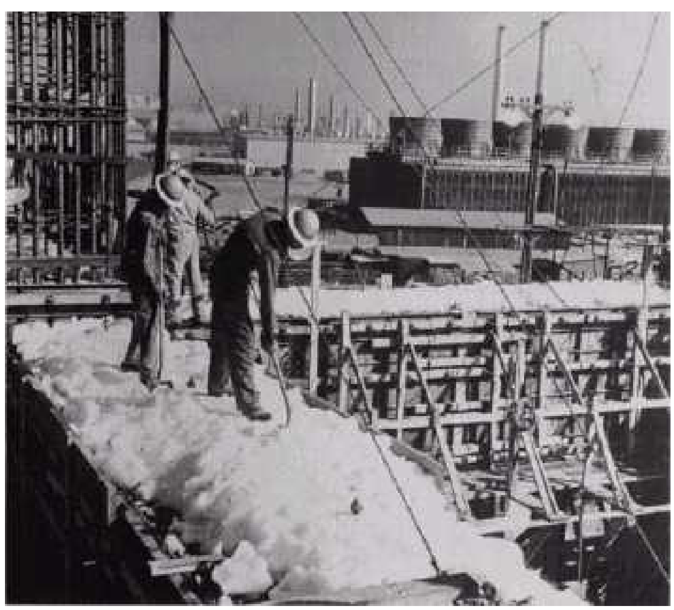

Figure 2.6. *The utilization of ice for cooling the coarse aggregate in place*

This procedure cools the aggregate by two mechanisms:

1. by allowing the cold air to settle through the voids in the aggregate mass
2. by trickling down cold water from the melting ice

This is an effective but time consuming procedure [20]. On the other hand, cooling the aggregate with liquid nitrogen has been reported to be successful, but the details of such an application are not available in the literature.

Cooling the grout is another procedure which is applied to reduce the initial temperature of PAC. This may be done by using cold mixing water, but unless the other dry materials are also cooled during the storage, it is not so effective. The more effective procedure is to introduce some portion of the mixing water as ice; because while it takes 1 cal. energy to raise the temperature of 1 gr. water by 1 ^{0}C, 1 gr. ice absorbs 80 cal. energy for melting. By this way, grout temperatures as low as 4.5 ^{0}C have been obtained. However there is one point to be paid attention in this procedure: It should be insured that mixing continues until all the ice particles melt, before starting to pump the grout. The extension of mixing time due to the amount of ice substitution can be determined by trial mixtures. Another advantage of using chilled grout is the reduction in the total mixing water requirement due to the increased fluidity upon chilling.

Although the topic is 'Temperature Control of PAC', upto this point it has been discussed how to avoid excess temperatures. However when the subject is temperature control, sometimes it is required to keep the temperature of the concrete above some critical limits, especially in cold weather placement. In such cases, the precautions and the limiting conditions stated for the conventional concrete are also valid for PAC [21]. However there are also some additional precautions peculiar to PAC; for example, for the grout fluidifier to work properly, the temperature of the grout should not drop below 4.4^{0}C. If the preplaced coarse aggregate is colder than this temperature (but not below 0^{0}C), in order to keep the temperature of the grout above this critical value, the grout may be heated by using warmed ingredients. As a general recommendation, grout temperatures above 10^{0}C in monolithic PAC and 15^{0}C in patches, where the cold concrete base acts as a heat sink, may be used to provide a suitable in-place temperature. As another recommendation, if a repair work has to be done in severely cold weather, the member of the existing structure to be repaired should be enclosed and heated before the placement of PAC, in order to insure a concrete base temperature above the freezing point [2].

2.7. Materials

2.7.1. Aggregates

As in conventional concrete, both fine and coarse aggregate should conform to the requirements given at ASTM C 33, except the grading limits [22]. The recommended grading limits for the aggregates of PAC are given at Table 2.1 [23].

Either manufactured or natural sand may be used as fine aggregate, which should be hard, dense and durable uncoated rock particles. Generally 95% of the fine aggregate is smaller than 1.2 mm and 100% of it is smaller than 2.4 mm [24]. Crushed stone or natural gravel can be used as coarse aggregate, which should be clean, free of surface dust and fines [25]. If not, it should be subjected to washing and screening operations before being used [Figure 2.7].

Table 2.1. *The Recommended Grading Limits for the Coarse and Fine Aggregates to be Used in PAC production*

PREPLACED AGGREGATE CONCRETE

Table 1 — Grading limits coarse and fine aggregates for preplaced aggregate concrete

Sieve size	Percentage passing		
	Grading 1 For 1/2 in. (12.5 mm) *minimum* size coarse aggregate	Grading 2 For 3/4 in. (19 mm) *minimum* size coarse aggregate	Grading 3 For 1-1/2 in. (38 mm) *minimum* size coarse aggregate
	Coarse aggregate		
1-1/2 in. (37.5 mm)	95-100	—	0.5
1 in. (25.0 mm)	40-80	*	—
3/4 in. (19.0 mm)	20-45	0-10	—
1/2 in. (12.5 mm)	0-10	0-2	—
3/8 in. (9.5 mm)	o-2	0-1	—
	Fine aggregate		
No. 4 (4.75 mm)	—		100
No. 8 (2.36 mm)	100		90-100
No. 16 (1.18 mm)	95-100		80-90
No. 30 (600 microns)	55-80		55-70
No. 50 (300 microns)	30-55		25-50
No. 100 (150 microns)	10-30		5-30
No. 200 (75 microns)	0-10		0-10
Fineness modulus	1.30-2.10		1.60-2.45

*Grade for minimum void content in fractions above 3/4 in. (19 mm).

Figure 2.7. *The coarse aggregates are being washed by rotary screen in order to remove the undersize particles*

While determining the gradation of the coarse aggregate, the most important criterion is the minimization of the void content; because, as the void content of the coarse aggregate is minimized, the amount of grout needed to fill these voids is minimized, and this means not only economy but also less temperature rise due to the reduction in the cementitious materials content. In general, the minimum void content is achieved when the coarse aggregate is graded from the smallest allowable particle size to the largest, which is limited by the thickness of the section and the spacing of the reinforcement. However in mass concrete, the maximum allowable size of coarse aggregate is limited only by feasibility of handling. The void dimensions, through which the grout must pass, are determined by the minimum size of the coarse aggregate used. Hence, the minimum size of the coarse aggregate and the maximum size of the fine aggregate to be used are closely related. If uniformly sized coarse aggregate is used, the void content reaches its highest value with 50%. This value can be reduced upto 35% by using an aggregate which is well graded between 19 mm and 150-200 mm. Even it has been reported in the literature that void contents as low as 25% have been achieved by applying a successful gap grading, in which half of the aggregate was 12-38 mm and the other half was 200-250 mm [2].

2.7.2. Cementitious Material

Except air-entraining type of cements, any type of cement, portland or blended, which satisfies the requirements of the related standards, can be used for the preparation of the grout [26,27]. However there is a lack of data in the literature about the use of blended cement in making PAC. Use of air-entraining type of cements in making PAC is not recommended, because use of them combined with a grout fluidifier which also has an air-entraining property causes an intolerable reduction in the strength. However, if the amount of air entrained by the grout fluidifier is not sufficient for the required freeze-thaw durability, air-entraining admixture can be used separately; but in this case, the dosage of the air-entraining admixture should be determined very carefully by laboratory tests.

As in selection of the cement type to be used, there are no strict limitations in the selection of pozzolans. Any type of pozzolans, natural or artificial, which satisfy the requirements of the related standards, can be used [28,29]. However in majority, class F fly ash is preferred, since it improves the pumpability of the grout and extends the grout handling time. Moreover it provides the same properties to PAC as it does to conventional concrete [30]. Though to a limited extent, some applications of class C fly ash and blast furnace slag have also been reported, but there is not enough data in the literature about the details of these applications, such as mixture proportions of the grout and properties of the resulting PAC. On the other hand, there is no data in the literature about the application of silica fume in grout for PAC.

2.7.3. Admixtures

Being the only admixture mentioned in the definition of PAC, grout fluidifier is the main chemical admixture used in PAC production [1]. A grout fluidifier is a commercially obtained material, which is preblended customarily. There is not any given standard composition for grout fluidifiers, but in order to satisfy the requirements given in the related standards, normally it is composed of a water reducing admixture (plasticizer), a suspending agent, aluminum powder, and a chemical buffer [31]. As a result of this composition, a grout fluidifier has the following effects on the grout in which it is used:

1. As an effect resulting from the plasticizer it contains, not only it reduces the water-cementitious material ratio for a given fluidity, but also it provides a longer grout handling time (in both mixing-pumping

cycle and in penetration through the voids in the coarse aggregate mass) by retarding stiffening.

2. The suspending agent in it helps the grout fluidifier to offset the adverse effect of bleed water that normally tends to collect under the coarse aggregate particles.

3. The aluminum powder in it gives reaction with the alkalies in portland cement, and as a result of this reaction hydrogen gas is generated, which causes expansion of the grout while it is fluid, and leaves minute bubbles in the hardened grout. (The role of the chemical buffer used in the composition of the grout fluidifier is to insure the timing of this reaction.)

The normal dosage of grout fluidifier is 1% by weight of the total cementitious material (i.e. cement plus pozzolan, if any pozzolan is used) in the grout mixture. This means that, in the laboratory tests, 1% of fluidifier should satisfy the expansion limits given in the related standard [31]. The grade and type of the aluminum powder used in the composition of the fluidifier should be selected to produce almost all of this expansion within 4 hours. Another important point to be paid attention while testing the qualification of the fluidifier is that, the amount of bleeding, which is determined according to the related standard, should not exceed the amount of expansion, which is also determined in accordance with the same standard, using the job materials [32].

As it is also mentioned in Section 2.6, the grout fluidifier can not work properly below 4.4^{0}C and the expansion of the grout caused by the fluidifier ceases at temperatures below this value. Because of this, the grout should be placed in an environment where the temperature will rise above this value. In fact, there is not such a problem in massive sections and in placements enclosed by timber formwork, because the heat generated by the hydration of the cement normally raises the internal temperature sufficiently high for the grout fluidifier to perform properly [2].

The amount of air entrained by the hydrogen gas generated during the expansion mechanism of the grout fluidifier is usually sufficient for a reasonable freeze-thaw resistance, but in more severe weather conditions, where this amount of entrained air is not sufficient for durability, air-entraining admixture can be used for additional freeze-thaw resistance. Any air-entraining admixture that complies with the related standards can be used in PAC production [33]. However the user should be more careful in adjusting the dosage of the air-entraining admixture while using it in PAC production than in conventional concrete, because in PAC, the total air in the hardened grout is the sum of that introduced by the air-entraining admixture and by the grout fluidifier; thus, though the air-entraining admixture is not used at over-dosages compared to the given limits for conventional concrete, when this combined effect is concerned, the reduction in the strength due to the air content may be over tolerable limits. However, though the dosage of the air-entraining admixture is adjusted very carefully, if the reduction in the strength is still intolerable, the solution should be the readjustment of the mixture proportions, because the air content can not be reduced below certain limits to insure the durability [15].

$CaCl_2$ (Calcium Chloride) is another chemical admixture that can be used in PAC, in order to accelerate the rate of strength development and to obtain high early strengths, as it is utilized in conventional concrete, provided that it satisfies all the requirements given in the related standards [34,35]. However when it is used more than 1% by weight of total cementitious material in the grout mixture, it has a side effect as depressing the expansive action of the grout fluidifier. Another reason for a limitation on the dosage of this admixture comes into account when reinforcement is present, because of the widely-known corrosive effect of the chloride ion (Cl^-) on steel [36].

Other than those mentioned above, there are some chemical admixtures which can be considered for special conditions. For example, a Type D admixture (water-reducing and retarding) has been reported to be used successfully to increase grout stiffening time from 15 to 60 minutes. By the way, the setting time of grouts for PAC is determined by the same apparatus used to determine the setting time of cement pastes for conventional concrete (i.e. by Vicat apparatus); but according to a different standard [37]. Similarly, Type F and G admixtures (i.e. High-range water reducing admixtures, in other words superplasticizers) appear to be potentially useful, but unfortunately there is no data available in the literature about their application on PAC [34].

2.7.4. Mixing Water

As in conventional concrete, the mixing water to be used in the preparation of the grout for PAC should be as clean as possible, and free from any kind of chemical or organic material that can affect the properties of the resulting concrete adversely [38].

2.8. Mix Design

Since the coarse aggregate is handled separately in PAC, when it is talked about the mix design of PAC, in fact it means the mix design of the grout. As in conventional concrete, the mix proportions, which are determined in accordance with the related standards, are specified by weight [39]. However as a partial exception, when the size and location of the work do not allow the use of on site weigh-batching equipment, volumetric batching can be used. In such a practice, all mixture proportions are rounded off to whole bags of cement. For example, 2:1:3 is a typical mix design which is used for small routine bridge pier repair works in USA. Here, 2:1:3 means 2 bags of cement (43 kg), 1 bag (32 kg) of pozzolan (usually fly ash) and 3 ft^3 (0.085 m^3) damp sand. In order to determine the water-cementitious material ratio for a suitable consistency, the first trial mixture is made using 5 gal. (0.019 m^3) of water and then it is adjusted in later batches by increasing or decreasing accordingly.

The most important criteria while determining the mix design of the grout for PAC is the consistency of the grout; because if the grout does not have a good consistency, it would fail to fill all the voids in the coarse aggregate mass. The grout consistency is determined by flow cone method [36]. In this method, 1725 ml of grout is poured into a funnel which has a discharge tube of 12.7 mm and the time of efflux of the grout is observed [Figure 2.8].

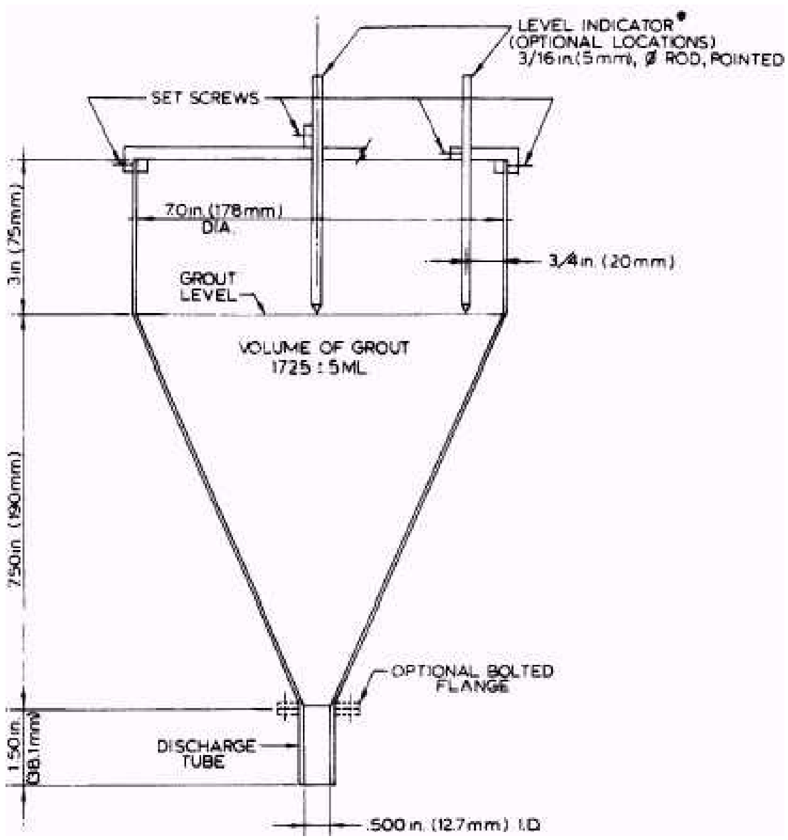

Figure 2.8. *Cross section of the flow cone used to determine the consistency of grout for PAC [40]*

Usually a grout with a time of efflux of 22 ± 2 sec. is satisfactory for most of the works, such as walls and structural repairs. On the other hand, for underwater constructions and massive sections where the maximum size of coarse aggregate is larger, it is practical to use a grout mixture with a consistency corresponding to a time of efflux ranging between 18-26 seconds. For special works where higher strengths are required, grout with a time of efflux as high as 35-40 sec. can be used. In order to be able to make a comparison, it would be useful to note that the time of efflux for water in flow cone method is 8.0 ± 0.2 seconds. Another note about this subject is that, for flow cone method to be applicable, the grout must have a fine aggregate with Grading 1 or 2 (Table 2.1). When a Grading 3 fine aggregate is used, the flow cone must be replaced by another suitable device to determine the consistency of the grout. For example, the flow table apparatus used for determining the consistency of mortar in conventional concrete can be a good alternative [41]. If this method is preferred, a flow of approximately 150%, which is measured after 5 drops in 3 sec., would indicate a suitable consistency for the grout to flow through the voids in the preplaced coarse aggregate mass [2].

After consistency, the next important criterion while determining the mix design of the grout for PAC is that, the strength of the PAC made by using this grout should meet the required values [42]. For optimal results between consistency and strength, bleeding should be less than 0.5%; but in any case, the bleeding measured at in-place temperatures must not exceed the expansion [32]. Since it does not reflect the weakening effect of bleeding, testing of the grout alone in cubes or cylinders for prediction of the strength of PAC is not recommended. In fact there is a standard about testing the grout alone under compression, but it is not for prediction of concrete strength, it is used for verification of the grout fluidifier [43].

While determining the mix design of the grout, it is preferred to use fine aggregate (sand) as much as possible; because as the proportion of sand increases, the amount of cementitious material needed to produce the same amount of grout is decreased, which means less heat of hydration and more economical concrete. However there are some factors which limit the sand carrying capacity of the grout; such as compressive strength, pumpability and void penetrability requirements. For example, if a Grading 1 (Table 2.1) fine aggregate is used, for the application of PAC in beams, columns, and thin sections, a cementitious material to sand ratio of 1:1 would be suitable. For the application of PAC in massive sections where the nominal size of the coarse aggregate is around 19 mm, this ratio can be increased upto 1:1.5. Even, if fine aggregate with Grading 3 is used and if the equipment is appropriate for pumping, a cementitious material to sand ratio as high as 1:3 can be applied [9, 44].

As in conventional concrete, a part of the portland cement amount intended to be used can be replaced by pozzolans. In such cases, the proportion of pozzolan to portland cement is usually in the range of 20-30% by weight; in other words the proportion of pozzolan in the total amount of cementitious material (i.e. pozzolan plus portland cement) is 17-23%. As the total amount of cementitious material used gets higher, the PAC provides comparable strengths with conventional concrete having the same proportions of pozzolan and portland cement. As the mixture gets leaner, the strength obtained by the conventional concrete at 28 days can hardly be provided in 60-90 days by the PAC which has the same proportions of cementitious materials. The proportion of pozzolan in the total amount of cementitious material can be increased upto 30% where low heat of hydration is required (as in mass concrete), and can be decreased below 10% where comparatively higher strength values are required. Though very rarely, in some cases even no pozzolan is used at all [30].

2.9. Special Grouts

There are some special grout mixtures readily available in the market, which are called ‘prepackaged grout products’. As their name implies, they are produced by premixing a proper cementitious material composition with a properly graded fine aggregate, and served in packages as commercial products. They are ready to use only after being mixed with a proper amount of water that will provide the desired consistency. This amount of water can either be specified by the manufacturer or determined by the user according to the requirements of the job; but in both cases, the grout mixture should be capable of remaining at a suitable consistency for a sufficient period of time to permit a proper intrusion into the preplaced aggregate. Some prepackaged grouts tend to stiffen rapidly, but use of retarding admixtures is not recommended, because there is not enough data available in the literature about the compatibility of retarders with the ingredients in premixed grouts. There are types of prepackaged grouts with different mix designs, which are formulated to be used for different purposes. However in any case, the maximum size of fine aggregate used in them should meet the requirements given at Table 2.1.

Another type of special grouts is resinous grout, where a two-component epoxy is used as cementitious material. This type of special grout is particularly preferred when the resulting PAC is going to be cast against an existing concrete structure, in order to obtain a bond strength which is equal to concrete strength. It may also be used where high-early strength is needed. Epoxies produce large amount of heat while they are hardening, because of this the most important thing to be cared while using this type of grout is the optimization of the epoxy formula in such a way to minimize the exothermal potential. Also the preplaced aggregate must be completely dry in order to prevent steam generation during the hardening of the grout. Because of this, cooling the preplaced aggregate by sprinkling cold water or by spreading over crushed ice is not suitable when this type of grout is used; instead, utilization of compressed or liquid gas (such as nitrogen) is recommended. Another measure against thermal effects is limiting of the thickness; for example in surface patches, to approximately 5 cm. However for massive sections, postcooling may be necessary to remove the heat as it is generated, by circulating cold water through the piping installed in the section during construction. Lastly, the other two things to be cared while using epoxy resin grouts are that, the epoxy should have a pot life of at least 30 min. and its viscosity should be as low as possible [2].

2.10. Construction Practice

In general, the procedure to be followed in PAC construction practice are listed below step by step and in order of execution:

1. Preparation of existing concrete surfaces
2. Installation of reinforcement and pipes
3. Erection of formwork
4. Placement of coarse aggregate
5. Mixing and pumping of the grout
6. Finishing and curing

It should be noted that step 4 may coincide with step 2 and 3, if the placing conditions are difficult because of closely spaced reinforcement or if high lifts of joint-free in-place concrete are desired.

2.10.1. Preparation of existing concrete surfaces

If the PAC is used to repair surface defects, or cast as an addition to an existing concrete structure, in order to establish a good bond between the new PAC and the existing old concrete, the surface of the existing concrete must be cleaned very carefully by removing all deteriorated concrete till the sound concrete is reached, and a space which is at least four times the maximum coarse aggregate size should be provided behind the existing reinforcement, or the new reinforcing to be added [Figure 2.9].

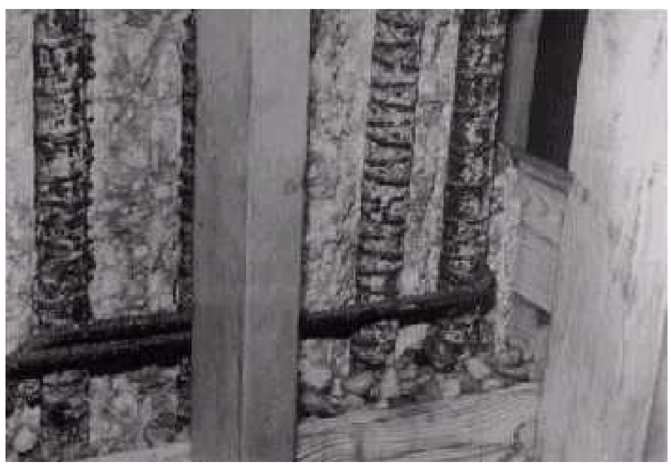

Figure 2.9. *After the existing concrete surface is prepared properly, the coarse aggregate is placed as the timber forms are erected*

2.10.2. Installation of Pipes

After the preparation of the existing concrete surface, the next step, as listed above, is the installation of the pipes through which the grout will be pumped. For this purpose, usually Schedule 40 type of pipes are used, and the diameters of the pipes are normally 2-3 cm for usual concrete structures. However for mass concrete, pipes with diameter upto 4 cm can be used.

The grout insert pipes can be located either vertically or horizontally through the formwork. If they are located vertically, they should extend to the bottom of the preplaced aggregate within 15 cm. If they are located horizontally, they should be set at different elevations and occasionally with an angle to permit the injection of the grout around the embedded items or into the restricted areas.

During the grouting operation, the pipes should be withdrawn in such a way that the ends of the pipes always remain at least 30 cm below the grout surface. If the depth of the preplaced aggregate exceeds 15 m, Schedule 120 type of pipe is recommended. If the pipe extends to a depth of 30 m or more in the preplaced aggregate, such as while building bridge piers in very deep water, it may be difficult to withdraw it because of the friction. In such cases, using an additional pipe may be required. For example, while building the Mackinac Straits Bridge piers, a pipe with a diameter of 2.5 cm was placed to the full depth, and a pipe which is 5 cm in diameter was slipped over it to the half depth. After the lower half of the whole depth was grouted by the pipe with 2.5 cm diameter, this pipe was totally withdrawn with the help of the surrounding larger pipe, and then this larger pipe was used to grout the remaining half of the depth above.

In general, the spacing of the pipes ranges between 1.2-3.7 m, but the most commonly used spacing is 1.5-1.8 m. If more than one pump is used, it should be identified which pipes are served by which pump to prevent confusion.

Straight pipes are preferable since they can be cleaned by rodding if they become obstructed and they can be totally removed after the grouting is complete, but in case of an embedment which does not permit the use of straight pipes, non-removable curved pipes can also be used. In such a case, extra pipes should be placed, since it will not be possible to clean curved pipes by rodding if some of them become obstructed.

For shallow injections, like surface repairs and sections thinner than 46 cm, grouting may be accomplished through pipe nipples screwed into the forms. Spacing of these injection points can be as little as 50-90 cm for sections as thin as 10 cm, and as big as 90-150 cm for thicker sections.

Another type of pipes besides the ones used for grout injection is vent pipes. As the name implies, they are used for ventilation. Because of this they must be located into the areas that are likely to trap air and water as the grout rises in the coarse aggregate. They may be placed either before or concurrently with the reinforcement, like the injection pipes, according to the placing conditions [2].

2.10.3. Formwork

After the pipes are inserted, the next step is the erection of the forms. Forms should be designed and erected according to the same guidelines followed in conventional concrete, but while designing formwork for PAC, one should keep in mind that the pressure exerted by the grout on the forms is a result of the static head of the grout; grout pumping pressure is not a factor, because grout moves through the preplaced aggregate so freely that the pressure in the grout pipes is dissipated at the exit of the pipe by the transfer of the grout from a restricted diameter to comparatively a much more free space [45]. As it is known, the pressure on a container exerted by the fluid in it is equal to the static head of the fluid multiplied by its density. A grout weighs approximately 2080 kg/m^3 and assuming a 3 m head of grout, in most projects the minimum static grout pressure is taken as 0.07 MPa while using the standard form design tables. However for deep and massive placements, such as bridge piers, additional allowance should be made for the lateral load resulting from the big mass of ungrouted coarse aggregate. Also, if the PAC method is used for heavyweight concrete application, the constant 2410 kg/m^3 in the formulas given for calculation of the lateral pressure exerted on the forms should be replaced by the actual unit weight of the PAC. If differential placement to be done, a rule of thumb used by the site engineers is that, at 21 ^0C grout in preplaced aggregate stiffens sufficiently in 4 hr to resist superimposed pressures of upto 0.03 MPa, which is approximately equivalent to 1.5 m of fluid grout [46].

Another important point about the formwork for PAC is that, there should not be any leakage [24]. In order to provide this, the workmanship must be of very high quality. In fact the injected grout fills the small voids on the formwork and stops water seepage, but this does not work for openings wider than 1.5 mm. All the penetrations on the form panels like anchor bolts and the joints between the form panels that do not match perfectly should be sealed with a self-adhesive tape from inside; the use of mastics that do not harden for this purpose is not recommended, because they tend to blow out as the grout rises behind the forms.

About the materials that the forms can be made of, plywood is frequently employed for small jobs where tailoring at the job site is necessary. For larger projects, including nuclear shields, steel forms are preferred. Even precast slabs of air-entrained concrete have been employed successfully as formwork for the refacing of large concrete dams by PAC method [7,11]. On the other hand, for underwater construction, such as bridge piers, steel sheet piling is most frequently used. However for deep-water piers, where the placement of coarse aggregate is done intermittently and the grouting is continued as the new layer of coarse aggregate is placed, it should be paid attention to provide a sufficient internal anchorage for the sheet piling; because after a day or more of pumping, the layers below will start to get hardened, and the pressure of the fresh grout injected into the new layer of coarse aggregate placed above may cause the deflection of the piling, if its internal anchorage is inadequate. This will lead the flow of grout between the piling and hardened concrete, which will cause further deflection, and may be even result in the failure of the formwork [2].

2.10.4. Placement of Coarse Aggregate

Before placement, all the coarse aggregate should be washed and screened carefully to remove dirt and to eliminate undersized particles. Washing the coarse aggregate after being placed in the forms should be strictly avoided, because it will cause the accumulation of the fine particles at the bottom of the forms, and this will result in a poor bottom surface or an unbonded joint [47]. If more than one size of aggregate is used, the sizes should be well mixed before starting to be placed. The coarse aggregate is commonly conveyed to the forms by either dump trucks or conveyors. If the aggregates have to be dropped from a height of over 1.5 m, special care must be taken to minimize breakage and segragation. A steel pipe having a diameter of at least four times the maximum aggregate size can be used for this purpose [Figure 2.10]. The rate of aggregate flow is controlled by keeping the lower end of the pipe slightly below the surface of stone already deposited.

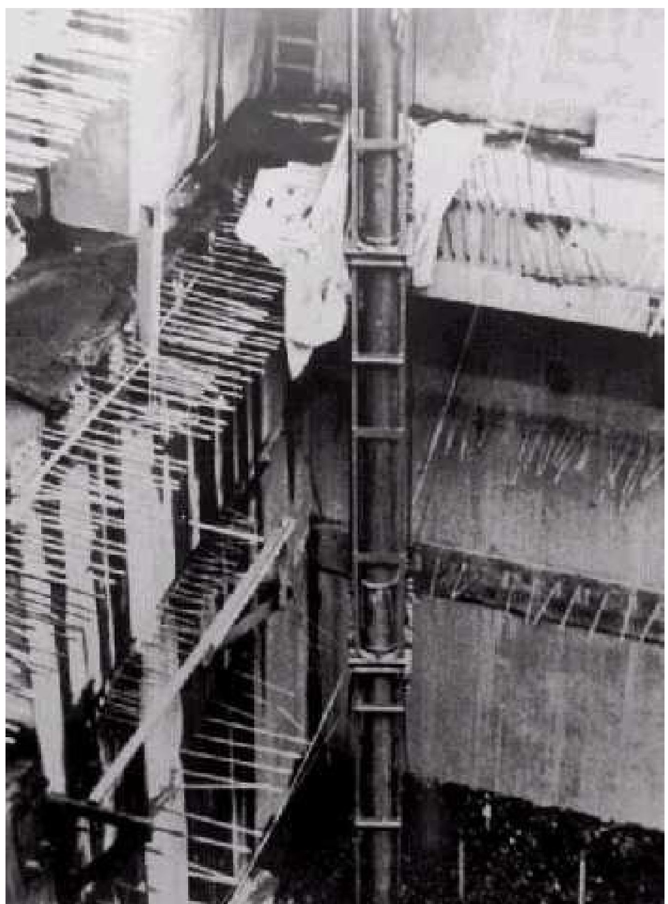

Figure 2.10. *The utilization of a tremie pipe for lowering the aggregate from a height of 15 m during the embedment of draft tubes (Tumut III Hydropower Plant, Snowy Mountains Project, Australia)*

It is reported in the literature that this method of placing coarse aggregate has been used successfully for lowering aggregate from a height as much as 300 m [48].

For underwater construction, as in bridge piers, the coarse aggregates can be dropped directly into the water, because the aggregates falls down slowly enough in the water to avoid particle breakage, and the segregation caused by the difference in the falling rates between the different sized particles is negligible for the size range used. The most significant example in the literature to such an application is the piers of the Mackinac Bridge, which was constructed by PAC method in 1954-1955. The huge amount of coarse aggregate needed for the production of approximately 380,000 m^3 PAC, which was consumed during the construction of 34 piers, was placed from huge self-unloading boats as shown at Figure 2.11. The aggregates were placed at a rate of approximately 2000 ton/hr, and were chilled to 4.4-7.2 ^0C by the sea water in the forms which was as deep as 60 m [8].

Figure 2.11. *The coarse aggregate was placed from the self-unloading boat seen on the right into the pier form seen at the middle and then grouted by the semi-automatic plant seen on the left during the construction of Mackinac Bridge*

Another important point about underwater construction is that, there may be some organic contamination in the water, which may accumulate on the immersed aggregate and affect the quality of the concrete adversely. Because of this, the water should be sampled and tested before the placement of aggregate. If it is clean, the aggregates can be allowed to remain in situ for upto 6 months before the grouting operation without apparent adverse results. If it is suspected, the grouting operation should be done as soon as possible after the aggregate is placed in water. If the contaminants are present in such quantity or of such character that the harmful effects can not be eliminated, or if the construction schedule imposes a long delay between the aggregate placement and grout injection, then the PAC method should not be used [49].

In general there is little to be gained from consolidating the coarse aggregate in place by rodding or vibration, but while placing aggregate in congested reinforcement or in overhead repair areas, it may be needed to consolidate aggregates by rodding or by utilization of compressed air [Figure 2.12].

Figure 2.12. *An overhead repair with congested reinforcement where the need for consolidation of preplaced coarse aggregate is unavoidable*

Even hand placing may be required in some placements around closely spaced piping, reinforecements and other penetrations, like in some nuclear shielding situations, as illustrated at Figure 2.1 [3,4].

2.10.5. Mixing and Pumping of the Grout

The standard batching order of grout ingredients into the mixer is water, grout fluidifier, cementitious material, and sand [7]. In general, the fluidifier, and the other chemical admixtures if used, should be added with water for a good distribution in the grout mixture, but if additional retardation is desired, as in hot weather situations, the grout fluidifier may be added after the cementitious materials have been mixed with water for a few minutes.

At the time grouting operation starts, the preplaced coarse aggregate and any existing concrete surfaces must be in a saturated condition, otherwise they may absorb some of the water in the grout mixture, and affect the consistency of grout adversely. Because of this, if the placement is not under water, it is a good practice to fill the forms with water after the placement of aggregates and before starting grouting. By this way, not only the aggregates are saturated with water, but also the forms can be checked for any excessive leakage. If the preplaced coarse aggregates or the existing concrete surfaces that the new concrete will get in touch with are initially dry, it is recommended to remain the forms filled with water for at least 12 hr before starting the grouting operation. After waiting for a sufficient time to achieve saturation, the water may be drained by pumping. It is also recommended to wet the inside of grouting pipes and the hose that is used to transfer the grout from the pump to the pipes before starting the grouting operation, because if the grout is pumped through a dry hose or pipe, it may loose its consistency as a result of the absorption of its water by the dry surfaces. Also it is a good practice to check whether the grout comes out of the hose with the same consistency as it is pumped from the mixer, before connecting the hose to the grouting pipe.

As the equipment used for grout mixing, vertical-shaft paddle-type double-tub mixers are commonly used for preparing grout on small jobs [Figure 2.13].

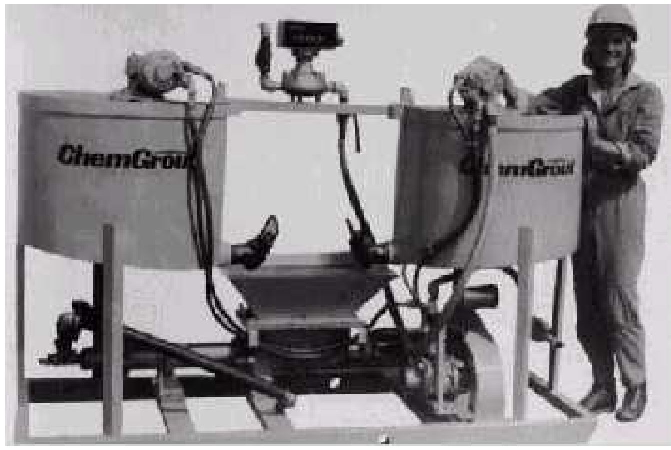

Figure 2.13. *A commercially available double-tub grout mixer*

The capacity of the mixer tubs ranges from 0.2 to 0.4 m^3 and the requirement for power supply ranges from 8.3 to 16.7 hp/ m^3. One of the two tubs serves as a mixer and the other acts as an agitator to feed the grout pump. Both mixers can be driven from a common shaft, which uses gasoline, electricity or compressed air as the power source, but individual air motors for each tub are preferable, because this type of mechanism provides separate control of operation speed for each mixer, which ranges from 60 to 120 rpm. The maximum grout output rate of these mixers is 0.077 m^3/min. If large volume of grout output is required, horizontal-shaft mixers discharging by gravity into a third agitating mixer can be used instead [Figure 2.14].

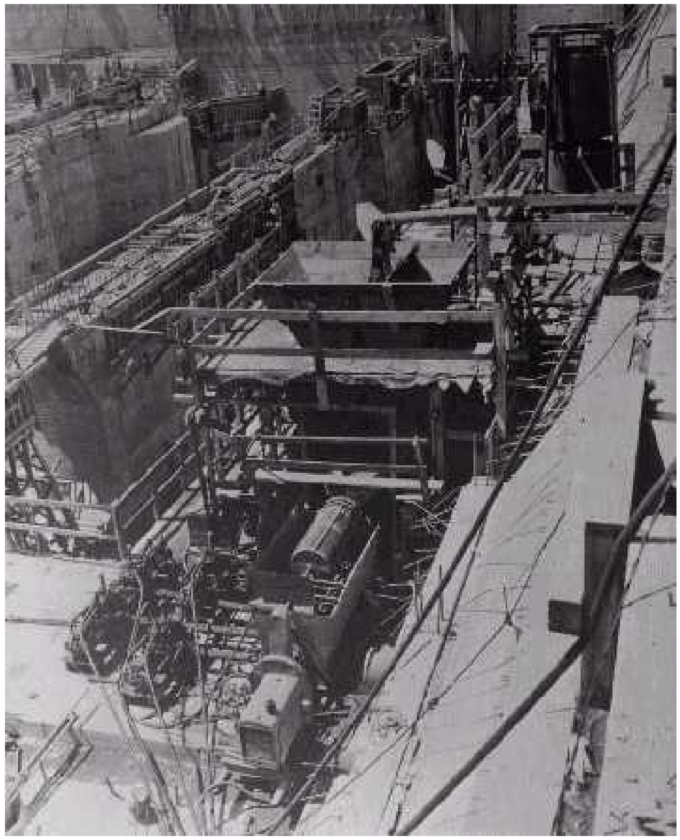

Figure 2.14. *The grout mixing and pumping plant which was used during the embedment of turbine scroll cases of Bull Shoals Dam by PAC method in 1951*

Ready-mixed concrete plants are another source of grout, where large quantities are needed, but the transit time to work site should be less than 30 min. for a grout which has an acceptable pot life of over 2 hr. The revolving-drum mixers used for mixing conventional concrete can also be used for mixing grout for PAC, but the mixing period should be prolonged enough to insure a proper mixing. The colloidal or shear mixers, which mix the cement and water first and then the cement slurry with sand, provide a relatively bleed-free mixture, but because of the high energy input, mixing time should be as short as possible to avoid heating up the grout. The pan or turbine-type concrete mixers are another alternative for mixing grout, but maintenance of a sufficiently tight seal at the discharge gate can cause trouble.

Before transferring the mixed grout from the mixer to the pump(s), it should be passed through a screen, with openings 6-10 mm, to remove lumps and other oversized material that can cause difficulty in pumping or prevent a proper flow of the grout in the preplaced aggregate mass. Coming to the equipment used to pump the grout, positive displacement type of pumps, such as piston, progressive cavity, or diaphragm, should be used. Centrifugal pumps should not be used, unless the mixer used is a high-speed colloidal type, which provides a rapid and low-pressure discharge. There should be a bypass at the pump outlet connecting the discharge with the pump hopper to permit a continuous pumping during the interruptions in grouting. Also there should be a pressure gauge on the grout line in the full view of the pump operator to indicate the grouting resistance and any possible line blockage. It is recommended to transfer the grout mixture through a single line directly from the pump to an injection pipe extending into the preplaced aggregate. Manifold systems, intended to supply two or more pipes simultaneously are not advisable, because the variation of grout flow from pipe to pipe may result in a non-uniform grout distribution within the coarse aggregate. On the other hand, when single line system is used, it may be a problem to provide a continuous grout flow while changing the connection from one pipe to another. However this may be solved by using a simple wye apparatus which has an inlet and two outlets, provided with valves at both ends. The inlet of the wye is connected directly to the pump with a hose, and the outlets to two adjacent pipes. While supplying one pipe, the valve at the other outlet of the wye is kept closed. When it is desired to convert the grout supply to the other pipe, first the closed valve at the wye outlet connected to this pump is opened, and immediately after, the other valve is closed. Then the outlet with closed valve is disconnected from the pipe already supplied, and connected to a new pipe adjacent to the one which is being supplied. By this way all the pipes can be supplied one by one without any need to stop the pump during the conversion of grout supply from one pipe to another. It is recommended to keep the length of the delivery line from the pump to the pipe as short as possible, and the line should have a sufficient diameter to maintain a grout velocity in the range of 0.6-1.2 m/s, because too low velocities may result in segregation or line blockage due to the stiffening of the grout, on the other hand too high velocities may result in wearing and will increase the energy consumption unnecessarily. As a common application, a high-pressure hose having a capacity of at least 2.8 MPa is used for the delivery line from the pump to the pipes, and again as a common application, for small works the inside diameter of the hose is chosen as 25 mm. However for long distances, upto 150 m, a larger diameter, in the range of 30-40 mm, is used. For longer distances, in the range of 150-300 m, a diamater of 50 mm is preferred. All pipe and hose connections should be completely watertight, because any loss of water from the mixture due to leakage will cause thickening of the grout and probably result in blockage at the point of leakage. The connections should also be of a quickly-disconnect type to facilitate rapid cleaning of pipes, because pipes should be cleaned out at 1 to 4 hr intervals, depending on the temperature and continuity of the operation. All the valves used in the system should be of a type which can be opened and closed easily and quickly, and they should provide an undisturbed grout flow when they are open. For this purpose, plug or ball valves are preferred, globe valves are not recommended.

Grouting can be done in two basic patterns: the horizontal layer and the advanced slope pattern. In the horizontal layer pattern, grout is injected into a pipe until it comes out from the next pipe which is 0.9-1.25 m above the injection point. Then grout is introduced into the next horizontally adjacent pipe which is 1.25-1.5 m away. A horizontal layer of coarse aggregate is grouted successively by repeating this procedure. Then the pipes are withdrawn until their lower ends remain at least 0.3 m below the grout surface, and the same procedure of grouting is repeated again. By this way, all of the aggregate is grouted layer by layer. If grouting is done through horizontal pipes located at the formwork, the grout is injected into a pipe until it comes out from second higher pipe above, and then grout is introduced into the next pipe just above the completed one, which is already below the grout surface.

The advanced slope pattern is used when the horizontal layer pattern is not practical, as in construction of thick slabs, where the plan dimensions are relatively large compared to the depth. In this procedure, injection is started into a pipe at one end of the form, and continued through that row of pipes until the grout appears at the surface or at least 0.3 m below the surface of the next row of pipes. The procedure is repeated until the entire slab is grouted. The reason why this procedure is called 'advanced slope' is that the grout surface takes a slope as the grout rises in the coarse aggregate mass [Figure 2.15].

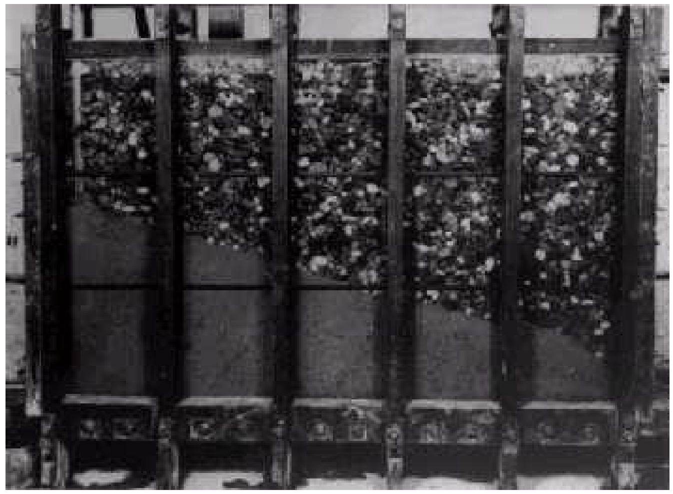

Figure 2.15. *The slope of the grout surface can be seen clearly as the grout raises within the coarse aggregate preplaced in a glass-faced form*

The slope of the grout surface depends on the consistency of grout, the nominal minimum size of coarse aggregate and the placement's being dry or underwater. For example, the natural slope of a grout with a time of efflux of 22 sec. [39] in a coarse aggregate with a nominal minimum size of 19 mm is 1:10 in an underwater placement. This slope may be as steep as 1:5 if it is a dry placement, other conditions remaining the same.

The rate of grout injection and the rate of grout rise within the preplaced aggregate should be under control. The injection rate is adjusted depending on the form design, aggregate grading and the grout fluidity. Normal injection rates through a given pipe vary from less than 0.03 m^3/min to over 0.11 m^3/min. On the other hand, the rate of grout rise is adjusted in such a way to avoid cascading of the grout. Normally a rate of grout rise less than 0.6 m/min. will assure against cascading. As the grout rises within a mass of preplaced aggregate, the location of the grout surface can be determined by observing seepage of grout from the small holes drilled on the forms. If the grouting is done through vertical pipes, sounding wells may be used for this purpose. The sounding line is usually equipped with a 25 mm diameter float, which is weighted in such a way that it sinks in water but floats on the grout. Even it is reported in the literature that an electronic system, replacing the sounding line, has been used to determine the location of the grout surface continuously on graphs at the pumping plan, during the construction of Honshu-Shikoku bridge piers in Japan. However the details of this electronic system are not available in the literature.

If a pipe has been idle for sometime, it is recommended to clean the pipe before continuing grout injection through that pipe. Cleaning of pipes should be done by rodding, it must not be done by water, especially when the lower end of the pipe is below the grout surface, otherwise the water-cementitious material ratio in the vicinity of the end of the pipe is increased undesirably. If pumping is stopped for longer than the time it takes the grout to harden, the pipes should be pulled just above the grout surface before the grout stiffens. To resume pumping, the pipes should be inserted back in the preplaced aggregate mass to the nearest contact with the hardened grout surface. In such cases, cold joints are formed within the preplaced coarse aggregate, which is undesirable. However sometimes joints in the same manner may be formed purposely to establish a good bond between the recently placed PAC and the new PAC which will be placed over it in the near future. Such joints are called construction joints, and they are formed by stopping the grout rise approximately 30 cm below the aggregate surface. However in this case it should be cared to prevent the collection of dirt and debris within the layer of exposed aggregate. Otherwise the bond of the new concrete with the existing one will be affected adversely. If a new layer of PAC is to be placed over an existing one which had been grouted upto the surface of coarse aggregate, then the construction joint can be obtained by water or sand-blasting the surface of the existing PAC after its grout has set but before it hardens appreciably, which provides a clean and rough surface for the grout in the next lift of PAC. Either in cold joints or in construction joints, since the coarse aggregate pieces cross the joint, shear strength is usually not affected; but if the grout of the PAC lying below bleeds excessively, the grout surface at the joint may be affected adversely, and this results in the weakening of tensile strength.

PAC method usually does not require any vibration operation, but sometimes low-frequency, high-amplitude external vibration of forms at or just below the grout surface helps in achieving excellent, smooth surface appearance, since it permits the grout to cover aggregate-form contacts better. However it should be kept in mind that, as in conventional concrete, excessive vibration will encourage bleeding, and usually causes sand-streaking as a result of the upward movement of bleeding water [2].

2.10.6. Finishing and Curing

When the rising grout surface within the preplaced coarse aggregate mass reaches the aggregate surface, the grout injection rate should be slowed down to avoid the lifting or dislodging of the aggregates at the surface. The tendency of the coarse aggregate at or near the surface to float on the rising grout can be restrained by holding a wire screen at the top of the aggregates. This screen is removed before finishing. If finishing with trowel is required, the grout should be let to flood the aggregate surface, then the excess grout should be removed, and a thin layer of 9-13 mm crushed aggregate is worked into the surface by raking and tamping. After the surface has stiffened sufficiently, it may be trowelled as required. Sometimes the surface of PAC is left 7.5-15 cm below the grade, and then finished with conventional concrete [9].

Curing of PAC should be done in the same manner as conventional concrete [50]. If pozzolans are used as a portion of the cementitious material content, curing time should be extended to improve the strength and durability [51].

2.11. Quality Control

In order to assure a quality work in PAC construction, first of all it should be determined that the contractor has had enough experience in making PAC. If not, at least he should demonstrate his capability by making some small test sections, and the performance of these sample specimens should be tested in a laboratory.

Secondly, the equipment to be used for mixing and pumping should be in good working condition. The outlet gates should be watertight to prevent the loss of batching water trough leakage. In addition the equipment should not require any break longer than 15-30 min. during the grouting process especially where cold joints must be avoided. Although a skilled operator can usually realize if there is an excessive rise in the pumping pressure, a pressure gage should be provided at the pump outlet for a continuous supervision of the pumping pressure through the pumping process, so that it can be interfered on time if something goes wrong.

Lastly, the materials to be used should meet the specified requirements, as in conventional concrete. The coarse aggregate should be checked frequently as it is being placed in the forms to assure that it is free of undersize particles and coatings, otherwise the use of dirty aggregate to which grout cannot bond will result in weakened concrete. Fine aggregate should be graded as specified in Table 2.1. Oversize particles can cause problems with the valving system of pump, moreover the void spaces in the preplaced aggregate to be filled by the grout can be clogged by these oversize particles. In fact such oversize particles can be eliminated by screening the grout while transferring from the mixer to the pump, but excessive quantities eliminated like this will not only lead to wasted material, but also cause a change in the mix design of the grout. In order to determine the acceptability of the other ingredients, it is recommended to prepare some trial grout mixtures prior to placing and test them for consistency, bleeding, expansion, and if time permits for strength. However it should be kept in mind that the strength of grout determined from testing cubes or cylinders can be used just as an indicator of the performance of the resulting PAC; otherwise it does not have a direct relationship with the strength of the PAC made with that grout, since the weakening effect of excessive bleeding of grout within the preplaced aggregate is not revealed [43]. Because of this, it is recommended to use PAC cylinders to obtain more representative results about the strength prior to placement of concrete [42]. However, of course the actual strength can be determined most accurately on the core specimens taken after the placement of concrete. The cores are taken in the same manner as in conventional concrete [52]. It has been shown that properly made PAC cylinder specimens can be as representative as the core specimens taken in place for the actual strength of the PAC, provided that they have been cast and kept at the same conditions as at the work site [Figure 2.16]

The accuracy of job-site batching of grout materials can be most easily checked by use of the flow cone [39]. Flow cone measurements should be made on successive batches of grout from each mixer until the desired fluidity in the allowable limits with only a tolerance of ± 2 sec. is achieved. After the desired fluidity has been achieved, generally it is enough to continue flow testing by random measurements at 5 to 10 batch intervals [Figure 2.17].

According to the results of these flow tests, if an adjustment of consistency is required, it should be tried to make this adjustment first of all by varying the amount of mixing water within the allowable water-cementitious material ratios. If the desired consistency still could not be achieved, then the cementitious material proportions should be readjusted.

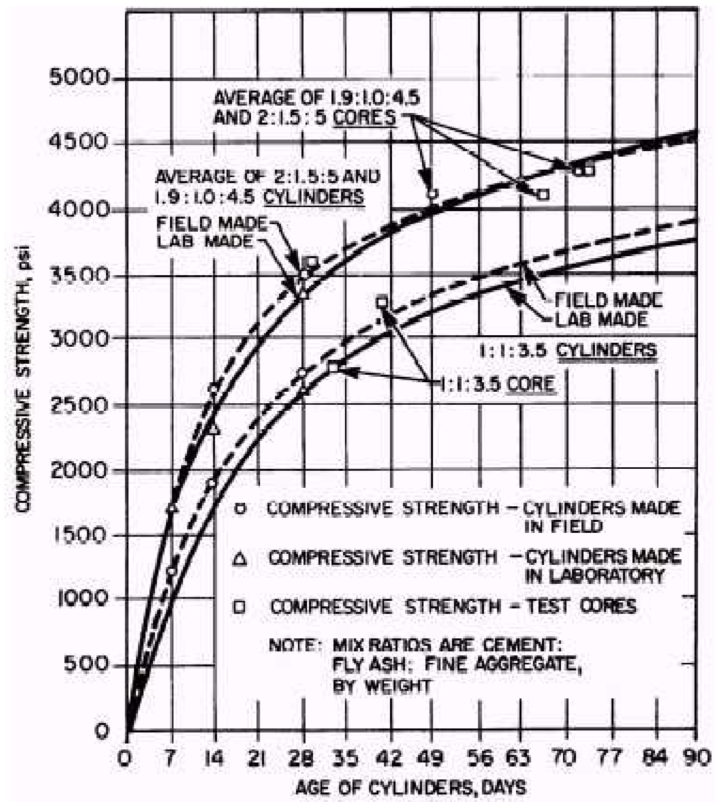

Figure 2.16. *Comparison of test results obtained from lab-made and field-made cylinder specimens with those obtained from the core specimens taken in place [53]*

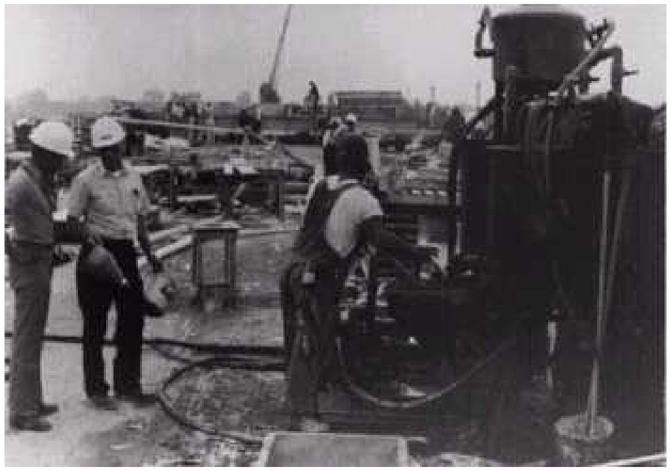

Figure 2.17. *As the mixing and pumping operations are continued by the equipment seen on the right, the inspector seen on the left holds a flow cone for the periodical checking of grout fluidity*

2.12. Economical Point of View

Although it depends on the case where it is used, whether PAC is more economical than conventional concrete or not, since in PAC almost 60% of the material is directly placed into the forms, and only 40% goes through mixing and pumping procedure, it is expected to be more economical in most cases. For example, in water structures which are accessible to self-unloading craft, and in land structures where the aggregate can be deposited by bulk handling equipment, PAC is more favorable. Since the coarse aggregate grading is not critical in PAC, except for the minimum particle size, it is more feasible to process the aggregate as it is being excavated, and place it in the forms immediately. For example, during the construction of deep mines in South Africa, forms were filled with hand selected rock from a nearby heading and the grout was pumped from 760 to 915 m through a 38 mm pipe, which was an economical solution compared to the elevator system used for normal mine operations.

Construction of bridge piers is another field of application where PAC method is more feasible, because it is very expensive to dewater the forms before the placement of concrete. Moreover water leakage into the forms during concrete placement, whether from the bottom or through the forms, will damage the concrete. However when PAC method is employed, most of the water in the forms is flooded upon the placement of coarse aggregate into the forms. Then, when the grout is pumped, any water leakage that does occur will be outward.

On the other hand, for column, beam, and surface repairs, the PAC method is usually more expensive than conventional concrete, since the formwork required for PAC is more qualified. Thus it is up to the owner of the job and the engineer to decide whether the bond, durability, or the other properties worth this extra cost or not.

In the case of large monolithic placements, the most critical factor that determines whether the PAC method is economically more advantageous or not is the location of the work with respect to the supply of concrete. In other words, if an adequate supply of conventional concrete is available at a reasonable distance, then the standard method of placement should be preferred. However if ready-mixed concrete is not available, the PAC method may be economically more favorable than constructing a concrete plant on site. Moreover, if heavy reinforcement is used, positioning the reinforcing bars on the coarse aggregate as it is placed may be more economical than supporting the bars in the empty forms [2].

It has been stated by the Oak Ridge National Laboratory that, in production of heavyweight concrete for nuclear biological shielding, from economical point of view conventional concrete should be preferred wherever there is enough space for placing low-slump concrete. However if embedded items require higher slump which may result in segregation, the PAC method should be considered [3].

In some situations there may be other factors than the cost that dictate the use of PAC method. For example, if the reinforcing bars are so closely spaced that vibrators could not be inserted, the PAC method can be the only alternative to using a higher slump concrete.

CHAPTER 3

EXPERIMENTAL STUDY

3.1. Experimental Program

In the scope of the study, twelve cubes of concrete specimens, each having a volume of 1 m^3, have been prepared. Eleven of these specimens have been prepared as PAC, and one of them as conventional concrete. Seven of the PAC specimens have been prepared by the conventional injection method, but four of them have been prepared by a different and newer approach, which will be explained later. The reason why the size of the specimens have been chosen as so big for laboratory conditions is that, the temperature measurements made by thermocouples on small concrete specimens do not give representative results for mass concrete studies; because when the concrete section is small, the central and surface temperatures can easily get balanced and significant temperature differences can not be observed as in mass concrete.

During the preparation of the specimens, four thermocouples have been located in each specimen, two of them at the surface and two of them at the centre of the specimen, as sketched at Figure 3.1.

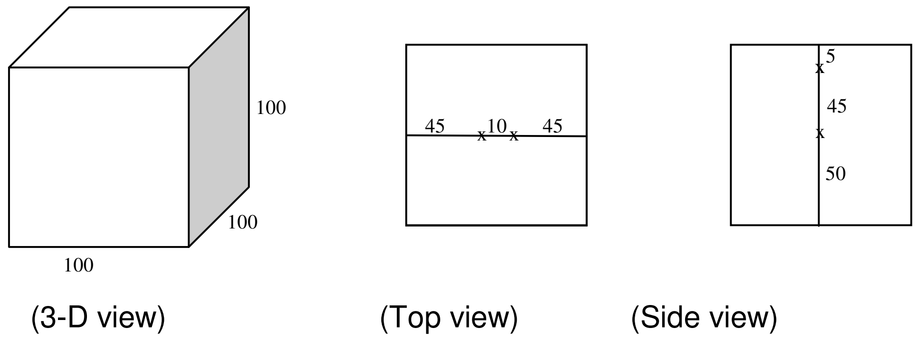

Figure 3.1. *The location of the thermocouples in the concrete cube specimens (the dimensions are given in cm)*

The reason of using two thermocouples at the same location of the specimen was both to insure the continuity of temperature measurement in case one of the thermocouples gets out of order, and to check if the thermocouples are working properly or not by providing a comparison between the outputs of two thermocouples.

After the preparation of a specimen had been finished, the surface and the central temperatures of the specimen has been followed by taking periodical measurements from these thermocouples till the temperatures reach a steady-state.

Since it was not possible to carry such big specimens to the curing room of the laboratory, a special tent was built for these specimens, and the specimens were kept in this tent till the formworks are removed [Figure 3.2]. The formworks were removed after the process of following concrete temperatures had been ended up with the reaching of temperature values to a steady-state, which took approximately one to two weeks depending on the type of specimen.

The ambient temperature and the relative humidity inside the tent were monitored by a digital equipment, and they were tried to be kept as constant as possible for the sake of the comparison of test results between concrete specimens which were cast at different times; because, maximum two specimens could have been cast at once because of the limitation of the laboratory conditions. The ambient temperature was kept around $20 \pm 2\ ^{0}C$ and the relative humidity was kept around 70 ± 20 %.

Figure 3.2. *The tent which was used as a portable curing room for the huge concrete specimens*

After the process of following temperature change had been finished and the forms had been removed, the next step was the following of strength development by taking core specimens and testing them under compression at certain ages as 28 days, 90 days, 6 months and 1 year. At each age, three core specimens were taken and the average of their results was noted down as the strength of the specimen at that age. After the first core specimen was crushed under compression, the modulus of elasticity was also determined on the remaining two specimens before they are crushed, and the average of these two modulus of elasticity values obtained from the core specimens was noted down as the modulus of elasticity of that concrete specimen at that age. The three core specimens were taken from different depths of the concrete volume in order to check the uniformity of concrete by comparing the results of individual core specimens before taking the average of them. Taking the top surface of the concrete cube as reference, one of the core specimens were taken from the upper portion which is 5-35 cm deep, one from the mid portion which is 35-65 cm deep, and one from the bottom portion which is 65-95 cm deep.

Also the Schmidt Hammer test and the ultrasonic pulse velocity test have been carried out on the big concrete specimens, and the results of these non-destructive tests have been compared with the compressive strength test results obtained from the core specimens. While carrying out these non-destructive tests, in order to check the uniformity of concrete also, again as in done while taking the cores, the data were obtained from different depths of the specimen; and before taking the average of these data, it has been examined whether the data taken from different portions of the specimen are consistent with each other or not. While checking the uniformity of the concrete specimens along the depth by non-destructive tests, the specimens have been examined in three portions again as done while taking the cores, but differently here, the horizontal uniformity within these portions have also been checked by taking extra data. However since taking core specimens is much more difficult and time consuming compared to taking data by non-destructive tests, it was not possible to test the horizontal uniformity by taking extra cores from the same portion of the specimen. In fact, this has been partially done by taking the cores from horizontally different locations at each portion, as sketched at Figure 3.3. A photo taken while taking one of the three core specimens, which have been located this way, is given at Figure 3.4.

While carrying out the Schmidt Hammer test, ten data have been taken from each selected portion. On the other hand, three readings per portion have been considered to be enough in ultrasonic pulse velocity test.

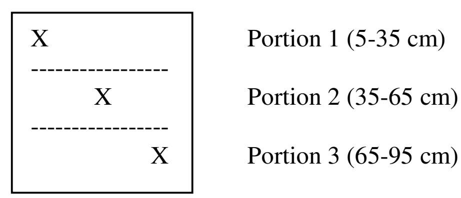

Figure 3.3. *Selection of the locations of the three core specimens in order to check the horizontal and the vertical uniformity of the concrete simultaneously*
(While giving the depths of the portions, the top surface of the specimen is taken as datum)

Figure 3.4. *One of the three core specimens, which have been located as shown at Figure 3.3, is being taken (the one that belongs to portion 2 according to Figure 3.3)*

In order to prevent any confusion, a designation is used to identify each of the twelve concrete cube specimens. The designations used for each specimen and the corresponding descriptions are given at Table 3.1.

As it can be easily understood from Table 3.1, while designating the PAC specimens, the superscript indicates the method by which the specimen has been produced, and the subscript indicates the specimen no. The superscript 1 indicates that the PAC specimen has been produced by the conventional injection method, and the superscript 2 indicates that it has been produced by the new method, the details of which are given at Section 3.2.

Table 3.1. *The Designations Used for the Concrete Specimens and Their Descriptions*

Specimen No.	Designation	Description
1	CC	Conventional Concrete specimen
2	PAC_1^1	PAC specimens produced by the conventional injection method
3	PAC_2^1	
4	PAC_3^1	
5	PAC_4^1	
6	PAC_5^1	
7	PAC_6^1	
8	PAC_7^1	
9	PAC_1^2	PAC specimens produced by the new method
10	PAC_2^2	
11	PAC_3^2	
12	PAC_4^2	

3.2. A New Approach in Making PAC

In the conventional method of making PAC, first the coarse aggregate is placed into the forms [Figure 3.5], and then mortar is injected through the pipes inserted in the coarse aggregate mass by special pumps, till the injected grout rises upto the top surface of the preplaced coarse aggregate mass [Figure 3.6].

Figure 3.5. *The preplacing of coarse aggregates into the formwork*

Figure 3.6. *The grout is pumped through the insert pipes till it rises upto the top surface of the preplaced coarse aggregate mass*

Despite the many advantages of making concrete by this way comparing to conventional concrete in many situations, it is obvious that this method requires special equipment and experienced workmanship. On the other hand, self-compacting concrete (SCC) is one of the newest and most popular technique used in placing concrete, especially where the closely-spaced reinforcement does not allow compaction of the concrete by vibration. In SCC, the fluidity of concrete is increased by the help of new generation superplasticizers so much that concrete fills all the voids in the formwork without requiring any injection or vibration. In this research, the same idea has been applied in making PAC; such that, by utilizing high-range water reducing admixtures, the fluidity of the mortar has been increased to a level at which it could fill the voids itself when it is poured over the preplaced coarse aggregate [Figure 3.7]. Thus, this method can be named as making PAC by SCM (self-compacting mortar).

Figure 3.7. *The self-compacting mortar is just poured over the preplaced coarse aggregate without applying any injection*

This way of making PAC has been found to be much more advantageous compared to the conventional injection method, since it does not require any special equipment, such as pumps and insert pipes, or any special workmanship. The mortar can be prepared by a simple conventional concrete drum mixer, and it can be poured over the preplaced coarse aggregate by ordinary workmen. The only question about this method can be that, how can one be sure whether all the voids have been filled or not? This can be simply checked by the amount of mortar consumed; because, if the volume of voids within the preplaced coarse aggregate mass is known, the amount of mortar required to fill these voids can be determined easily before starting the operation. The volume of voids within a preplaced coarse aggregate can easily be determined by filling a container of known volume with that coarse aggregate gradation, and then adding water to the container till it overflows. In such a test, it is obvious that the volume of the water consumed is equal to the volume of voids within the aggregate mass; and proportioning this volume to the volume of the container used for the test, the actual volume of voids can easily be calculated by using the volume surrounded by the formwork.

In this research, it has been observed that the amount of mortar required to fill all the voids within the preplaced coarse aggregate mass, which had been determined before starting to place the mortar, was totally consumed when the mortar appeared at the top of the aggregate mass, which means that all the voids within the preplaced coarse aggregate mass could have been filled successfully by this method. Moreover the success of the method has also been verified by the uniformity checks done while taking core specimens and while taking data by non-destructive tests.

3.3. Equipment

Since there is no local company which gives service in the field of PAC projects, it has not been possible to provide a machinery specialized for making PAC. Instead, a grouting machine which is used by a foundation engineering company was brought to the Materials of Construction Laboratory, and the PAC[1] specimens have been produced by using this machine. Since handling of such a machine requires experienced workmanship, some workmen were sent by the company to help the laboratory staff during the preparation of those specimens [Figure 3.8]. The machine was working both as a mixer [Figure 3.9], and a pump [Figure 3.10]. There was a pressure gauge provided at the outlet of the machine in order to warn the operator if there is any congestion in the insert pipes [Figure 3.11]. However since this machine was made to be used for grouting operations in foundation engineering, it was not capable of pumping grout mixtures which include sand. In other words, it was designed to pump grout mixtures which include cementitious material and water only. Because of this fine aggregates could not have been used in the grouts for the PAC[1] specimens which are made by injection method. However this situation has provided another comparison criterion, such as PAC with and without fine aggregates, because fine aggregates could have been used without any problem in the PAC[2] specimens which are made by the other method which has already been explained at Section 3.2.

Figure 3.8. *The workmen sent by the company, who are experienced in using the grouting machine, to help the laboratory staff during the preparation of PAC*[1] *specimens*

Figure 3.9. *The part of the machine which works as mixer*

Figure 3.10. *The outlet of the pipe which serves as a pump*

Figure 3.11. *The pressure gauge provided on the machine*

As insert pipes, ordinary electrical insulation pipes which are obtained from the market have been used. Since the outlet diameter of the pump is 15.5 mm, the diameter of the insert pipes have been chosen as 16 mm. A pipe clip has been used at the connection point of the pump outlet and the inlet of the insert pipes in order to prevent any seepage of the grout [Figure 3.12]. Since the formworks are 1 m deep, the length of the insert pipes have been chosen as 1.2 m in order to provide one end of the pipes remaining 20-30 cm above the surface of the aggregate mass, while the other end extending through the bottom of the formwork [Figure 3.13]. A special attention was paid to prevent the touching of the outlet of insert pipes with the formwork base in order to avoid any congestion during pumping of the grout. For this purpose, the insert pipes have been located in the coarse aggregate mass in such a way that their outlet stayed maximum 5 cm close to the bottom of the formwork. Also the insert pipes have been pierced at a point which is 10 cm above the outlet end in order to provide another outlet point incase the main outlet can be congested by an aggregate in the formwork or by a lump in the grout.

Figure 3.12. *The connection of the pump outlet and the insert pipe*

Figure 3.13. *The length of the insert pipes were chosen in such a way that an extension of 20-30 cm was obtained above the surface of the aggregate mass*

Five insert pipes have been used for each formwork; four at the corners and one at the centre of the cube. The ones at the corners are located at a point which is 10 cm away from the corners of the formwork. The pipe pattern is sketched at Figure 3.14, and a photo taken while the insert pipes are being located in the formwork during the placement of aggregates is given at Figure 3.15. While grouting, these five insert pipes were pumped one by one uniformly in order to provide a uniform distribution of the injected grout within the preplaced coarse aggregate mass [Figure 3.16].

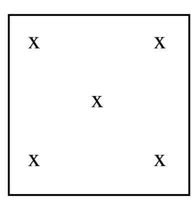

Figure 3.14. *The pattern of the insert pipes from top view*

Figure 3.15. *Locating of the insert pipes during the placement of aggregates*

Figure 3.16. *The alternating pumping of the five insert pipes*

As the equipment used for preparation of the PAC[2] specimens, only a simple conventional concrete drum mixer with 0.2 m^3 capacity has been used to prepare the mortar to be poured over the preplaced aggregate mass. Other than this, no special equipment is required for this method, as it has been mentioned while explaining the details of the method at Section 3.2.

3.4. Formwork

Two modular formworks which are made of timbers each 5 cm thick and 10 cm wide have been used. The formworks are in cubic shape and their inner volume is exactly 1 m^3. The inner faces of the formworks have been coated by a thick nylon before the placement of coarse aggregate in order to prevent any leakage of the grout from the connections of the timbers. This nylon also served as a sealing curing till the formworks are removed. After setting, the concrete specimen could easily be taken out of the formwork by removing just one face of the cubic formwork; and then the formwork could become ready for another use by reassembling this removed face [Figure 3.17].

Figure 3.17. *The specimens could be taken out easily by removing only one face of the modular formwork*

3.5. Curing

Curing can roughly be defined as the supply of heat and moisture till the concrete reaches the desired strength. In this study, since the thick nylon, which is used for coating the inner faces of the formwork in order to prevent the seepage of the grout during pumping, also served as a sealing curing by preventing the loss of moisture from the specimens until the removal of the formwork, it was adequate to apply wet covering only on the top surface of the specimens, as supply of moisture.

Coming to the supply of heat, as it is mentioned earlier, the specimens were kept in a special tent till the removal of formwork. A heater was put in this tent to keep the inner temperature as high as possible. In addition to this, the inside of the tent was sprinkled periodically in order to keep the relative humidity as high as possible. The inner temperature and the relative humidity of the tent have been monitored by a digital equipment during the curing period. It was observed that the room temperature could have been kept around $20 \pm 2\ ^{0}C$ and the relative humidity could have been kept around 70 ± 20 %.

After the process of following concrete temperatures using the thermocouples located in the specimens was ended up with the reaching of concrete temperatures to a steady-state, the specimens were taken out of the tent, the formworks were removed and the specimens were left to air-curing.

3.6. Tests Performed

All the tests have been carried out in the Materials of Construction Laboratory of the Civil Engineering Department of Middle East Technical University, and in accordance with the related ASTM Standards [54].

3.6.1. Tests on Aggregates

Tests Performed	Relevant Standards
Specific Gravity and Absorption	ASTM C 127, C 128
Sieve Analysis	ASTM C 136

3.6.2. Tests on Cementitious Materials

Tests Performed	Relevant Standards
Chemical Analysis	ASTM C 114
Density	ASTM C 188
Fineness	ASTM C 204
Setting Time	ASTM C 191
Normal Consistency	ASTM C 187
Heat of Hydration	ASTM C 186
45 μm Sieve Residue	ASTM C 430
Compressive Strength	ASTM C 109
Strength Activity	ASTM C 618
Slag Activity	ASTM C 989

3.6.3. Tests on Concrete

Tests Performed	Relevant Standards
Core Testing	ASTM C 42
Modulus of Elasticity	ASTM C 469
Schmidt Hammer Test	ASTM C 805
Ultrasonic Pulse Velocity Test	ASTM C 597

3.7. Materials Used

3.7.1. Aggregates

The geological origin of the aggregates used consists of 50% monzonite-monzodiorite and 50% meta-andesite.

3.7.1.1. Coarse Aggregates

Three classes of coarse aggregate, as 7-15, 15-30, and 30-60 mm, have been used, which were brought to the laboratory in the scope of a consulting study for the Deriner Dam Project [55]. The specific gravity and absorption capacity values of these aggregates are given at Table 3.2.

Table 3.2. *Specific Gravity and Absorption Capacity Values of the Coarse Aggregates Used*

Aggregate Class	**Specific Gravity**		**Absorption Capacity (%)**
	Dry	**SSD**	
7-15	2.66	2.68	0.79
15-30	2.66	2.69	1.02
30-60	2.70	2.71	0.60

The sieve analysis results of these three aggregate classes are given at Table 3.3.

Table 3.3. *Sieve Analysis of the Coarse Aggregate Classes Used*

Sieve No.	**% PASSING**		
	{7-15}	**{15-30}**	**{30-60}**
2 ½”	100	100	100
2”	100	100	69.26
1 ½”	100	100	14.85
1”	100	93.40	0
3/4”	100	70.42	0
1/2”	100	19.37	0
3/8”	85.41	1.54	0
#4	28.94	0	0

The gradation of coarse aggregate used for all PAC[1] specimens was kept identical by mixing these three aggregate classes at the same amounts for each specimen. The amounts at which these three aggregate classes have been mixed, and the resulting aggregate gradation are given at Table 3.4 and Table 3.5 respectively.

Table 3.4. *The Proportions of the Coarse Aggregate of PAC[1] Specimens*

Aggregate	Amount (kg/ m^3)	Proportion (%)
7-15	752	47.24
15-30	416	26.13
30-60	424	26.63
Total	1592	100

Table 3.5. *Gradation of the Coarse Aggregate Used for PAC[1] Specimens*

Sieve No.	% Passing
2 ½”	100
2”	91.81
1 ½”	77.33
1”	71.65
3/4”	65.64
1/2”	52.30
3/8”	40.75
#4	13.67

The aggregates at this gradation have a void content of 40% of the volume occupied by it in uncompacted state, which means that, when a container of 1 m^3 capacity is filled with these aggregate without any compaction, a volume of 0.4 m^3 will be left as voids within the aggregate mass; and this means that, if it is desired to make PAC by injecting grout into this aggregate mass, the amount of grout required to fill all the voids will be 0.4 m^3. Knowing the specific gravities of all the materials intended to be used in making the grout mixture, and defining their proportions in the total mixture, the required weights of each material can easily be calculated from this total volume of the grout. The required amounts of cementitious materials, water and admixtures for PAC^1 specimens have been determined by this way; and the consumption of all the pre-determined amount of materials have been used as a check of the success of the operation, from the point of leaving no voids being unfilled with grout.

The coarse aggregate of the CC specimen has also been composed of the three aggregate classes at hand. The amounts of each class by weight were a bit different than those used for PAC^1 specimens, but the percentages of each aggregate class in the total amount of coarse aggregate used were the same as the values given at Table 3.4. This can be seen better when Table 3.4 is compared with Table 3.6, where the composition of the coarse aggregate used for CC specimen is given.

Table 3.6. *The Proportions of the Coarse Aggregate of CC Specimen*

Aggregate	Amount (kg/ m^3)	Proportion (%)
7-15	940	47.24
15-30	520	26.13
30-60	530	26.63
Total	1990	100

The coarse aggregate of PAC^{2} specimens have consisted of only one aggregate class, which is {30-60}; because it has been desired to attain the largest volume of void content, since no pressure was used while placing the grout for these specimens, and the maximum void content could have been obtained by using such a coarse aggregate gradation. The aggregate composed of {30-60} only, the gradation of which have already been given at Table 3.3, leaves a void amount of 50% of the volume occupied by it in uncompacted state. In other words, when the formwork with 1 m^{3} capacity is filled with this aggregate without any compaction, 1592 kg of aggregate is consumed, and a void content of 0.5 m^{3} is left within the aggregate mass. Thus a greater amount of grout is required comparing to the grout amount consumed for the preparation of PAC^{1} specimens, but since there is no technical obstacle for using of sand in this method of making PAC, as it was faced during the preparation of PAC^{1} specimens, there is no economical disadvantage from the requirement of cementitious material point of view.

3.7.1.2. Fine Aggregate

As it is mentioned above, since the grouting machine used for the preparation of the PAC^{1} specimens was not capable of pumping grout mixtures which contain sand, no fine aggregate has been used in these specimens. From comparison point of view, in order to provide similar conditions, as it has been done while arranging the proportions of the coarse aggregate, no fine aggregate has been used during the preparation of the CC specimen either. Thus fine aggregates have been used only for PAC^{2} specimens.

There were two classes of fine aggregates in hand, which are geologically from the same origin as the coarse aggregates used, and which were again brought to the laboratory in the scope of Deriner Dam Project [55]. Both of the two classes of fine aggregates were being sold in the market under the same name of class, which is {0-3}, but one of them was comparatively finer. The sieve analysis results of these aggregates are given at Table 3.7, and the other data about these aggregates are given at Table 3.8.

Table 3.7. *Sieve Analysis of the Fine Aggregates Used*

Sieve No.	% PASSING	
	$\{0\text{-}3\}_1$	**$\{0\text{-}3\}_2$**
No.4	99.83	99.78
No.8	95.42	93.94
No.16	59.74	60.60
No.30	30.50	36.70
No.50	11.29	21.67
No.100	3.05	13.32

Table 3.8. *Specific Gravity and Absorption Capacity Values of the Fine Aggregates Used*

Aggregate Class	Specific Gravity		Absorption Capacity (%)
	Dry	**SSD**	
$\{0\text{-}3\}_1$	2.47	2.57	3.7
$\{0\text{-}3\}_2$	2.59	2.61	0.98

Half of the total amount of fine aggregates used for one specimen belonged to one of these two fine aggregate classes, and the other half belonged to the other class. Thus the gradation of the resulting fine aggregate used for PAC^2 specimens was as given at Table 3.9.

Table 3.9. *Gradation of the Fine Aggregate Used for* PAC^2 *specimens*

Sieve No.	% Passing
No.4	99.81
No.8	94.68
No.16	60.17
No.30	33.60
No.50	16.48
No.100	8.19

3.7.2. Cementitious Materials

3.7.2.1. Portland Cement

Only one type of Portland Cement which is obtained from Baştaş Cement Factory have been used, and it is classified as PÇ 42.5 according to Turkish Standards (similar to Type I - Ordinary Portland Cement according to ASTM classification). Chemical analysis and the physical properties of this cement are given at Table 3.10 and Table 3.11 respectively.

Table 3.10. *Chemical Analysis of the Portland Cement Used*

Oxides and Other Determinations	% by weight	ASTM Limits
CaO	62.56	-
SiO_2	20.47	-
Al_2O_3	5.68	-
Fe_2O_3	3.08	-
MgO	1.80	max. 6.0%
SO_3	3.22	max. 3.0%
K_2O	0.95	-
Na_2O	0.30	-
Free CaO	0.98	max. 3.0%
Cl^-	0.014	max. 0.1%
LOI	2.49	max. 3.0%
IR	0.47	max. 0.75%

Table 3.11. *Physical Properties of the Portland Cement Used*

Property		Value	ASTM Limits
Specific Gravity		3.12	-
Blaine Fineness (cm^2/g)		3915	min. 2800
W/C for NC		0.26	-
Setting Time (min)	Initial	108	min. 45
	Final	162	max. 375
Compressive Strength (MPa)	3 days	27.6	min. 12.4
	7 days	39.0	min. 19.3
	28 days	47.7	-
Heat of Hydration (cal/gr)	3 days	67.5	-
	7 days	74.9	-
	28 days	92.1	-

3.7.2.2. Blended Cements

Two types of Blended Cements have been used, one from Aşkale and the other from Ünye Cement Factory, which were brought to the laboratory in the scope of Deriner Project, like the aggregates used [55]. Both of them are classified as PKÇ B 32.5 according to Turkish Standards (similar to Portland-Pozzolan Cements according to ASTM classification), and both of them contain about 35% (by mass) pozzolanic addition, according to the producer's decleration. Chemical analysis and the physical properties of these cements are given in the following tables.

Table 3.12. *Chemical Analysis of the two Blended Cements Used*

Oxides and Other Determinations	**% by weight**		**ASTM Limits**
	PKÇ$_1$	**PKÇ$_2$**	
CaO	41.93	44.40	-
SiO_2	31.31	33.64	-
Al_2O_3	8.94	9.07	-
Fe_2O_3	4.19	4.21	-
MgO	3.23	1.60	max. 6.0%
SO_3	2.09	1.72	max. 3.0%
LOI	5.46	3.15	max. 3.0%
IR	24.32	26.42	max. 0.75%

Table 3.13. *Physical Properties of the two Blended Cements Used*

<table>
<tr><th colspan="2">Property</th><th>PKÇ $_1$</th><th>PKÇ $_2$</th></tr>
<tr><td colspan="2">Specific Gravity</td><td>2.78</td><td>2.86</td></tr>
<tr><td colspan="2">Blaine Fineness (cm^2/g)</td><td>4801</td><td>5064</td></tr>
<tr><td colspan="2">W/C for NC</td><td>0.27</td><td>0.38</td></tr>
<tr><td rowspan="2">Setting Time (min)</td><td>initial</td><td>110</td><td>110</td></tr>
<tr><td>final</td><td>165</td><td>200</td></tr>
<tr><td rowspan="4">Compressive Strength (MPa)</td><td>3 days</td><td>15.4</td><td>20.8</td></tr>
<tr><td>7 days</td><td>19.6</td><td>28.7</td></tr>
<tr><td>28 days</td><td>34.1</td><td>35.4</td></tr>
<tr><td>90 days</td><td>38.8</td><td>44.8</td></tr>
<tr><td rowspan="4">Heat of Hydration (cal/gr)</td><td>3 days</td><td>45.9</td><td>71.7</td></tr>
<tr><td>7 days</td><td>49.0</td><td>86.8</td></tr>
<tr><td>28 days</td><td>74.0</td><td>97.6</td></tr>
<tr><td>90 days</td><td>83.1</td><td>109.5</td></tr>
</table>

3.7.2.3. Fly Ash

A local fly ash which is obtained from Tunçbilek thermal power plant has been used in this research (similar to Class F fly ash according to ASTM classification). Chemical analysis and the physical properties of this fly ash are given at Table 3.14 and Table 3.15 respectively.

Table 3.14. *Chemical Analysis of the Fly Ash Used*

Oxides and Other Determinations	% by weight	ASTM Limits
CaO	3.34	-
SiO_2	58.44	-
Al_2O_3	18.79	-
Fe_2O_3	10.60	-
MgO	4.52	-
SO_3	1.75	max. 5.0%
K_2O	1.86	-
Na_2O	0.22	max. 1.5%
Cl^-	-	-
P_2O_5	0.25	-
TiO_2	-	-
Mn_2O_3	0.22	-
LOI	0.77	max. 6.0%
IR	86.72	-

Table 3.15. *Physical Properties of the Fly Ash Used*

Property		Value	ASTM Limits
Specific Gravity		2.10	-
Blaine Fineness (cm^2/g)		2890	-
W/C for NC		0.53	-
45 µm sieve residue (%)		27.3	max. 34%
Strength Activity Index (%)	7 days	89.1	min. 75%
	28days	82.0	min. 75%

3.7.2.4. Ground Granulated Blast Furnace Slag (GGBFS)

A local GGBFS which is obtained from İskenderun Iron Factory has been used in this research. Chemical analysis and some physical properties of this GGBFS are given at Table 3.16 and Table 3.17 respectively.

Table 3.16. *Chemical Analysis of the GGBFS Used*

Oxides and Other Determinations	% by weight	ASTM Limits
CaO	35.48	-
SiO_2	36.88	-
Al_2O_3	15.20	-
Fe_2O_3	0.70	-
MgO	9.95	-
SO_3	0.23	max. 4.0%
LOI	0.49	-
IR	0.69	-

Table 3.17. *Physical Properties of the GGBFS Used*

Property		Value	ASTM Limits
Specific Gravity		2.83	-
Blaine Fineness (cm^2/g)		4685	-
45 µm sieve residue (%)		0	max. 20%
Slag Activity Index (%)	7 days	64	min. 75%
	28days	103	min. 75%

3.7.2.5. Rock Powder

The rock powder, which is obtained by grinding the rocks that are geologically from the same origin as the aggregates used, and which is brought to the laboratory again in the scope of Deriner Dam Project, has also been used as a mineral admixture in this research [55], and it has showed a very good performance not only as a pozzolan but also as a filler. Chemical analysis and some physical properties of this rock powder are given at Table 3.18 and 3.19 respectively.

Table 3.18. *Chemical Analysis of the Rock Powder Used*

Oxides and Other Determinations	% by weight	ASTM Limits
CaO	6.74	-
SiO_2	60.16	-
Al_2O_3	14.64	-
Fe_2O_3	4.64	-
MgO	3.12	-
SO_3	0.11	max. 4.0%
LOI	5.68	max. 10.0%
IR	83.34	-

Table 3.19. *Physical Properties of the Rock Powder Used*

<table>
<tr><th colspan="2">Property</th><th>Value</th><th>ASTM Limits</th></tr>
<tr><td colspan="2">Specific Gravity</td><td>2.86</td><td>-</td></tr>
<tr><td colspan="2">Blaine Fineness (cm^2/g)</td><td>4300</td><td>-</td></tr>
<tr><td colspan="2">45 µm sieve residue (%)</td><td>23.8</td><td>max. 34%</td></tr>
<tr><td rowspan="2">Strength Activity Index (%)</td><td>7 days</td><td>91.6</td><td>min. 75%</td></tr>
<tr><td>28days</td><td>91.1</td><td>min. 75%</td></tr>
</table>

3.7.2.6. Brick Powder

The brick powder obtained from a factory in Manisa, which collects the broken bricks from brick factories and sell them after grinding to be used in racecources, have been used as a mineral admixture in this research. It is well known that pieces of clay bricks were widely used in Roman and Byzantine structures to obtain hydraulic mortars. Crushed and/or powdered bricks were utilized for both load bearing and water proofing purposes [56-58]. Recent studies have shown that ground clay brick when added to cement in certain amounts increase the resistance of mortar against some chemical attacks [59-61]. Chemical analysis and physical properties of the brick powder used are given at Table 3.20 and Table 3.21 respectively.

Table 3.20. *Chemical Analysis of the Brick Powder Used*

Oxides and Other Determinations	% by weight	ASTM Limits
CaO	3.94	-
SiO_2	62.7	-
Al_2O_3	17.1	-
Fe_2O_3	6.84	-
MgO	2.25	-
SO_3	0.84	max. 4.0%
LOI	2.67	max. 10.0%
IR	84.45	-

Table 3.21. *Physical Properties of the Brick Powder Used*

Property		Value	ASTM Limits
Specific Gravity		2.64	-
Blaine Fineness (cm^2/g)		4000	-
45 μm Sieve Residue (%)		37	max. 34%
Strength Activity Index (%)	7 days	66.2	min. 75%
	28days	80.3	min. 75%

3.7.3. Chemical Admixtures

Two types of superplasticizers, one of which is obtained from Sika and the other from a local company named Konsan, and one type of air-entraining admixture, which is obtained again from Sika, have been used as chemical admixtures in this research.

3.7.4. Mixing Water

The METU Campus water, which is drinkable and which does not contain any material at excessive amounts that can be harmful for the concrete, has been used as the mixing water.

3.8. Mix Design of the CC Specimen

In the preparation of CC specimen, $PKÇ_1$ has been used as cement and rock powder has been used as mineral admixture. Water-cementitious material ratio has been arranged as 0.45. As superplasticizer, the one produced by Sika has been preferred, and it has been used at an amount of 1% by weight of the total cementitious materials content (i.e. $PKÇ_1$ + rock powder). The amount of air-entraining admixture has been arranged as 0.2%, again by weight of the total cementitious materials content. Both the superplasticizer and the air-entraining admixture have been introduced into the mixture within the mixing water. The amounts of all the materials used for the preparation of CC specimen are given as kg/m^3 at Table 3.22. By the way, since the specimen itself is also 1 m^3 in volume, these values are directly equal to the amounts of materials consumed.

Table 3.22. *Mixture Proportions of the CC Specimen*

Material Used		**Amount (kg/m^3)**
Blended Cement ($PKÇ_1$)		200
Rock Powder		50
Water		112
Superplasticizer (Sika)		2.5
Air-entraining admixture		0.5
Coarse Aggregate	7-15 mm	940
	15-30 mm	520
	30-60 mm	530
Expected unit weight		**2355 kg/m^3**

3.9. Mixture Proportions of the Grouts for PAC Specimens

3.9.1. Grouts for PAC^1 Specimens

As it is mentioned above, no fine aggregate have been used in the preparation of the grouts for PAC^1 specimens because of technical reasons. Thus the grout mixtures of PAC^1 specimens have consisted of only cementitious materials and mixing water with the chemical admixtures in it. The compositions of the preplaced coarse aggregate used for PAC^1 specimens, which are all identical, have already been given at Table 3.4. From Table 3.23 to Table 3.29, mixture proportions of the grouts used for the seven PAC^1 specimens are given as kg per m^3 of concrete.

Table 3.23. *Mixture Proportions of the Grout Used for Specimen* PAC_1^1

Material Used	Amount (kg/m^3)
Blended Cement ($PKÇ_1$)	388
Rock Powder	97
Water	218
Superplasticizer (Sika)	4.85
Air-entraining admixture	0.97

Table 3.24. *Mixture Proportions of the Grout Used for Specimen* PAC_2^1

Material Used	Amount (kg/m^3)
Blended Cement (PKÇ$_1$)	243
Rock Powder	243
Water	219
Superplasticizer (Sika)	4.86
Air-entraining admixture	0.972

Table 3.25. *Mixture Proportions of the Grout Used for Specimen* PAC_3^1

Material Used	Amount (kg/m^3)
Blended Cement (PKÇ$_1$)	236
Rock Powder	118
Fly Ash	118
Water	212
Superplasticizer (Sika)	4.72
Air-entraining admixture	0.944

Table 3.26. *Mixture Proportions of the Grout Used for Specimen* PAC_4^1

Material Used	Amount (kg/m^3)
Blended Cement (PKÇ$_2$)	244
GGBFS	244
Water	220
Superplasticizer (Konsan)	6.10
Air-entraining admixture	0.976

Table 3.27. *Mixture Proportions of the Grout Used for Specimen* PAC_5^1

Material Used	Amount (kg/m^3)
Blended Cement (PKÇ$_2$)	227
Fly Ash	227
Water	204
Superplasticizer (Konsan)	5.68
Air-entraining admixture	0.908

Table 3.28. *Mixture Proportions of the Grout Used for Specimen* PAC_6^1

Material Used	Amount (kg/m^3)
Blended Cement (PKÇ$_2$)	240
Brick Powder	240
Water	216
Superplasticizer (Konsan)	4.80
Air-entraining admixture	0.960

Table 3.29. *Mixture Proportions of the Grout Used for Specimen* PAC_7^1

Material Used	Amount (kg/m^3)
Blended Cement (PKÇ$_1$)	232
Brick Powder	116
Fly Ash	116
Water	209
Superplasticizer (Sika)	4.64
Air-entraining admixture	0.928

3.9.2. Grouts for PAC^2 Specimens

It has been already mentioned above that the preplaced coarse aggregate used for PAC^2 specimens have consisted of a single aggregate class which is 30-60 mm. 1420 kg coarse aggregate has been used for each PAC^2 specimen, and since no machinery has been used in placing the grouts of these specimens, fine aggregate could have been used during the preparation of grout mixtures. The two classes of fine aggregates at hand have been used at equal amounts for each specimen, and this amount has been arranged according to the total amount of cementitious materials used for that specimen; thus it has changed from specimen to specimen. The prescriptions of the grout mixtures used for each of the four PAC^2 specimens are given in the following tables. The amounts are again given as kg per m^3 of concrete. Since each of the concrete specimens is 1 m^3, these amounts are also equal to the exact amounts of materials consumed for the preparation of each specimen.

Table 3.30. *Mixture Proportions of the Grout Used for Specimen PAC_1^2*

Material Used	**Amount (kg/ m^3)**
Portland Cement (PÇ 42.5)	306
$\{0\text{-}3\}_1$	306
$\{0\text{-}3\}_2$	306
Water	153
Superplasticizer (Konsan)	6.12
Air-entraining admixture	0.612

Table 3.31. *Mixture Proportions of the Grout Used for Specimen* PAC_2^2

Material Used	Amount (kg/ m^3)
Portland Cement (PÇ 42.5)	115
Fly Ash	173
$\{0\text{-}3\}_1$	288
$\{0\text{-}3\}_2$	288
Water	144
Superplasticizer (Konsan)	5.76
Air-entraining admixture	0.576

Table 3.32. *Mixture Proportions of the Grout Used for Specimen* PAC_3^2

Material Used	Amount (kg/ m^3)
Portland Cement (PÇ 42.5)	118
Brick Powder	177
$\{0\text{-}3\}_1$	295
$\{0\text{-}3\}_2$	295
Water	148
Superplasticizer (Konsan)	5.90
Air-entraining admixture	0.590

Table 3.33. *Mixture Proportions of the Grout Used for Specimen* PAC_4^2

Material Used	**Amount (kg/ m^3)**
Blended Cement ($PKÇ_2$)	175
Fly Ash	113
$\{0\text{-}3\}_1$	288
$\{0\text{-}3\}_2$	288
Water	144
Superplasticizer (Konsan)	5.76
Air-entraining admixture	0.576

When the above tables about the mixture proportions of the grouts for the PAC concrete specimens are examined, it can bee seen that except specimens PAC_1^2 and PAC_1^1, all the PAC specimens include pozzolan more than 50% (considering that the blended cements used include about 35% pozzolanic addition). The reason why the proportion of pozzolans are chosen as so much is that, the recent studies show that use of high volume pozzolan has a significant improvement on the durability of concrete, without causing any noticeable adverse effect on the mechanical properties [62-64].

CHAPTER 4

TEST RESULTS and DISCUSSIONS

4.1. Test Results

4.1.1. Temperature Measurements

As it is mentioned in Chapter 3, four thermocouples were located in each concrete specimen during the stage of coarse aggregate preplacement; two of which at the centre of the specimens, and two of which close to the surface. Again as it was mentioned earlier, the reason of using two thermocouples at the same concrete depth was both to insure the continuity of taking measurement in case one of the thermocouples gets out of order, and to provide a comparison between the outputs of the two thermocouples in order to be able to check whether the thermocouples are working properly or not. After making such a comparison between the individual results and getting satisfied that both of the two thermocouples are working properly, the average of the outputs of these two thermocouples located at the same concrete depth has been noted down as the temperature of concrete at that depth at that time. The surface and the central temperatures of the specimens have been followed by this way, by taking periodical measurements until the temperatures reached a steady-state. This periodical temperature measurements have been taken every 8 hours, and reaching of the temperatures to a steady-state has been defined as remaining of both surface and central temperatures at the same values more than 24 hours, with a temperature difference of 1-2 ^{0}C. This period has taken one to two weeks depending on the mix design of the specimen. During this period, the specimens have been cured in a special tent at T= 20 ± 2 ^{0}C and RH= 70 ± 20 %. Also the formworks have not been removed during this period, so that the nylon coating inside the formwork served as a sealing-curing. The results of surface and the central temperature measurements with the resulting temperature differences are given in both tabular and graphical format for the overall twelve concrete specimens one by one below.

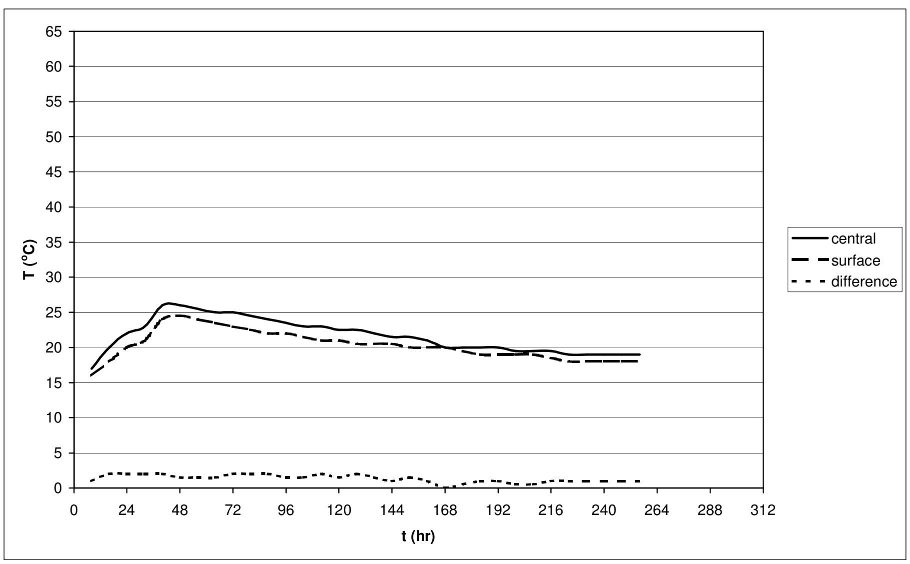

Figure 4.1. *Temperature Measurement Results of the CC Specimen in graphical format*

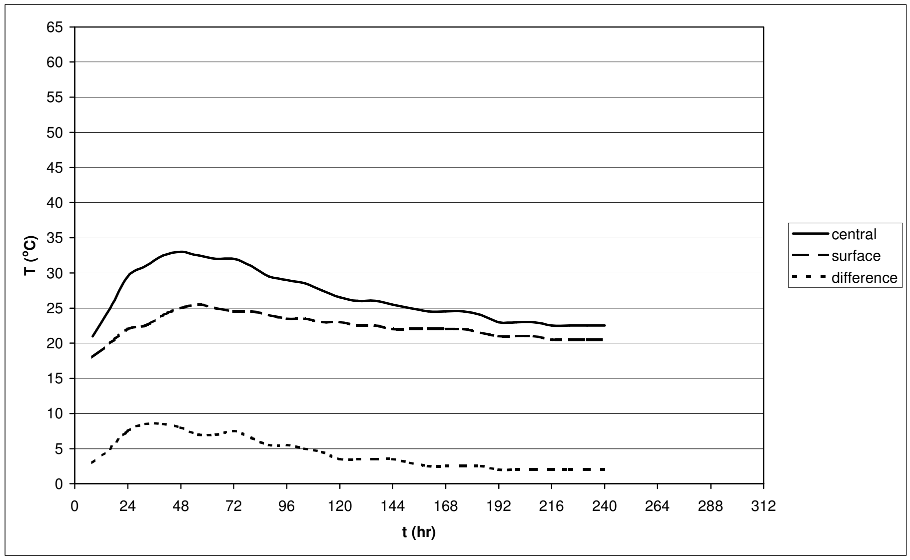

Figure 4.2. Temperature Measurement Results of the Specimen PAC_1^1 *in graphical format*

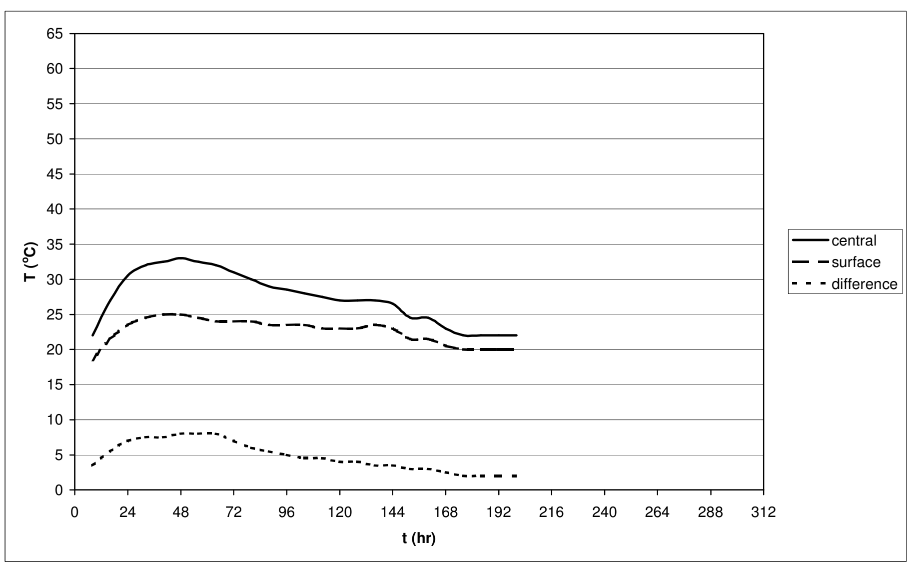

Figure 4.3. *Temperature Measurement Results of the Specimen PAC_2^1 in graphical format*

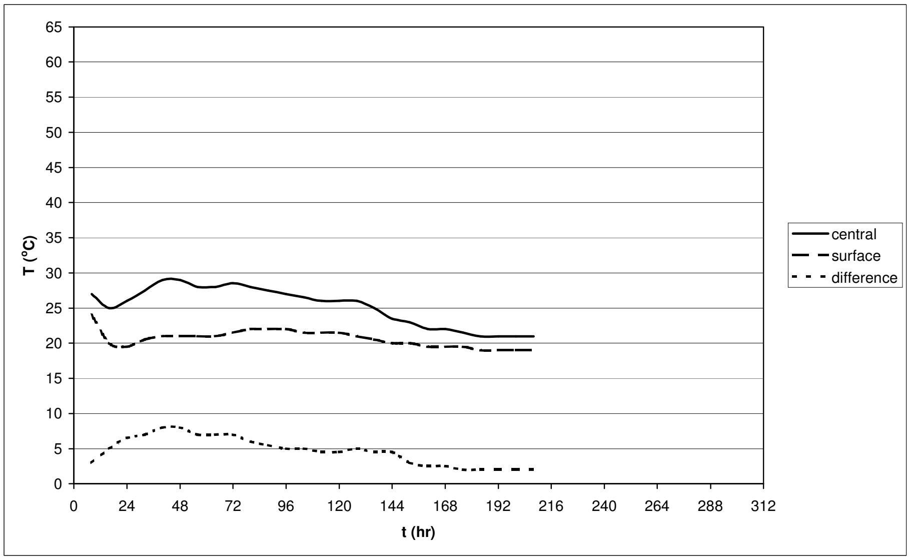

Figure 4.4. *Temperature Measurement Results of the Specimen PAC_3^1 In graphical format*

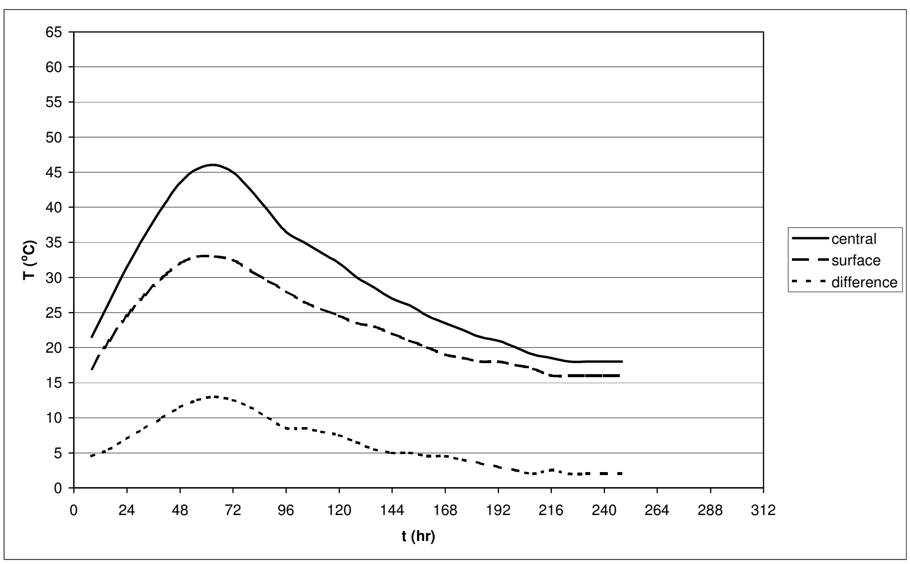

Figure 4.5. *Temperature Measurement Results of the Specimen PAC_4^1 in graphical format*

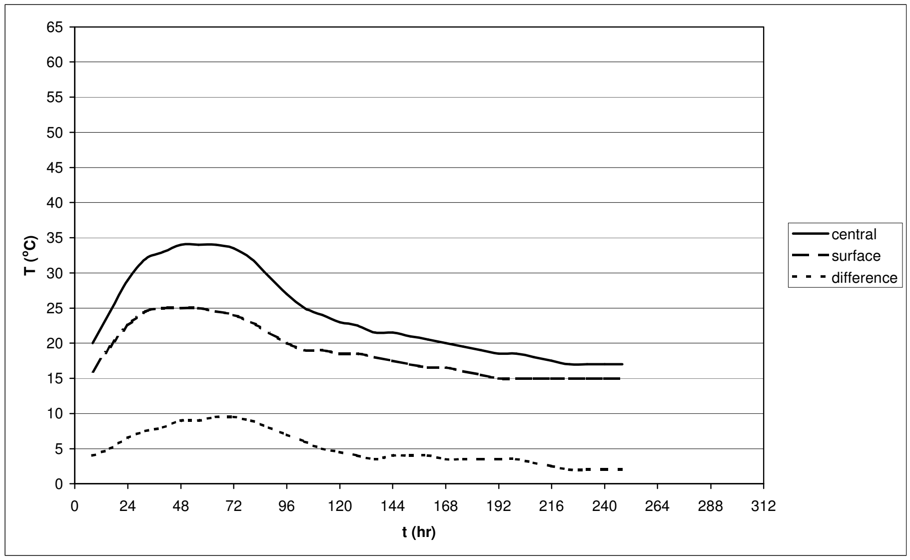

Figure 4.6. *Temperature Measurement Results of the Specimen* PAC_5^1 *in graphical format*

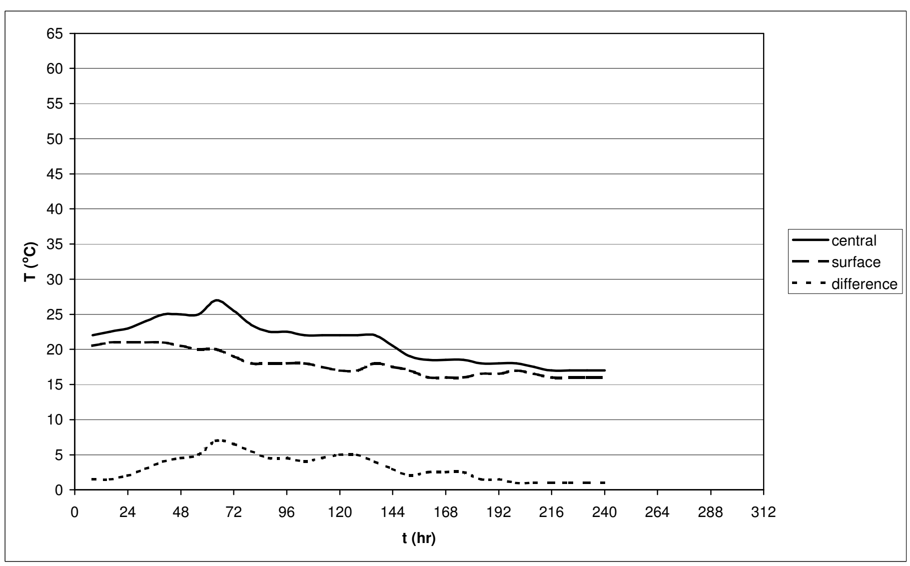

Figure 4.7. *Temperature Measurement Results of the Specimen* PAC_6^1 *in graphical format*

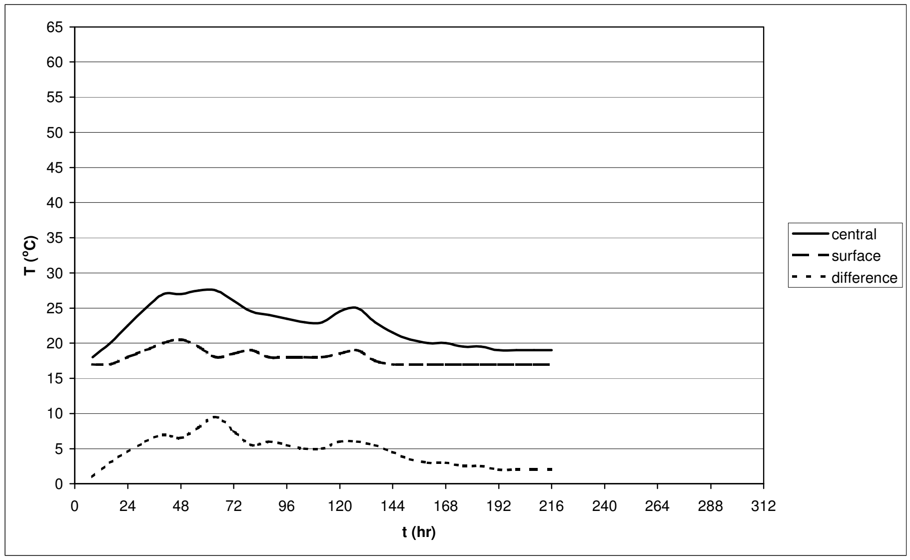

Figure 4.8. *Temperature Measurement Results of the Specimen* PAC_7^1 *in graphical format*

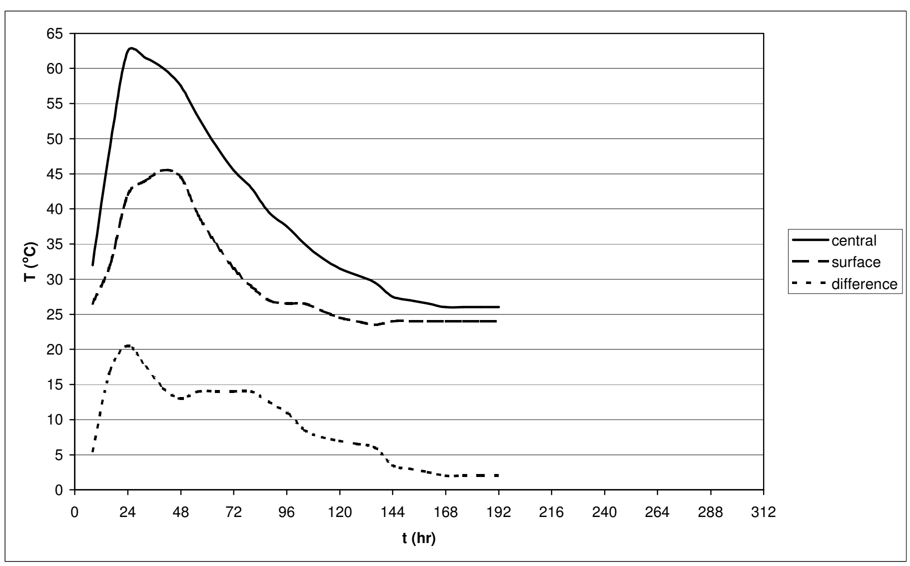

Figure 4.9. *Temperature Measurement Results of the Specimen* PAC_1^2 *in graphical format*

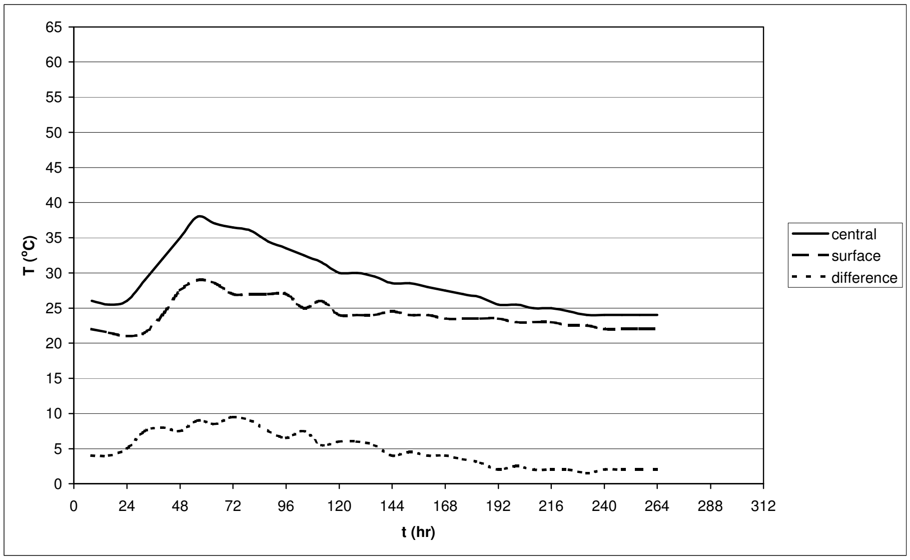

Figure 4.10. *Temperature Measurement Results of the Specimen* PAC_2^2 *in graphical format*

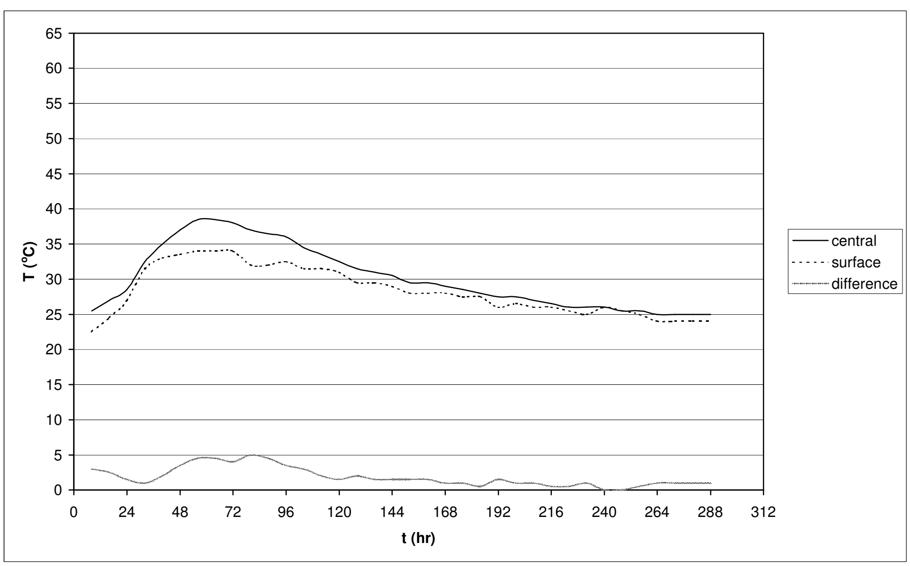

Figure 4.11. *Temperature Measurement Results of the Specimen* PAC_3^2 *in graphical format*

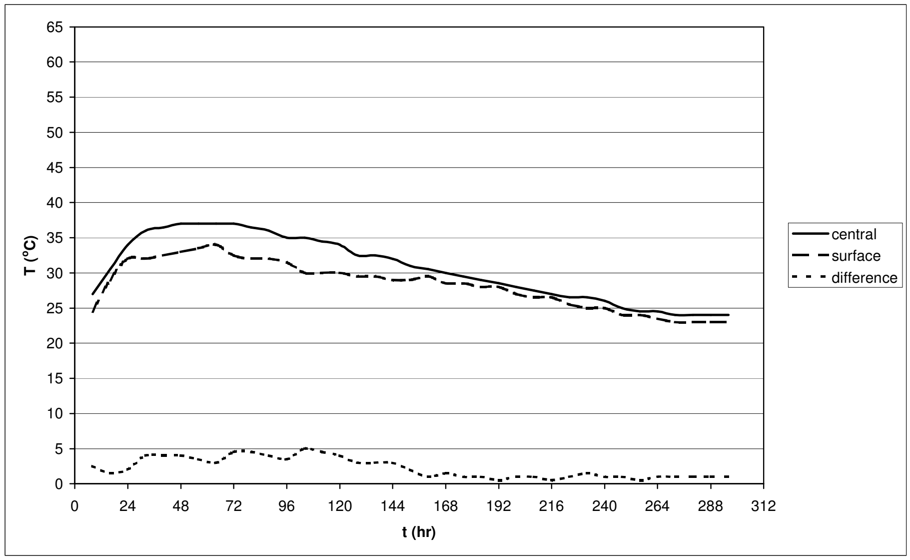

Figure 4.12.*Temperature Measurement Results of the Specimen PAC*$_4^2$
in graphical format

Table 4.1. *Temperature Measurement Results of the CC Specimen in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	17	16	1
16	20	18	2
24	22	20	2
32	23	21	2
40	26	24	2
48	26	24.5	1.5
56	25.5	24	1.5
64	25	23.5	1.5
72	25	23	2
80	24.5	22.5	2
88	24	22	2
96	23.5	22	1.5
104	23	21.5	1.5
112	23	21	2
120	22.5	21	1.5
128	22.5	20.5	2
136	22	20.5	1.5
144	21.5	20.5	1
152	21.5	20	1.5
160	21	20	1
168	20	20	0
176	20	19.5	0.5
184	20	19	1
192	20	19	1
200	19.5	19	0.5
208	19.5	19	0.5
216	19.5	18.5	1
224	19	18	1
232	19	18	1
240	19	18	1
248	19	18	1
256	19	18	1

Table 4.2. *Temperature Measurement Results of the Specimen PAC_1^1 in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	21	18	3
16	25	20	5
24	29.5	22	7.5
32	31	22.5	8.5
40	32.5	24	8.5
48	33	25	8
56	32.5	25.5	7
64	32	25	7
72	32	24.5	7.5
80	31	24.5	6.5
88	29.5	24	5.5
96	29	23.5	5.5
104	28.5	23.5	5
112	27.5	23	4.5
120	26.5	23	3.5
128	26	22.5	3.5
136	26	22.5	3.5
144	25.5	22	3.5
152	25	22	3
160	24.5	22	2.5
168	24.5	22	2.5
176	24.5	22	2.5
184	24	21.5	2.5
192	23	21	2
200	23	21	2
208	23	21	2
216	22.5	20.5	2
224	22.5	20.5	2
232	22.5	20.5	2
240	22.5	20.5	2

Table 4.3. *Temperature Measurement Results of the Specimen PAC$^1_2$ in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	22	18.5	3.5
16	27	21.5	5.5
24	30.5	23.5	7
32	32	24.5	7.5
40	32.5	25	7.5
48	33	25	8
56	32.5	24.5	8
64	32	24	8
72	31	24	7
80	30	24	6
88	29	23.5	5.5
96	28.5	23.5	5
104	28	23.5	4.5
112	27.5	23	4.5
120	27	23	4
128	27	23	4
136	27	23.5	3.5
144	26.5	23	3.5
152	24.5	21.5	3
160	24.5	21.5	3
168	23	20.5	2.5
176	22	20	2
184	22	20	2
192	22	20	2
200	22	20	2

Table 4.4. *Temperature Measurement Results of the Specimen* PAC_3^1
In tabular format

t (hr)	**T** $_{central}$ **(^{0}C)**	**T** $_{surface}$ **(^{0}C)**	**ΔT (^{0}C)**
8	27	24	3
16	25	20	5
24	26	19.5	6.5
32	27.5	20.5	7
40	29	21	8
48	29	21	8
56	28	21	7
64	28	21	7
72	28.5	21.5	7
80	28	22	6
88	27.5	22	5.5
96	27	22	5
104	26.5	21.5	5
112	26	21.5	4.5
120	26	21.5	4.5
128	26	21	5
136	25	20.5	4.5
144	23.5	20	4.5
152	23	20	3
160	22	19.5	2.5
168	22	19.5	2.5
176	21.5	19.5	2
184	21	19	2
192	21	19	2
200	21	19	2
208	21	19	2

Table 4.5. *Temperature Measurement Results of the Specimen PAC_4^1 in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	21.5	17	4.5
16	26.5	21	5.5
24	31.5	24.5	7
32	36	27.5	8.5
40	40	30	10
48	43.5	32	11.5
56	45.5	33	12.5
64	46	33	13
72	45	32.5	12.5
80	42.5	31	11.5
88	39.5	29.5	10
96	36.5	28	8.5
104	35	26.5	8.5
112	33.5	25.5	8
120	32	24.5	7.5
128	30	23.5	6.5
136	28.5	23	5.5
144	27	22	5
152	26	21	5
160	24.5	20	4.5
168	23.5	19	4.5
176	22.5	18.5	4
184	21.5	18	3.5
192	21	18	3
200	20	17.5	2.5
208	19	17	2
216	18.5	16	2.5
224	18	16	2
232	18	16	2
240	18	16	2
248	18	16	2

Table 4.6. *Temperature Measurement Results of the Specimen* PAC_5^1 *in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	20	16	4
16	24.5	19.5	5
24	29	22.5	6.5
32	32	24.5	7.5
40	33	25	8
48	34	25	9
56	34	25	9
64	34	24.5	9.5
72	33.5	24	9.5
80	32	23	9
88	29.5	21.5	8
96	27	20	7
104	25	19	6
112	24	19	5
120	23	18.5	4.5
128	22.5	18.5	4
136	21.5	18	3.5
144	21.5	17.5	4
152	21	17	4
160	20.5	16.5	4
168	20	16.5	3.5
176	19.5	16	3.5
184	19	15.5	3.5
192	18.5	15	3.5
200	18.5	15	3.5
208	18	15	3
216	17.5	15	2.5
224	17	15	2
232	17	15	2
240	17	15	2
248	17	15	2

Table 4.7. *Temperature Measurement Results of the Specimen* PAC_6^1 *in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	22	20.5	1.5
16	22.5	21	1.5
24	23	21	2
32	24	21	3
40	25	21	4
48	25	20.5	4.5
56	25	20	5
64	27	20	7
72	25.5	19	6.5
80	23.5	18	5.5
88	22.5	18	4.5
96	22.5	18	4.5
104	22	18	4
112	22	17.5	4.5
120	22	17	5
128	22	17	5
136	22	18	4
144	20.5	17.5	3
152	19	17	2
160	18.5	16	2.5
168	18.5	16	2.5
176	18.5	16	2.5
184	18	16.5	1.5
192	18	16.5	1.5
200	18	17	1
208	17.5	16.5	1
216	17	16	1
224	17	16	1
232	17	16	1
240	17	16	1

Table 4.8. *Temperature Measurement Results of the Specimen* PAC_7^1 *in tabular format*

t (hr)	$T_{central}$ **(^{0}C)**	$T_{surface}$ **(^{0}C)**	**ΔT (^{0}C)**
8	18	17	1
16	20	17	3
24	22.5	18	4.5
32	25	19	6
40	27	20	7
48	27	20.5	6.5
56	27.5	19.5	8
64	27.5	18	9.5
72	26	18.5	7.5
80	24.5	19	5.5
88	24	18	6
96	23.5	18	5.5
104	23	18	5
112	23	18	5
120	24.5	18.5	6
128	25	19	6
136	23	17.5	5.5
144	21.5	17	4.5
152	20.5	17	3.5
160	20	17	3
168	20	17	3
176	19.5	17	2.5
184	19.5	17	2.5
192	19	17	2
200	19	17	2
208	19	17	2
216	19	17	2

Table 4.9. *Temperature Measurement Results of the Specimen PAC_1^2 in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	32	26.5	5.5
16	49	32	17
24	62.5	42	20.5
32	61.5	44	17.5
40	60	45.5	14.5
48	57.5	44.5	13
56	53	39	14
64	49	35	14
72	45.5	31.5	14
80	43	29	14
88	39.5	27	12.5
96	37.5	26.5	11
104	35	26.5	8.5
112	33	25.5	7.5
120	31.5	24.5	7
128	30.5	24	6.5
136	29.5	23.5	6
144	27.5	24	3.5
152	27	24	3
160	26.5	24	2.5
168	26	24	2
176	26	24	2
184	26	24	2
192	26	24	2

Table 4.10. *Temperature Measurement Results of the Specimen* PAC_2^2 *in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	26	22	4
16	25.5	21.5	4
24	26	21	5
32	29	21.5	7.5
40	32	24	8
48	35	27.5	7.5
56	38	29	9
64	37	28.5	8.5
72	36.5	27	9.5
80	36	27	9
88	34.5	27	7.5
96	33.5	27	6.5
104	32.5	25	7.5
112	31.5	26	5.5
120	30	24	6
128	30	24	6
136	29.5	24	5.5
144	28.5	24.5	4
152	28.5	24	4.5
160	28	24	4
168	27.5	23.5	4
176	27	23.5	3.5
184	26.5	23.5	3
192	25.5	23.5	2
200	25.5	23	2.5
208	25	23	2
216	25	23	2
224	24.5	22.5	2
232	24	22.5	1.5
240	24	22	2
248	24	22	2
256	24	22	2
264	24	22	2

Table 4.11. *Temperature Measurement Results of the Specimen* PAC_3^2 *in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	25.5	22.5	3
16	27	24.5	2.5
24	28.5	27	1.5
32	32.5	31.5	1
40	35	33	2
48	37	33.5	3.5
56	38.5	34	4.5
64	38.5	34	4.5
72	38	34	4
80	37	32	5
88	36.5	32	4.5
96	36	32.5	3.5
104	34.5	31.5	3
112	33.5	31.5	2
120	32.5	31	1.5
128	31.5	29.5	2
136	31	29.5	1.5
144	30.5	29	1.5
152	29.5	28	1.5
160	29.5	28	1.5
168	29	28	1
176	28.5	27.5	1
184	28	27.5	0.5
192	27.5	26	1.5
200	27.5	26.5	1
208	27	26	1
216	26.5	26	0.5
224	26	25.5	0.5
232	26	25	1
240	26	26	0
248	25.5	25.5	0
256	25.5	25	0.5
264	25	24	1
272	25	24	1
280	25	24	1
288	25	24	1

Table 4.12. *Temperature Measurement Results of the Specimen* PAC_4^2 *in tabular format*

t (hr)	$T_{central}$ (^{0}C)	$T_{surface}$ (^{0}C)	ΔT (^{0}C)
8	27	24.5	2.5
16	30.5	29	1.5
24	34	32	2
32	36	32	4
40	36.5	32.5	4
48	37	33	4
56	37	33.5	3.5
64	37	34	3
72	37	32.5	4.5
80	36.5	32	4.5
88	36	32	4
96	35	31.5	3.5
104	35	30	5
112	34.5	30	4.5
120	34	30	4
128	32.5	29.5	3
136	32.5	29.5	3
144	32	29	3
152	31	29	2
160	30.5	29.5	1
168	30	28.5	1.5
176	29.5	28.5	1
184	29	28	1
192	28.5	28	0.5
200	28	27	1
208	27.5	26.5	1
216	27	26.5	0.5
224	26.5	25.5	1
232	26.5	25	1.5
240	26	25	1
248	25	24	1
256	24.5	24	0.5
264	24.5	23.5	1
272	24	23	1
280	24	23	1
288	24	23	1
296	24	23	1

4.1.2. Results of Core Testing

4.1.2.1. Compressive Strengths

Three core specimens have been taken from each concrete specimen at the ages of 28 days, 90 days, 6 months and 1 year. The concrete specimens have been examined in three portions along the depth as it is explained at Figure 3.3, and the three core specimens to be taken at the same age have been taken from different portions in order to check the uniformity of concrete along the depth. In order to have an idea about the uniformity of concrete along the width also, these three core specimens to be taken from vertically different portions have also been taken from horizontally different positions, again as sketched at Figure 3.3. These core specimens have been tested under compression and the average of the three cores have been noted down as the compressive strength of that concrete specimen at that age; but before taking the average of three, the individual results have been compared to check the concrete uniformity, as explained above. Since most of the twelve concrete specimens have been prepared at different times, when this thesis was written the results of PAC^2 specimens at the age of 1 year were not ready yet. The available results are given at Table 4.13.

Table 4.13. *Results of Compressive Strength Test carried out on the Core Specimens*

Specimen	**Compressive Strength (MPa)**			
	28 days	**90 days**	**6 months**	**1 year**
CC	11.3	15.1	18.3	20.0
PAC_1^1	10.9	14.9	18.6	19.5
PAC_2^1	11.6	15.3	18.9	21.3
PAC_3^1	13.7	15.5	23.1	28.9
PAC_4^1	19.5	23.7	24.2	25.7
PAC_5^1	10.6	14.9	18.2	19.6
PAC_6^1	12.1	16.0	19.9	21.2
PAC_7^1	11.9	13.6	17.5	19.2
PAC_1^2	28.4	31.3	36.0	-
PAC_2^2	16.5	19.9	21.9	-
PAC_3^2	15.9	19.9	24.0	-
PAC_4^2	14.6	18.9	21.3	-

4.1.2.2. Moduli of Elasticity

As it is mentioned at Section 4.1.2.1, in order to determine the compressive strength at a specified age, three core specimens have been taken from each specimen; but after one of these three cores was crushed under compression, before crushing the remaining two, modulus of elasticity has also been determined, and the average of the results obtained on these two core specimens has been noted down as the modulus of elasticity value of that concrete specimen at that age.

However again because of the variety in the ages of the specimens, by the time this thesis was written, the modulus of elasticity values of the PAC^2 specimens at the age of 1 year had not been obtained yet. The available results are given at Table 4.14.

Table 4.14. *Modulus of Elasticity Values obtained from the Core Specimens*

Specimen	Modulus of Elasticity (MPa)			
	28 days	**90 days**	**6 months**	**1 year**
CC	11620	16340	19811	21429
PAC_1^1	10146	11947	12890	13833
PAC_2^1	10695	11689	12764	14283
PAC_3^1	11123	13253	15529	16745
PAC_4^1	12732	13936	14461	15487
PAC_5^1	8960	10671	11532	12804
PAC_6^1	9013	10404	11459	12258
PAC_7^1	9407	10313	11098	12261
PAC_1^2	19806	28521	39414	-
PAC_2^2	16033	25930	26662	-
PAC_3^2	15879	19874	25816	-
PAC_4^2	11408	17825	20499	-

4.1.3. Results of the Non-destructive Tests

4.1.3.1. Schmidt Hammer Test

The results of the compressive strength test carried out on the core specimens have also been verified by the non-destructive tests carried out on the actual concrete specimens. Schmidt Hammer Test was one of these non-destructive tests that have been carried out. While carrying out the Schmidt Hammer Test, again the concrete specimens have been examined in three portions with respect to depth as sketched in Figure 3.3, and ten data have been taken from each portion. First of all the horizontal uniformity of the concrete was checked by comparing the ten data taken from a portion among themselves. After repeating this procedure for the three portions, the average of the ten data taken from the same portion is noted down as the Schmidt Hammer test result of that portion. Then the vertical uniformity of the concrete was checked by comparing the results of these three portions. Finally the average of the results of the three portions has been taken and noted down as the Schmidt Hammer test result of that concrete specimen at that age. However again because of the variety in ages of the specimens, some data are missing. The available data about this test are given at Table 4.15.

Table 4.15. *Results of the Schmidt Hammer Test*

Specimen	Rebound Number			
	28 days	**90 days**	**6 months**	**1 year**
CC	19	22	26	27
PAC_1^1	20	23	26	26
PAC_2^1	22	25	27	28
PAC_3^1	25	26	29	31
PAC_4^1	31	34	35	36
PAC_5^1	26	30	32	33
PAC_6^1	23	25	27	27
PAC_7^1	22	24	27	28
PAC_1^2	29	30	33	-
PAC_2^2	26	28	29	-
PAC_3^2	24	27	29	-
PAC_4^2	23	25	26	-

4.1.3.2. Ultrasonic Pulse Velocity Test

Ultrasonic Pulse Velocity Test was another non-destructive test which has been carried out on the actual concrete specimens in order to verify the results of the compressive strength test carried out on the core specimens. While carrying out this test, again the specimens have been examined in three portions with respect to depth, but this time three data from each portion has been considered as adequate. Again first of all the three data taken from each portion have been compared among themselves in order to check the horizontal uniformity. Then the average of the three data taken from the same portion has been noted down as the ultrasonic pulse velocity test result of that portion. After repeating this procedure for each portion, this time the results of the three portions have been compared among themselves in order to check the vertical uniformity as well. Finally the average of the results of the three portions has been noted down as the ultrasonic pulse velocity test result of that concrete specimen at that age. Again due to the same reason, by the time this thesis was written the data of PAC^2 specimens for 1 year were not ready yet. The available data about this test by the time of the preparation of this thesis are presented at Table 4.16.

Table 4.16. *Results of the Ultrasonic Pulse Velocity Test*

Specimen	Ultrasonic Pulse Velocity (m/s)			
	28 days	**90 days**	**6 months**	**1 year**
CC	2857	2985	3030	3279
$\mathbf{PAC}_1^1$	2548	2723	3337	3571
$\mathbf{PAC}_2^1$	2565	2725	3054	3571
$\mathbf{PAC}_3^1$	3205	3493	3810	4000
$\mathbf{PAC}_4^1$	3054	3308	3762	3884
$\mathbf{PAC}_5^1$	2565	2703	3294	3555
$\mathbf{PAC}_6^1$	2436	2668	2989	3487
$\mathbf{PAC}_7^1$	2564	2703	3278	3460
$\mathbf{PAC}_1^2$	4167	4333	4546	-
$\mathbf{PAC}_2^2$	4000	4167	4224	-
$\mathbf{PAC}_3^2$	4063	4205	4255	-
$\mathbf{PAC}_4^2$	4016	4093	4178	-

4 2. Discussion of Results

4.2.1. Discussion of Temperature Measurements

Since the topic of this study is 'Use of PAC in Mass Concrete Applications', the major criterion during the evaluation of the specimens is the heat of hydration, or in other words the concrete temperatures obtained as a result of hydration heat. When the above tables and graphs about the results of temperature measurements are examined, it is seen that maximum peak temperature has occurred at the specimen PAC_1^2 with 62.5 ^{0}C, and the minimum peak temperature has occurred at the CC specimen with 26 ^{0}C. However from the risk of thermal cracking point of view, the peak temperature difference is more important than the peak temperature itself. Thus while evaluating the performance of the specimens from resistance against thermal cracking point of view, one should concentrate on the peak temperature differences rather than the peak temperatures. While evaluating the specimens under the topic of temperature measurements, another important criterion is the rate of hydration heat evolution, which is represented by the time of occurrence of the peak temperature difference. In Table 4.17, the peak temperature differences (ΔT_{peak}), the time of occurrence of these peak temperature differences (t_{peak}), and the length of the duration for the temperatures to get stabilized ($t_{stab.}$) are given one by one for each specimen.

__Table 4.17.__ The Peak Temperature Differences, the Corresponding Times of Occurrence and the Length of Duration for the Temperatures to Get Stabilized

Specimen	ΔT_{peak} (^0C)	t_{peak} (hr)	$t_{stab.}$ (hr)
CC	2	16	256
PAC_1^1	8.5	32	240
PAC_2^1	8	48	200
PAC_3^1	8	40	208
PAC_4^1	13	64	248
PAC_5^1	9.5	64	248
PAC_6^1	7	64	240
PAC_7^1	9.5	64	216
PAC_1^2	20.5	24	192
PAC_2^2	9.5	72	264
PAC_3^2	5	80	288
PAC_4^2	5	104	296

As it is seen from Table 4.17, the maximum peak temperature difference has occurred at the specimen PAC_1^2, at the end of 24 hours, which is as expected, since this specimen contains only PC as cementitious material. On the other hand the minimum peak temperature difference seems to have occurred at the CC specimen, only 16 hours after the finishing of the specimen, which is very unexpected, since a so low peak temperature difference is not reasonable for a conventional concrete which contains 250 kg/m^3 cementitious material. Thus such an explanation can be made: This specimen had a very porous structure since it did not contain any fine aggregate and since the cementitious material content used was not sufficient to fill all the voids. As a result of this the ends of the thermocouples which had been placed inside the specimen and which worked as a receiver for the determination of internal temperature, probably remained in a pore and did not contact the concrete directly; so they could not provide correct measurement of concrete temperatures. Except this one, the other results seem quite reasonable according to the mix design of the corresponding specimen. The minimum peak temperature difference among the PAC^1 specimens has occurred at the specimen PAC_6^1 as 7 ^0C, which contains 67% mineral admixture at total, 50% of which is brick powder. On the other hand the minimum peak temperature difference among the PAC^2 specimens has occurred as 5 ^0C at two specimens, PAC_3^2 and PAC_4^2, one of which contains again high volume brick powder. This proves that the occurrence of minimum peak temperature difference at the specimens which contain brick powder is not a coincidence. As it is mentioned at Section 3.7.2.6, the recent studies show that use of brick powder as mineral admixture improves the durability of concrete significantly. As a result of this study it can be added that it also decreases the heat of hydration significantly when used at high volumes. In other words it can be said that brick powder is a very suitable mineral admixture for mass concrete applications.

As it is explained before, the procedure of following concrete temperatures by the thermocouples has been continued until the temperatures got stabilized, which is defined as remaining of the surface and the central temperatures at the same values more than 24 hours, with a negligible temperature difference, such as 1-2 ^0C. Thus while discussing the results of temperature measurement, another important criterion is the length of this duration, which represents the rate of heat release of the specimen. Evaluating the specimens from this criterion point of view, again as it is seen from Table 4.17, the shortest duration for the temperatures to get stabilized has occurred at the specimen PAC_6^1, which also showed the maximum peak temperature difference at the minimum time. On the other hand if the result of the CC specimen is again kept out of comparison because of the same reason, the longest durations have been shown by the specimens PAC_3^2 and PAC_4^2, which also showed the minimum peak temperature differences with the slowest rates of heat release. As a result it can be concluded that, as the peak temperature increases (which means that as the amount of heat of hydration increases), the time required to reach this peak value decreases (which means that the rate of heat evolution increases), and the length of duration for the temperatures to get stabilized decreases.

To make further discussion about the results of temperature measurement, relating the temperature obtained with the mix design of the corresponding specimen, the specimens PAC_2^1 and PAC_3^1 have the same peak temperature differences. However PAC_3^1 reached this peak value faster than PAC_2^1. On the other hand the temperatures of PAC_2^1 got stabilized faster than those of PAC_3^1, which seems as a contradiction according to the conclusion made above. However since the time differences under discussion are so negligible, this can be taken as a minor exception. However reaching of PAC_3^1 to the peak temperature difference value faster than PAC_2^1 (which indicates that the rate of heat evolution of PAC_3^1 is higher than that of PAC_2^1) can be related to the mix designs of the specimens; in such a way that, although both of the two specimens include 68.8% mineral admixture at total, and although in both of the two specimens 18.8% of this total mineral admixture content is the same truss, in PAC_2^1 the rest 50% of the total mineral admixture content is rock powder, on the other hand in PAC_3^1 it consists of 25% rock powder and 25% fly ash. Considering that both of the two specimens have the same aggregate gradation, same cement, same chemical admixtures at the same percentages, the only difference in the mix designs of the two specimens is fly ash. Thus it can be concluded that addition of fly ash causes a little bit faster evolution, and a little bit slower release of heat.

Another comparison can be made between the specimens PAC_5^1 and PAC_7^1, which have reached the same peak temperature difference values at the same time. However the temperatures of PAC_7^1 got stabilized faster than those of PAC_5^1. Both of these two specimens contain 50% portland-composite cement (PKÇ) and 50% mineral admixture as cementitious material. Considering that the two types of PKÇ they contain have more or less the same percentage of pozzolanic addition, the main difference between the cementitious material composition of these two specimens is the type of mineral admixture they contain at a proportion of 50%. PAC_5^1 has all of these 50% mineral admixture as fly ash, on the other hand the 50% mineral admixture of PAC_7^1 is composed of 25% fly ash and 25% brick powder. Thus the faster stabilization of the temperatures of PAC_7^1 can be related to its brick powder content. As a result of a previous comparison, it has already been concluded that use of brick powder decreases the peak temperature difference, and as a result of this comparison it can be added that use of brick powder also decreases the length of duration for the temperatures to get stabilized; in other words it increases the rate of heat release, which is again a positive affect.

Another comparison can be made between the specimens PAC_4^1 and PAC_5^1, the temperatures of which have got stabilized at the same time and which reached the peak temperature differences at the same time, but to different values. The peak temperature difference of PAC_4^1 is higher than that of PAC_5^1. Since both of the two specimens are PAC^1 specimens, there is no need to state that they have the same preplaced aggregate gradation. Coming to the mixture proportions of their grouts, if Table 3.26 and Table 3.27 are compared, it can be seen that the only difference between these two specimens is the type of mineral admixture they contain. Thus the difference between their peak temperature difference values can easily be related to this factor. In fact when it is considered that the mineral admixture of PAC_4^1 is GGBFS and that of PAC_5^1 is fly ash, it is not surprising that the peak temperature difference of PAC_4^1 is a higher value.

Finally a last discussion about the results of temperature measurements can be made by comparing the specimen PAC_6^1 with PAC_4^1 and PAC_7^1, since both of the three specimens have the same length of duration for reaching the peak temperature differences, but different values of peak temperature difference and duration of temperature stabilization at different lengths. For example PAC_4^1 has a higher value of peak temperature difference and a longer duration of temperature stabilization compared to those of PAC_6^1. On the other hand while PAC_7^1 has a value of peak temperature difference higher than that of PAC^1_6, it has a duration of temperature stabilization shorter than that of PAC^1_6. In other words both PAC^1_4, PAC^1_6 and PAC^1_7 have peak temperature values higher than that of PAC^1_6; but while PAC^1_4 has a longer, PAC^1_7 has a shorter duration of temperature stabilization than that of PAC^1_6. If Table 3.26 and Table 3.28 are examined, it is seen that the only difference between the mix designs of PAC^1_4 and PAC^1_6 is the type of mineral admixture they contain. While the mineral admixture used in PAC^1_4 is GGBFS , the one used in PAC^1_6 is brick powder. Thus in the light of the conclusions made in the above paragraphs, it is not surprising that PAC^1_4 has a higher value of peak temperature difference and a longer duration of temperature stabilization than those of PAC^1_6. On the other hand when Table 3.28 and Table 3.29 are examined, it is seen that there are two differences between the mix designs of PAC^1_6 and PAC^1_7; one of which is again between the mineral admixtures, the other is between the cements of the specimens. The difference between the mineral admixtures is that, while PAC^1_6 has 50% brick powder, PAC^1_7 has 25% brick powder and 25% fly ash. In the light of the previously made conclusions, it is known that brick powder provides a shorter duration of temperature stabilization compared to that provided by fly ash. Thus the reason of PAC^1_7's having a shorter duration of temperature stabilization than that of PAC^1_6 can not be related to its mineral admixture. So the only possible reason left is the difference between the cements of the specimens. In fact both of the two specimens have been made by using portland composite cements which are classified under the same type according to the related Turkish Standard (PKÇ B 32.5), but they are produced by two different factories and consequently they have some different properties, which can be seen when Table 3.12 and Table 3.13 are examined.

4.2.2. Discussion of Compressive Strengths

When Table 4.13, where the results of compressive strength test which were available by the time this thesis was written have been given, is went over, in general the compressive strengths seem quite low. However it should not be forgotten that these are the compressive strength test results obtained on the core specimens, not on the standard cylinder or cube specimens; because it is well known that the compressive strength test results obtained on core specimens are always lower than those obtained on standard cylinder or cube specimens. The only difference of opinion about this world widely agreed subject is the degree of reduction. Many researches have been carried out on this subject in order to find out a relation between compressive strength test results obtained on core specimens and those obtained on standard specimens [65-70]. The Concrete Society in UK has published a technical report on this subject as an evaluation of the results of these researches. In this report the relation between the compressive strength test result obtained on the core specimens which have length to diameter ratio equal to 2.0 and that obtained on the standard cube specimens that have 15 cm dimension has been given as follows [71].

$$\sigma_{core} = 0.67\ \sigma_{15}$$

Considering that the compressive strength test result obtained on standard cube specimen with 15 cm dimension is accepted as 1.25

times of that obtained on standard cylinder specimen with 15 cm x 30 cm dimensions, this relation can be modified as follows [51].

$$\sigma_{core} = 0.84\ \sigma_{cylinder}$$

The compressive strength value obtained from core specimens is known as ‘the real strength’, on the other hand the one obtained from standard specimens is known as ‘the potential strength’ of the concrete; and while designing a structure, the potential strength of the concrete is used. Thus while evaluating the compressive strength test results given at Table 4.13, in order to transform these real strengths to potential strengths, which is more widely used, the values should be divided by 0.84. The potential compressive strengths of the specimens, which have been obtained by such a transformation, are given at Table 4.18.

Table 4.18. *Potential Compressive Strengths*

Specimen	Potential Compressive Strength (MPa)			
	28 days	**90 days**	**6 months**	**1 year**
CC	13.5	18.0	21.8	23.8
PAC$_1^1$	13.0	17.7	22.1	23.2
PAC$_2^1$	13.8	18.2	22.5	25.4
PAC$_3^1$	16.3	18.5	27.5	34.4
PAC$_4^1$	23.2	28.2	28.8	30.6
PAC$_5^1$	12.6	17.7	21.7	23.3
PAC$_6^1$	14.4	19.0	23.7	25.2
PAC$_7^1$	14.2	16.2	20.8	22.9
PAC$_1^2$	33.8	37.3	42.9	-
PAC$_2^2$	19.6	23.7	26.1	-
PAC$_3^2$	18.9	23.7	28.6	-
PAC$_4^2$	17.4	22.5	25.4	-

As it is seen from Table 4.18, the highest strength values at all ages are given by the specimen PAC_1^2, as it is expected, since this specimen contains only PC as cementitious material. On the other hand among the PAC^1 specimens, the highest strength values at all ages are given by the specimen PAC_4^1, which is again not surprising, since this specimen contains 50% GGBFS as mineral admixture.

The specimen which has started the strength development from the lowest point; in other words the one which gave the lowest strength value at 28 days is PAC_5^1. This is probably because this specimen contains the highest volume of fly ash with 50% (by weight of the total cementitious material content) among all the specimens.

While carrying out a discussion on the compressive strengths, another important criterion other than the compressive strength values is the rate of strength development. In order to see the rate of strength development of each specimen, it will be useful to draw the relative compressive strength versus time graphs, where the compressive strength values at 28 days have been taken as 100% and the other values at 90 days, 6 months and 1 year have been given relative to this value.

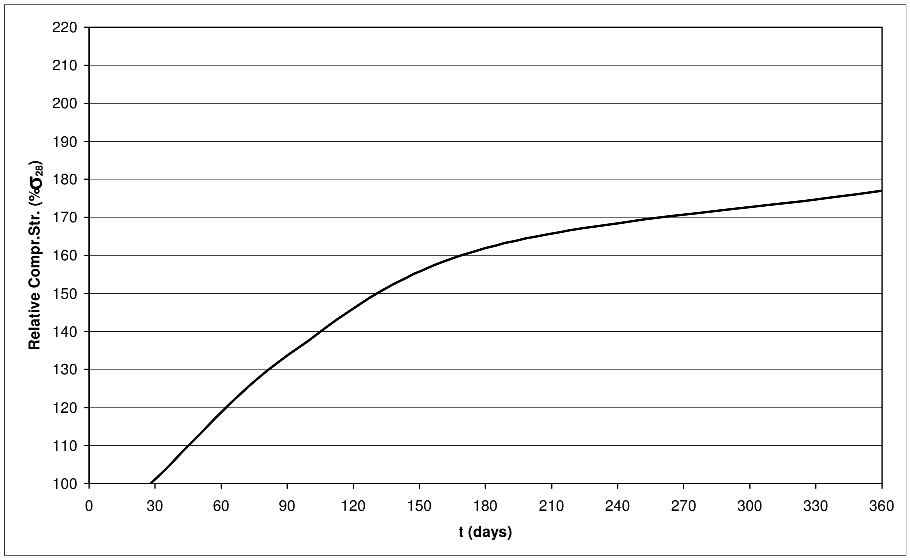

Figure 4.13. Rate of Compressive Strength Development of the CC Specimen

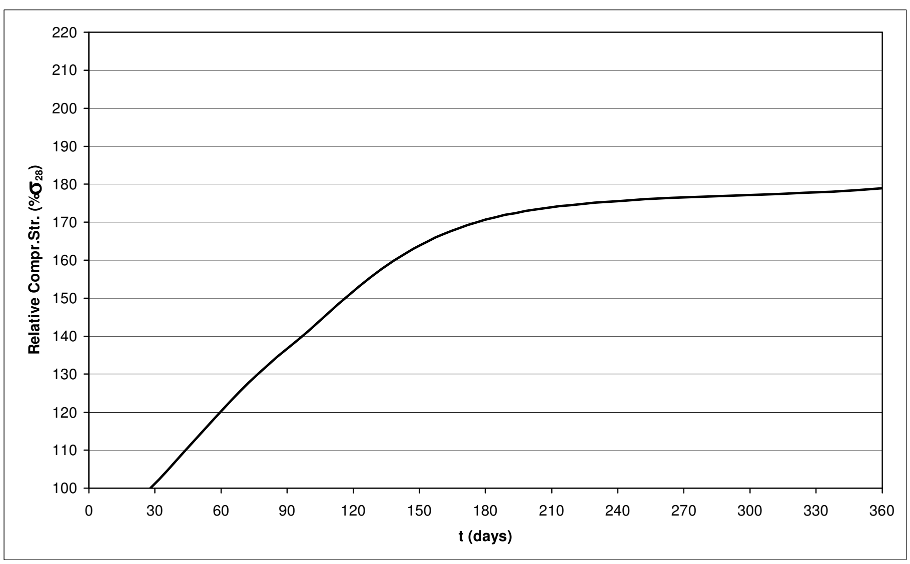

Figure 4.14. Rate of Compressive Strength Development of the Specimen PAC_1^1

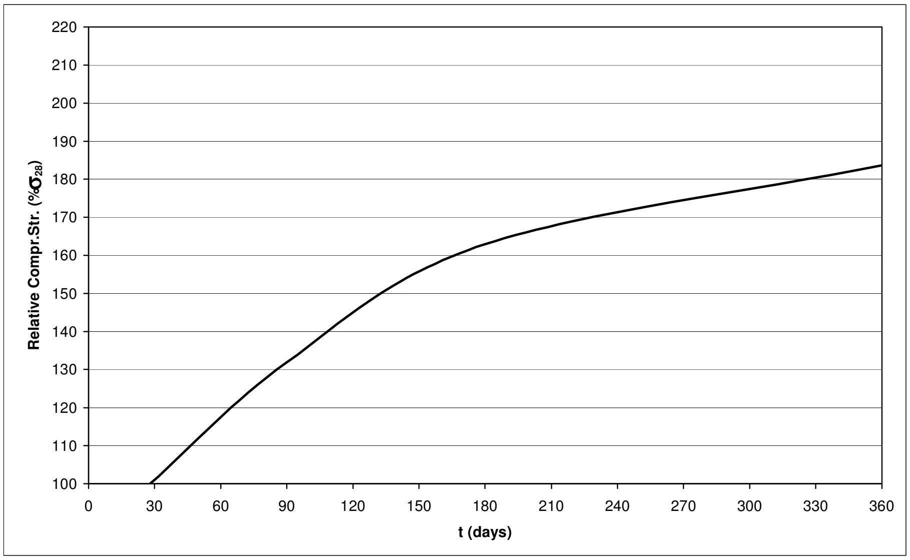

Figure 4.15. Rate of Compressive Strength Development of the Specimen PAC_2^1

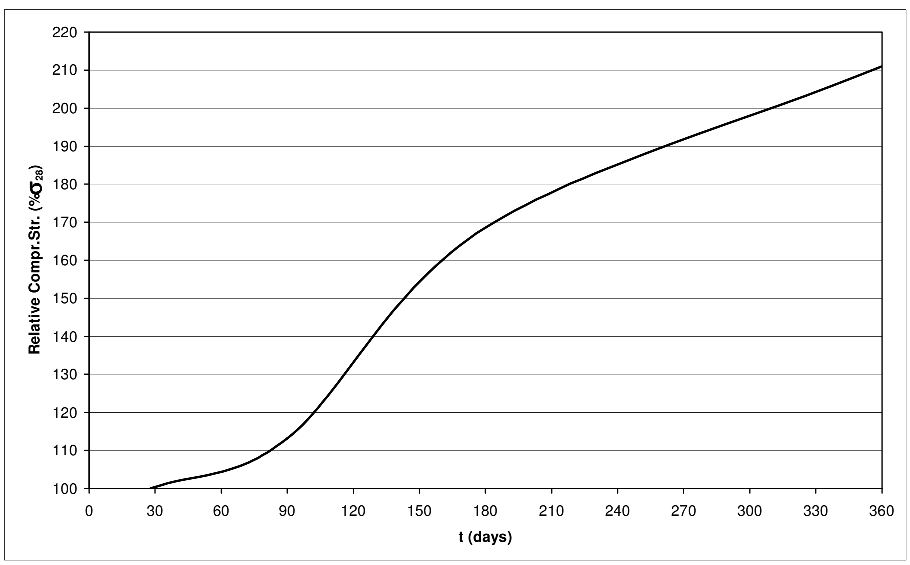

Figure 4.16. Rate of Compressive Strength Development of the Specimen PAC_3^1

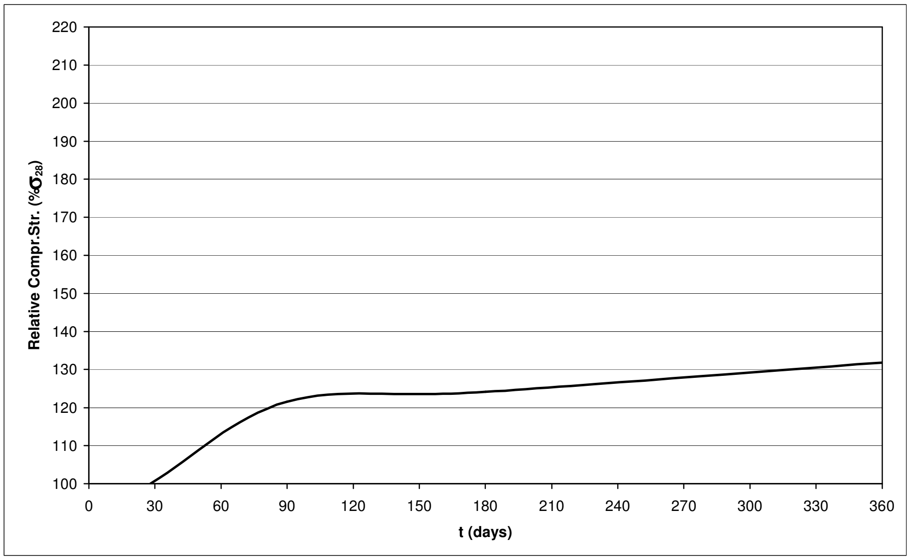

Figure 4.17. Rate of Compressive Strength Development of the Specimen PAC_4^1

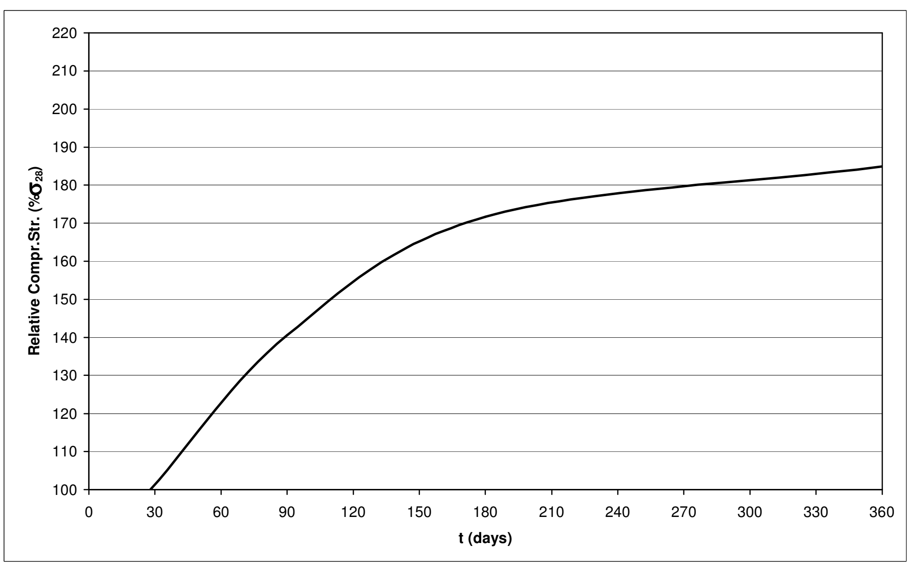

Figure 4.18. Rate of Compressive Strength Development of the Specimen PAC_5^1

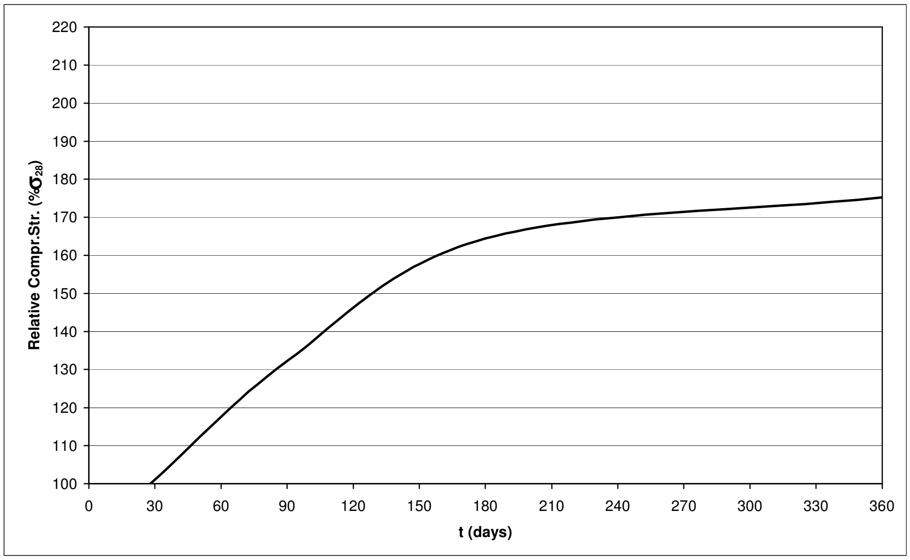

Figure 4.19. Rate of Compressive Strength Development of the Specimen PAC_6^1

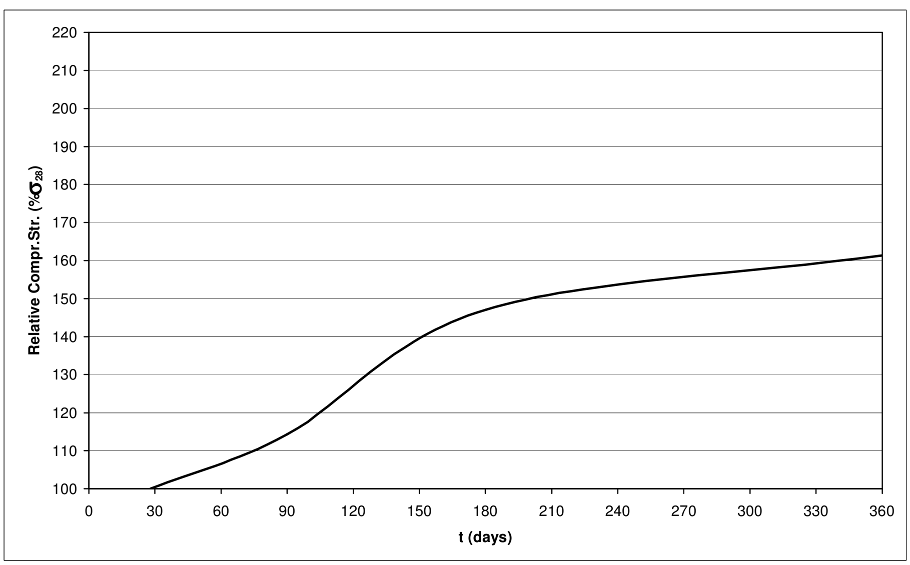

Figure 4.20. Rate of Compressive Strength Development of the Specimen PAC_7^1

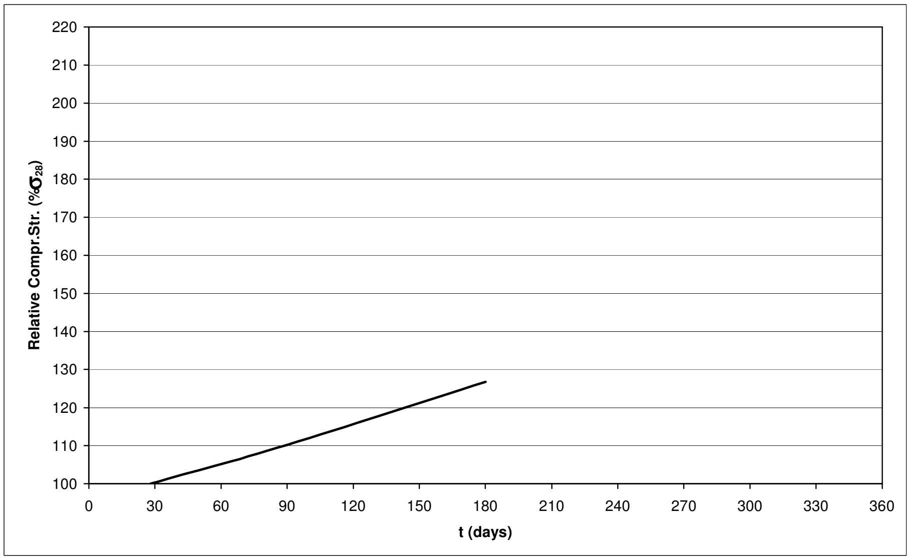

Figure 4.21. Rate of Compressive Strength Development of the Specimen PAC$_1^2$

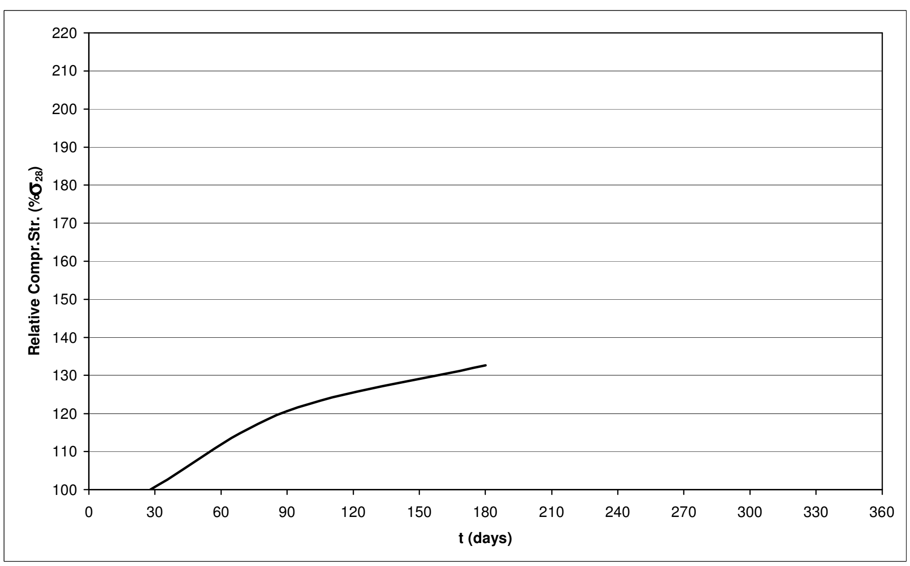

Figure 4.22. Rate of Compressive Strength Development of the Specimen PAC$_2^2$

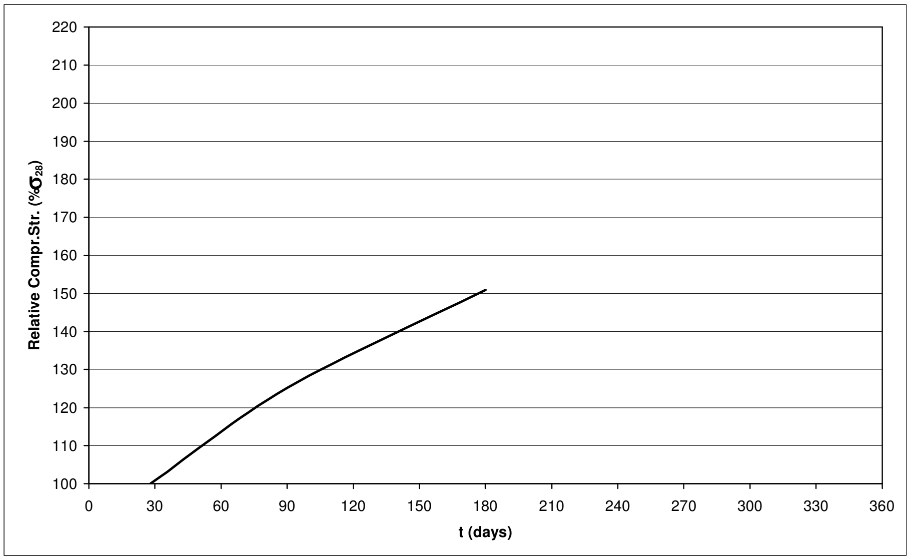

Figure 4.23. Rate of Compressive Strength Development of the Specimen PAC$^{2}_{3}$

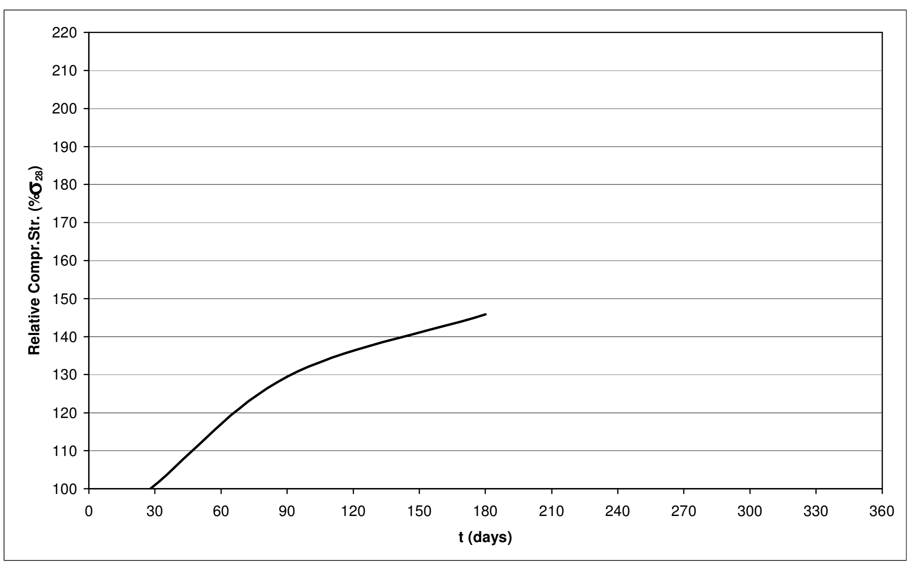

Figure 4.24. Rate of Compressive Strength Development of the Specimen PAC$^{2}_{4}$

As seen from the above graphs, although the specimen PAC_5^1 gave the lowest strength value at the end of 28 days, it showed the maximum rate of strength development at the end of both 90 days and 6 months among all the specimens. Considering that this specimen contains 50% $PKÇ_2$ and 50% fly ash as cementitious material, and since it is known that as the amount of fly ash content increases rate of strength development decreases, it can be said that the reason of the high rate of strength development of this specimen is based on the type of cement it contains; because when Table 3.13 is examined, it can be seen that $PKÇ_2$ is finer than $PKÇ_1$, and it is known that as the fineness of the cement increases the rate of strength development increases.

On the other hand, although the specimen PAC_3^1 gave the second highest strength value at the age of 28 days among all the PAC^1 specimens, it showed the lowest rate of strength development at the age of 90 days again among all the PAC^1 specimens. The cementitious material content of this specimen consists of 50% $PKÇ_1$, 25% fly ash and 25% rock powder. The contribution of $PKÇ_1$ to the early strength is lower than that of $PKÇ_2$, because $PKÇ_2$ is finer than $PKÇ_1$ and its pozzolanic content is a bit less than that of $PKÇ_1$. Thus the high early strength of specimen PAC_3^1 can not be related to its cement type. As a mineral admixture, fly ash and rock powder can not be considered responsible for the high early strength either. Under these circumstances, the only reasonable explanation for the high strength of PAC_3^1 at the age of 28 days can be made by the filler effect of rock powder. On the other hand the low rate of strength development between 28 and 90 days can be related to its fly ash content, rather than the rock powder; because although PAC_2^1 contains 50% $PKÇ_1$

and 50% rock powder, in other words the same proportion of the same type of cement but a much higher amount of rock powder, it has not shown such a low rate of strength development at the same age. Also it is seen that the specimen PAC_3^1 has one of the highest strength value and the rate of strength development among all the PAC^1 specimens at the age of 6 months, and the highest ones again among all the PAC^1 specimens at the age of 1 year. This can be again related to fly ash, because the contribution of fly ash to the strength is usually observed at later ages.

As it is mentioned above, the highest strength values at all ages among all the PAC^1 specimens, in fact among all the specimens except PAC_1^2, are given by the specimen PAC_4^1. However at the same time, the lowest rates of strength development at all ages except 90 days among all the specimens are again given by this specimen. This is probably because GGBFS shows a much faster reaction compared to the other mineral admixtures; and as a result of this, the rate of strength development starts to slow down after 90 days.

Although the specimens PAC_6^1 and PAC_7^1 have more or less the same strength values at the age of 28 days, at the later ages both the strength values and the rates of strength development of the specimen PAC_7^1 has remained much behind of those given by the specimen PAC_6^1. Considering that PAC_6^1 contains 50% $PKÇ_2$ + 50% brick powder and PAC_7^1 contains 50% $PKÇ_1$ + 25% brick powder + 25% fly ash, it can be concluded that the contribution of brick powder to the strength development is higher than that of fly ash. Again considering that the early strength is determined by the cement rather than the pozzolans, the reason of 28 days strengths of the two specimens' being close to each other can be explained reasonably. Even the small difference between these values in favour of the specimen PAC_6^1 can be explained by the small difference between the types of cements; because, as previously mentioned too, $PKÇ_2$ is a bit finer than $PKÇ_1$ and it contains a bit less amount of pozzolanic addition than $PKÇ_1$ does.

As a last discussion on the compressive strength test results of the PAC^1 specimens, the specimens PAC_1^1 and CC can be compared. When Table 4.13 is examined, it is seen that these two specimens have given strength values very close to each other at all ages. Furthermore when Figure 4.13 and Figure 4.14 are examined, it is again seen that the rates of strength development given by these two specimens at each age are also very close to each other. This result is quite reasonable, because the specimen PAC_1^1 had been designed as a PAC version of the CC specimen, having the same types of aggregate with the same proportions as shown at Table 3.4 and Table 3.6, and having the same types of cement and mineral admixture with the same cement to mineral admixture ratio as shown at Table 3.22 and Table 3.23.

Coming to the PAC^2 specimens, although it is seen from Table 4.13 that the highest strength value at each age is given by the specimen PAC_1^2, which had been designed as the control mixture for the PAC^2 specimens, when Figure 4.22, Figure 4.23 and Figure 4.24 are compared with Figure 4.21, this time it is seen that the rate of strength development of this specimen at a specified age has remained as the lowest one among all the PAC^2 specimens. This result is quite reasonable, because this specimen contains only PC as cementitious material, which means the significant portion of strength development occurs in 28 days and then the rate of strength development slows down at the later ages.

The lowest strength values at all ages among all the PAC^2 specimens have been given by the specimen PAC_4^2. Considering that all the PAC^2 specimens have the same preplaced aggregate composition, the different results obtained among these specimens can be related to the mix designs of their mortar. From this point of view, this result is not surprising, because the only specimen among the PAC^2 specimens that contain PKÇ 32.5 as cement type is PAC_4^2, all the others contain PÇ 42.5.

The specimens PAC_2^2 and PAC_3^2 have given more or less the same strength values at 28 days, and absolutely the same value at 90 days, but at 6 months PA_3^2 has given slightly a higher strength value. Also when Figure 4.22 and Figure 4.23 are compared, it is seen that the rate of strength development at a specified age given by PAC_3^2 is higher than that is given by PAC_2^2. When the mix designs of these two specimens are compared, it is seen that the only difference between these two specimens is the type of mineral admixture they include. Considering that PAC_2^2 contains fly ash and PAC_3^2 contains brick powder as mineral admixture, the conclusion that 'the contribution of brick powder to the strength development is higher than that of fly ash', which has already been achieved above while comparing the compressive strength test results of the specimens PAC_6^1 and PAC_7^1, is verified.

4.2.3. Discussion of Elasticity Moduli

The modulus of elasticity of a concrete specimen can either be determined experimentally, as done in this research, or calculated from the compressive strength of that specimen, using one of the empirical formulas offered in the literature. However while designing a concrete structure, usually the modulus of elasticity value which is calculated from the compressive strength value is used, instead of determining it experimentally. The most widely used formulas for this purpose are given below:

- The formula offered by American Concrete Institute (ACI) [72]:

$$E = 0.043 \times U^{1.5} \times \sigma^{0.5}$$

- The formula offered by European Concrete Committee (CEB) [73]:

$$E = 9500 \times (\sigma+8)^{1/3}$$

- The formula offered by Turkish Standards (TS) [74]:

$$E = (3250 \times \sigma^{0.5}) + 14000$$

where;

E: Modulus of Elasticity (MPa)

σ: Compressive Strength (MPa)

U: Unit Weight (kg/m^3)

Since the unit weight data of the specimens have been followed in the scope of this research, the formula offered by ACI can be used while discussing the moduli of elasticity of the specimens. In fact the unit weight of the specimens have not been determined for this purpose, they have been used as a check for concrete uniformity. For this purpose, every core specimen had been weighed before being crushed under compression. First of all the unit weights of the three core specimens taken at a specified age from different portions of the concrete specimen had been compared among each other in order to check whether there is a significant difference, which can be an indication of non-uniformity. If the difference among the three values is reasonable, then the average of them has been recorded as the unit weight of that specimen at that age. Then as another check, the unit weights of each specimen determined at different ages has been compared among themselves. Since the unit weight of a concrete specimen does not change significantly in time, no significant differences among the unit weight data of a specimen determined at different ages are expected. In fact knowing the total amount of material consumed during the preparation of a specimen, there is an expected value of unit weight which can be determined before taking any core specimens. As a last check, after comparing the unit weight data of a specimen among themselves, they can also be compared with this expected value. The unit weight data of each specimen determined at each age which are available by the time this thesis was written and the expected values of unit weight for each specimen which had been calculated by using the amounts of material consumed during the preparation of the specimens are presented at Table 4.19.

Table 4.19. *The Expected and the Obtained Unit Weight Values*

Specimen	Expected Unit Weight (kg/m^3)	Unit Weight obtained from the Core Specimens (kg/m^3)			
		28 days	90 days	6 months	1 year
CC	2355	2356	2471	2292	2547
PAC$_1^1$	2301	2276	2396	2215	2260
PAC$_2^1$	2303	2484	2471	2273	2310
PAC$_3^1$	2282	2452	2365	2490	2248
PAC$_4^1$	2307	2509	2440	2311	2291
PAC$_5^1$	2257	2440	2228	2364	2192
PAC$_6^1$	2294	2242	2260	2219	2260
PAC$_7^1$	2271	2297	2210	2290	2318
PAC$_1^2$	2498	2636	2610	2647	-
PAC$_2^2$	2434	2617	2572	2641	-
PAC$_3^2$	2461	2459	2371	2547	-
PAC$_4^2$	2434	2482	2508	2499	-

The expected modulus of elasticity values for each specimen at each age calculated by the ACI formula using the unit weight and the compressive strength values obtained at the corresponding age are presented at Table 4.20.

Table 4.20. ***Table 4.20.*** *Moduli of Elasticity calculated by the ACI formula*

Specimen	Expected Modulus of Elasticity (MPa)			
	28 days	90 days	6 months	1 year
CC	16530	20524	20184	24719
PAC_1^1	15415	19467	19332	20401
PAC_2^1	18131	20660	20258	22033
PAC_3^1	19325	19471	25679	24638
PAC_4^1	23864	25231	23500	23904
PAC_5^1	16874	17456	21085	19537
PAC_6^1	15879	18480	20051	21272
PAC_7^1	16330	16475	19712	21028
PAC_1^2	31013	32078	35136	-
PAC_2^2	23384	25021	27311	-
PAC_3^2	20908	22146	27078	-
PAC_4^2	20316	23480	24792	-

When Table 4.14 and Table 4.20 are compared, it is seen that for the PAC^1 specimens the modulus of elasticity values obtained on core specimens are approximately 60% of the expected modulus of elasticity values. However this is not a contradiction, on the contrary this result is very consistent with the literature, because in the literature the modulus of elasticity of PAC is found to be slightly higher than that of conventional concrete. The reason of this is explained by the production of PAC by depositing coarse aggregate directly into the forms, where there is a point-to-point contact, instead of being contained in a flowable plastic mixture as in conventional concrete [9,10,11].

On the other hand, as a result of the same comparison, it is seen that the modulus of elasticity values of both the CC specimen and the PAC^2 specimens which have been obtained on the core specimens are more or less same as the expected modulus of elasticity values which have been calculated by the ACI formula, except only for the 28 day values. The modulus of elasticity values obtained on core specimens at the age of 28 days are much lower than the expected modulus of elasticity values calculated for the same age. The reason for the incompatibility between the obtained and the expected modulus of elasticity values at the age of 28 days is quite unclear, but the compatibility between the results of the comparison of obtained and expected modulus of elasticity values for the PAC^2 specimens and for the CC specimen can be accepted as another advantage of the new method of PAC production by which the PAC^2 specimens have been prepared.

4.2.4. Discussion of Schmidt Hammer Test Results

The expected compressive strength values at the specified ages determined from the rebound numbers obtained as a result of the Schmidt Hammer test held at the corresponding ages according to the relation given in the manual of the standard Schmidt Hammer used during the test are presented at Table 4.21 [75].

Table 4.21. *Expected Compressive Strength Values according to the corresponding Schmidt Hammer Rebound Numbers*

Specimen	**Expected Compressive Strength (MPa)**			
	28 days	**90 days**	**6 months**	**1 year**
CC	6.3	10.9	17.0	18.5
PAC_1^1	7.8	12.4	17.0	17.0
PAC_2^1	10.9	15.5	18.5	20.0
PAC_3^1	15.5	17.0	21.6	24.6
PAC_4^1	24.6	29.2	30.7	32.2
PAC_5^1	17.0	23.1	26.1	27.6
PAC_6^1	12.4	15.5	18.5	18.5
PAC_7^1	10.9	13.9	18.5	20.0
PAC_1^2	21.6	23.1	27.6	-
PAC_2^2	17.0	20.0	21.6	-
PAC_3^2	13.9	18.5	21.6	-
PAC_4^2	12.4	15.5	17.0	-

Before starting to make discussion on these values, it should not be forgotten that these compressive strength values are assumed to be obtained on standard cylinder specimens. Thus in order to make a proper comparison, these expected compressive strength values should be compared with the potential compressive strengths of the concrete specimens which are given at Table 4.18; in other words, comparing these values with the compressive strength values obtained on the core specimens (Table 4.13) will not be a proper approach.

When Table 4.21 is compared with Table 4.18, it is seen that except for only the two specimens PAC_4^1 and PAC_5^1, for all the specimens the expected compressive strength values determined from the Schmidt Hammer test are much lower than the corresponding potential compressive strength values. The reason of the exception for these two specimens is quite unclear, but the reason of expected values' being lower than the potential values for the majority of the specimens can be related to the existence of high volumes of mineral admixtures. However this time the specimen PAC_1^2 becomes an exception for this interpretation, since it does not contain any mineral admixture. This way or another, the incompatibility between the potential compressive strength values and the expected values determined from the rebound numbers by using the relation given in the instruction manual of the Schmidt Hammer is quite reasonable, because in the manual it is stated that the given relation is valid for concrete specimens which contain only PC as cementitious material and which have a well-graded aggregate; whereas in this research except one, all the specimens include high volumes of mineral admixtures and none of the specimens include a well-graded aggregate. In fact a new relation between Schmidt Hammer rebound numbers and the expected compressive strength could have been offered for concrete specimens including high volume mineral admixtures in the scope of this study, but to be able to do this, it should have been worked on much more core specimens at each age. This can be thought of for a further study.

4.2.5. Discussion of Ultrasonic Pulse Velocity Test Results

Although there is no direct relation between the velocity of ultrasonic pulses passing through a concrete specimen and the compressive strength of that concrete, the quality of concrete can be evaluated based on the relation between the ultrasonic pulse velocity and the concrete density; in such a way that, as the amount of capillary pores in a concrete specimen increases, the density of that concrete decreases and the ultrasonic pulses can pass that concrete specimen with a lower velocity. The numerical criteria to be used for evaluation of concrete quality based on the ultrasonic pulse velocity test results, which was offered by Whitehurst in 1951 and which is still most widely used, are given at Table 4.22 [76].

Table 4.22. *Evaluation of Concrete Quality by Ultrasonic Pulse Velocity Test*

Ultrasonic Pulse Velocity (m/s)	Concrete Quality
> 4500	very good
3500 - 4500	good
3000 - 3500	questionable
2000 - 3000	weak
< 2000	very weak

The interpretation of the ultrasonic pulse velocity test results according to the criteria offered by Whitehurst is presented at Table 4.23.

Table 4.23. *Interpretation of the Ultrasonic Pulse Velocity Test Results*

Specimen	Concrete Quality			
	28 days	**90 days**	**6 months**	**1 year**
CC	weak	weak	questionable	questionable
PAC$_1^1$	weak	weak	questionable	good
PAC$_2^1$	weak	weak	questionable	good
PAC$_3^1$	questionable	questionable	good	good
PAC$_4^1$	questionable	questionable	good	good
PAC$_5^1$	weak	weak	questionable	good
PAC$_6^1$	weak	weak	weak	questionable
PAC$_7^1$	weak	weak	questionable	questionable
PAC$_1^2$	good	good	very good	-
PAC$_2^2$	good	good	good	-
PAC$_3^2$	good	good	good	-
PAC$_4^2$	good	good	good	-

When Table 4.23 is compared with Table 4.18, where the potential compressive strengths of the specimens are given, it is seen that the interpretation of the ultrasonic pulse velocity test results according to the criteria given at Table 4.22 does not reflect the actual condition of the specimens; according to this interpretation, the specimens seem weaker than they are. The reason of this is clarified when the following relation, which is the basis of ultrasonic pulse velocity test machine, is taken into account [77].

$$V = \sqrt{\frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)}}$$

Where V is the velocity of ultrasonic pulses passing through a concrete specimen; E, ρ, and μ are the modulus of elasticity, density and the Poisson's ratio of that concrete respectively.

As it is seen from this relation, the velocity of ultrasonic pulses passing through a concrete specimen is proportional to the modulus of elasticity of that concrete; and since the modulus of elasticity of PAC is much lower than that of conventional concrete as previously explained, when the ultrasonic pulse velocity test results obtained on PAC are interpreted according to the criteria developed on conventional concrete, the concrete quality seems worse than it is.

There are also some relations existing in the literature that have been offered to evaluate the results of the two non-destructive tests, Schmidt hammer test and the ultrasonic pulse velocity test, simultaneously [78]. However since neither Schmidt hammer test nor the ultrasonic pulse velocity test could have provided reasonable evaluations alone, due to the probable reasons discussed above, there is no need to try to evaluate them together.

CHAPTER 5

CONCLUSIONS

A new method of making PAC has been investigated. For this purpose, twelve cubes of concrete, each with a volume of 1 m^3 and with different compositions of cementitious material, have been prepared; seven of which as conventional PAC, four as PAC prepared by the new method, and one as conventional concrete. The specimens have been examined from thermal and mechanical properties points of view. As a result of the experiments carried out, the following conclusions have been achieved:

1. It is known that in laboratory studies made on mass concrete by thermocouples, using small sized specimens do not give representative results, since the inner and the outer temperatures get easily balanced and significant temperature differences can not be observed. In this study, the size of the cubic specimens have been chosen as 1 m^3, and the test results about measurement of concrete temperatures has shown that such a specimen size is quite adequate for mass concrete studies.

2. The maximum peak temperature difference has been obtained at the specimen PAC_1^2 as 20.5 ^{0}C. However, even this value does not indicate a significant risk of thermal cracking; because it is reported in the literature that concrete can stand a 20 ^{0}C drop in temperature without cracking [13]. On the other hand, this specimen contains only PC as cementitious material; which means that, this value can be

decreased furthermore by replacing some portion of this cement content by pozzolans. Thus it can be concluded that PAC method is very effective in fighting against the risk of thermal cracking. As it is mentioned in Chapter 1, there are two main ways in fighting against the risk of thermal cracking; one of which is to reduce the initial temperature of concrete, the other is to control the heat of hydration evolution; but according to the literature, reducing of the initial temperature of concrete has been found to be much more effective in fighting against the risk of thermal cracking than controlling of the heat of hydration evolution [13]; and since the advantage of PAC method in fighting against the risk of thermal cracking appears in reducing of the initial temperature of concrete, it can be seen that this conclusion is quite consistent with the literature.

3. Recent studies show that, use of brick powder as mineral admixture in concrete improves the durability of concrete significantly [59-61]. As a result of this study, it can be added to this conclusion that, use of brick powder also reduces the risk of thermal cracking; because, among both PAC^{1} and PAC^{2} specimens, the minimum peak temperature differences have been obtained at the specimens which contain brick powder as mineral admixture. Thus it can be concluded that brick powder is a very suitable mineral admixture to be used in mass concrete.

4. On the other hand, among the specimens that contain mineral admixtures, the maximum peak temperature difference has been obtained at the specimen PAC^{1}_{4}, which contains GGBFS as mineral admixtures. Thus it can be concluded that GGBFS is not a suitable mineral admixture to be used in mass concrete.

5. All the PAC[1] specimens have provided quite low temperature differences, and in general they have provided compressive strength values which are comparable to that of conventional concrete. However their modulus of elasticity values are much lower comparing to that of conventional concrete. Although this is quite consistent with the literature, where it is reported that the modulus of elasticity of PAC is found to be slightly higher than half that of conventional concrete [9,10,11], it does not change the fact that this is a disadvantage of PAC when compared with conventional concrete from mechanical properties point of view.

6. On the other hand, the PAC[2] specimens have provided compressive strength and modulus of elasticity values both comparable to that of conventional concrete, besides providing low temperature differences that are quite acceptable for mass concrete. Thus it can be concluded that the disadvantage of PAC which is observed in modulus of elasticity when compared with conventional concrete from mechanical properties point of view, is not observed at the PAC specimens which have been prepared by the new method.

7. The interpretation of the non-destructive test results according to the relations offered in the literature could not have provided reasonable results for the concrete specimens prepared in the scope of this research. While the reason of this incompatibility for the Schmidt Hammer test was more closely related to the mix designs of the specimens, for ultrasonic pulse velocity test, it was more closely related to the method of preparation of the specimens; but no matter what the reason of the incompatibility is, it has been concluded that proposal of new relations for the interpretation of the non-destructive tests carried out on such specimens is definitely required.

CHAPTER 6

RECOMMENDATIONS

Although the subject of this study seems as PAC in general, as the title implies, this was rather a study on mass concrete in particular. Since it is required to work on specimens, which are much bigger than usual, in laboratory studies on mass concrete, the study becomes much more troublesome and time consuming. Because of this, although the subject is very comprehensive, it was not possible to study all the topics in plan in a limited time period. Thus the following recommendations can be given for a further relevant study:

1. In this study, it was worked on 1 m sized cubic specimens. Thus it was not possible to produce so many specimens under limited laboratory conditions. Because of this, in order to check more factors on a limited number of specimens, sometimes more than one factor had to be changed at the same time. However this may cause inconvenience while making comparisons among the specimens. Thus it can be recommended to repeat a similar study on a larger quantity of specimens, changing only one factor at each time.

2. Since the major topic was mass concrete, it was mainly concentrated on thermal properties of the specimens; but as thermal properties, only the resulting concrete temperatures were taken in to account. The other thermal properties of concrete that affect the resulting concrete temperatures; such as coefficient of thermal expansion, specific heat, thermal conductivity and thermal diffusivity

are not studied separately. Thus it can be recommended to make a more comprehensive study from thermal properties point of view.

3. While examining the specimens from resistance to thermal cracking point of view, only the difference between the inner and the outer temperatures of the concrete was taken into account; and if this temperature difference is under certain limits, the specimen has been regarded as resistant to thermal cracking. However no micro study has been carried out in order to check whether this is the case in reality or not. Thus, supporting of the investigation by micro study can be noted as another recommendation.

4. Beside the thermal properties, some of the mechanical properties were also examined, one of which was compressive strength; and since it was already known how long the specimens have been cured at what temperatures, the maturity concept could have also been studied with the available data.

5. In this study, a new method for making PAC has been investigated. However it was concentrated on only mass concrete applications. As a last recommendation, the study of this new method for other applications can be added.

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