A STUDY OF THE EFFECTS OF BLASTING VIBRATION ON GREEN CONCRETE

GEO REPORT No. 102

A.K.H. Kwan & P.K.K. Lee

GEOTECHNICAL ENGINEERING OFFICE
CIVIL ENGINEERING DEPARTMENT
THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. A charge is made to cover the cost of printing.

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R.K.S. Chan
Head, Geotechnical Engineering Office
May 2000

FOREWORD

With the fast-track construction in Hong Kong, programme and operational constraints often call for blasting to be carried out in close proximity to freshly cast concrete structures. This has raised concerns among the industry on the possible adverse effects of blasting vibration on the integrity of 'green' concrete.

In June 1994, the Geotechnical Engineering Office engaged Mr P K K Lee and Dr A K H Kwan of the University of Hong Kong to undertake a research project with an aim to providing data for the development of a rational framework and methodology for assessing the effects of blasting on green concrete and for setting up appropriate vibration control criteria for use in Hong Kong. This Report documents the experimental work carried out under the project.

P.L.R. Pang
Chief Geotechnical Engineer/Special Projects

EXECUTIVE SUMMARY

In order to study the effects of blasting vibration on concrete including freshly placed concrete (green concrete) so that blasting vibration control limits may be established to avoid causing damage to concrete structures, a literature review and an experimental programme have been conducted.

The findings of the literature review have been reported separately in the Literature Review Report submitted in October 1994. Basically, very large discrepancy exists in the vibration control limits currently adopted by different building authorities. There have been relatively few rigorous studies on the possible effects of blasting vibration on concrete and up to now the vibration resistance of concrete has never been directly measured. Nevertheless, putting the scanty results of previous studies together, it may be concluded that when considering the effects of blasting vibration on concrete, concrete structures should be classified into unrestrained structures (aboveground structures) which are free to vibrate and would respond like being attacked by an earthquake and restrained structures (underground structures) which are forced to deform with the surrounding soil. The response of an unrestrained structure depends on the structural form and thus dynamic analysis is required to evaluate the effects of blasting vibration on it. On the other hand, since restrained structures would deform with the same strain as that of the ground, the effects of blasting vibration are dependent on the ground strain produced by the propagation of the shock wave through the ground. Regarding the estimation of vibration resistance of concrete, most engineers assume that the vibration resistance of concrete is proportional to the strength of the concrete but it is not quite clear whether the strength should be compressive strength or tensile strength. The situation is rather confusing and to date no rational framework exists for establishing vibration control limits to avoid causing blasting damage to nearby concrete structures.

The experimental programme involved the testing of 6 typical concrete mixes and 1440 concrete specimens. A method of testing concrete for their shock vibration resistance by subjecting prisms cast of the concrete to hammer blows was developed. Vibration intensities up to a peak particle velocity of 3000 mm/s have been applied. After hammering, the prismatic specimens were continued to be cured until the age of 28 days and then tested for their compressive and tensile strengths. It was found that even at intensities high enough to break the concrete specimen into pieces, the shock vibrations applied have little effect on the compressive strength of concrete. Vibration damages were mainly in the form of cracking and reduction in tensile strength. Hence, the vibration resistance of concrete is dependent more on tensile strength than compressive strength. The vibration resistance results (in terms of peak particle velocity of vibration) were quite scattered but the large number of experimental data allowed us to determine statistically the relationship between the lower bounds or characteristic values of the shock vibration resistance to concrete age, compressive strength, tensile strength and ultrasonic wave velocity with fairly good correlation coefficients. This is the first time that the vibration resistance of concrete is correlated to other mechanical properties. Comparison of the vibration resistance values of different types of concrete with or without PFA and cured at different temperatures revealed that the correlation of vibration resistance to other mechanical properties is not dependent on PFA content or curing temperature. Thus, a single set of vibration control criterion may be applied to concretes within the range studied herein. From these vibration resistance results, vibration control limits that may be applied to Hong Kong and perhaps also to other places

may be obtained by dividing the vibration resistance values by a suitable factor of safety. In view of the large variation of vibration resistance results, a factor of safety of 5 is recommended. Despite the use of a fairly large factor of safety, the vibration control limits so derived are higher than most existing limits currently being used in actual construction. Thus if the vibration control limits derived in the present study are adopted, higher intensity blasting and therefore quicker and more economical construction would be allowed.

It should, however, be borne in mind that so far no field testing has been carried out. Before the recommended vibration control limits are applied, it should be prudent to first conduct field tests to confirm the safety of their use. Soil type and possible soil-structure interaction should be included in further studies.

CONTENTS

1. INTRODUCTION

This research study was carried out by The University of Hong Kong for Geotechnical Engineering Office, Civil Engineering Department of the Hong Kong Government. It was initiated by Special Projects Division of the Geotechnical Engineering Office and was launched in June 1994 after Agreement No. CE21/94 was signed between The University of Hong Kong and the Hong Kong Government.

The scope of the study was specified in the Brief of Agreement No. CE21/94. According to Clause 2 of the Brief, the aim of the study is to develop a rational framework or methodology for assessing the effects of blasting on green concrete and for setting up appropriate control criteria in Hong Kong. The study consists mainly of the following three parts:

(1) a literature review covering the presently available data, existing methods of assessing the effects of vibration on green concrete, and vibration control criteria being adopted in Hong Kong and elsewhere;
(2) an experimental programme of subjecting typical concrete used in Hong Kong to high intensity vibrations to evaluate their vibration resistances at different ages and investigate the short/long term effects of vibration on concrete; and
(3) a theoretical study to correlate the vibration resistance of a concrete to its mechanical properties and develop a method of setting up appropriate vibration control criteria based on the known properties of concrete.

As required by Clauses 5.1 and 5.2 of the Brief, an Inception Report on the proposed method of study, detailed testing programme and schedule of works had been submitted in July 1994. Subsequently, the study was carried out to completion following generally the method outlined in the Inception Report except for some minor details which are explained in relevant parts of this report.

As originally scheduled in the Inception Report, the study first started with the literature review. After three months of study, a 165-page Literature Review Report No.1 was completed and submitted in accordance with Clause 5.3 of the Brief in October 1994. Although the literature review was substantially completed after the submission of the report, it is being continued till the end of the Assignment so that the most updated information on the topic can be included. Further findings of the literature review will be reported in the Literature Review Report No. 2 which will be submitted separately.

Preparation for the laboratory tests started in August 1994. A number of testing equipment, including 15 steel moulds for concrete prisms, a specially designed hammering tester with an automatic hammer release device, a high-g (high acceleration range) accelerometer together with its conditioning amplifier, an A/D conversion card for vibration data acquisition, an electronic stiffening time tester, a direct tension testing equipment and a 486-PC computer for controlling the hammer release operation and the data acquisition process, were purchased/fabricated. By January 1995, all the necessary equipment were acquired and ready for use.

However, due to the lack of a suitable research assistant to help with the laboratory works, the tests did not start until June 1995. Two Research Assistants, Mr. G.L. Ni and

Mr. W. Zheng, both from Guangdong Provincial Research Institute of Water Conservancy and Hydro-power, had been employed to perform the actual testing. Together, they have contributed twelve man-month's of hard work towards the successful completion of the experimental programme. In total, 720 concrete prisms, 432 concrete cubes and 288 concrete cylinders have been cast and tested. All the testing works were carried out in Hong Kong using local materials under the supervision of Dr. A.K.H. Kwan and Mr. P.K.K. Lee, both Senior Lecturers of Department of Civil and Structural Engineering, The University of Hong Kong. Details of the experimental programme, the raw and processed data, theoretical studies carried out, the findings arrived at and the recommendations made from the work are all presented in this Final Report, which is written jointly by Dr. A.K.H. Kwan and Mr. P.K.K. Lee. All the data processing, graph plotting and drawings in the report were done by Mr. W. Zheng, who has spent three whole months to help completing this report.

2. BACKGROUND OF STUDY AND RESEARCH SIGNIFICANCE

With the fast-track construction in Hong Kong, programme and operation constraints often call for blasting to be carried out in close proximity to freshly cast concrete piles and structures. Ground vibration caused by blasting may induce cracks and may have other adverse effects on the green concrete. This has raised concern among the construction industry the possible adverse effects of blasting vibration on the integrity of green concrete.

The construction industry in Hong Kong tends to adopt very restrictive limits on the maximum ground vibration induced by blasting on a freshly cast concrete structure. The limits are generally based on experience and limited available literature from overseas countries. However, site conditions in those countries may not be strictly applicable locally in Hong Kong. Therefore, there is a need to carry out a comprehensive research study to assess the effects of blasting vibration on green concrete.

"Green concrete", which may be taken to mean freshly placed concrete or more specifically concrete having an age of less than 24 hours (Hulshizer and Desai 1984), is particularly vulnerable to weakening of its properties if subjected to high intensity shock vibration that could disrupt the concrete matrix during the formative bond development process. The possible effects of shock vibration on curing concrete, especially freshly placed concrete, are very complicated and up to now, little is known about these effects. This is reflected in the large differences in the permitted peak particle velocity specified by different building authorities.

Nevertheless, most engineers accept that fully hardened concrete can withstand shock vibration up to a peak particle velocity of around 100 mm/s without detrimental effects (Wiss 1981). It is also often assumed that the resistance of "curing concrete" (concrete that has not yet fully developed its strength) to shock vibration is proportional to its strength and thus the permitted peak particle velocity may be taken as proportional to the strength of the concrete. Based on this principle, Gamble and Simpson (1985), Bryson and Cooley (1985), and Olofesson (1988) established their criteria for permitted peak particle velocity as in Table 2.1.

Table 2.1 - Permitted peak particle velocity commonly adopted for concrete with ages of more than 12 hours

References	Permitted peak particle velocity at different ages (mm/s)						
	12 hrs	1 day	2 days	3 days	7 days	28 days	90 days
Gamble and Simpson, 1985	5	10	20	30	50	100	100
Bryson and Cooley, 1985	5	50	50	50	100	100	100
Olofesson, 1988 - curing at 5ºC	-	-	8	11	35	80	100
Olofesson, 1988 - curing at 21ºC	-	14	30	40	60	85	100

Table 2.2 - Permitted peak particle velocity commonly adopted for green concrete (concrete with ages of less than 24 hours)

References	Permitted peak particle velocity at different times after placement (mm/s)						
	0-2 hrs	4 hrs	8 hrs	12 hrs	18 hrs	24 hrs	>24 hrs
Gamble and Simpson, 1985	-	2	3	5	8	10	10
Bryson and Cooley, 1985	5	5	5	5	5	5	50
Olofesson, 1988 - curing at 5ºC	100 (0-10 hrs)			No blasting within 30 m (10-70 hrs)			
Olofesson, 1988 - curing at 21ºC	100 (0-5 hrs)		No blasting within 30 m (5-24 hrs)				14

Since the strength of concrete is relatively low during the first 24 hours after casting, blasting is usually greatly restricted near green concrete. On the other hand, however, it has also been suggested that within the first few hours before initial set, the freshly placed concrete should be able to withstand shock vibration up to 100 mm/s (Olofesson 1988). The permitted peak particle velocity commonly adopted for green concrete are shown in Table 2.2. From the tabulated values, it can be seen that very large discrepancies exist between the various recommended vibration limits, especially those being applied to concrete cast within a few hours.

As described by Dowding (1985), an interesting experiment had been carried out by Esteves in 1978. The experiment revealed the surprising result that although there is a period of greater susceptibility to vibration cracking between 10 to 20 hours after placement of the concrete, the threshold vibration for cracks to appear during this period is as high as 150 mm/s. Hulshizer and Desai (1984) had also carried out a testing program, which included both laboratory and field tests, to evaluate the effects of high intensity shock vibrations on green concrete. It was found that although many concrete specimens were subjected to shock vibrations up to and in the range of 200 to 250 mm/s, there was no evidence to indicate that the vibrated concrete would not perform like un-vibrated concrete. Accordingly, they suggested fairly high vibration limits of 100 mm/s, 38 mm/s, 50 mm/s, 100 mm/s and 175 mm/s for concrete cast in less than 3 hours, within 3 - 11 hours, within 11 - 24 hours, within 1 - 2 days and over 2 days respectively. Thus, on one hand, the first 24 hours after concrete placement is generally regarded as the most critical period and engineers are extremely cautious in controlling nearby blasting during this period, on the other hand, test results have indicated that the green concrete can, in fact, withstand fairly high intensity shock vibration during the first few hours after casting. More research studies are needed in order to clarify the situation.

Relatively speaking, the case for concrete placed more than 24 hours is more straight forward. However, it is not without problems. Firstly, although the assumption that the peak particle velocity the curing concrete can withstand is proportional to the strength of the concrete is quite commonly adopted, it has never been testified by any experimental results. It is also not clear whether the strength should be the compressive strength or the tensile strength of the concrete. Since the most likely damage that will be caused by the shock vibration is cracking, the tensile strength should be more relevant as a measure of vibration resistance. However, it seems that most engineers are still using the compressive strength for specifying the vibration limits. Secondly, there is also the problem that the maximum stress induced by the shock vibration is not really proportional to the peak particle velocity. According to Dowding (1984), the maximum dynamic strain ε_d induced by the shock vibration is given by:

$$\varepsilon_d = \frac{v}{c} \qquad ...(2.1)$$

where v is the peak particle velocity and c is the wave propagation velocity. Multiplying by the dynamic modulus E_d, the maximum stress induced by the shock vibration is obtained as:

$$\sigma = E_d \frac{v}{c} \qquad ...(2.2)$$

Vibration cracking occurs when the maximum stress exceeds the dynamic tensile strength f_{td} of the concrete, i.e.

$$E_d \frac{v}{c} > f_{td} \qquad ...(2.3)$$

From this inequality, the threshold peak particle velocity v' for vibration cracking to occur can be evaluated as:

$$v' = \frac{c}{E_d} f_{td} \qquad ...(2.4)$$

Hence, the peak particle velocity that the curing concrete can withstand is not really just a function of the tensile strength of the concrete. In determining the shock vibration resistance of the concrete, it is necessary to take into account also the wave propagation velocity and dynamic modulus of the concrete. As a matter of course, tests are required to verify the above arguments. Finally, the rate of strength development is dependent on many factors, e.g. mix composition and curing temperature, and thus it would be too simplistic to establish just a single set of vibration limits regardless of the mix composition and curing temperature. As concrete mixes containing PFA (pulverized fuel ash) may develop their strength at much slower rates than those without, it is necessary at least to distinguish between PFA concrete and non-PFA concrete. Tests on different types and grades of concrete under a curing temperature representative of the Hong Kong conditions are necessary so that the actual rates of strength development of the different concrete can be taken into account in establishing the vibration limits.

In this study, a series of tests on concrete mixes of different grades, with or without PFA and at different ages has been carried out to determine their shock vibration resistance. Although the Brief asked for study on only green concrete (concrete having an age of less than 24 hours), concrete with ages of more than 24 hours but less than 28 days were also included in the study because it was felt that concrete at such ages might also be affected by vibration. The effects of the shock vibration applied to the concrete were determined by continuing to cure the concrete after they were subjected to the shock vibration until the age of 28 days and testing the concrete for their tensile and compressive strengths. In order to correlate the vibration resistance of the concrete to their mechanical properties, they were also tested for their mechanical properties if possible at the time of vibration test. As the vibration resistance of a green concrete was expected to be quite sensitive to its rate of stiffening, stiffening time tests of the concrete were also carried out. After all the tests were completed, the vibration resistances of the various grades and types of concrete at different ages were evaluated and then correlated to their respective mechanical properties and stiffening times to see whether a theory for estimating the vibration resistance of concrete could be developed.

3. EXPERIMENTAL PROGRAMME

3.1 Concrete Mixes Tested

The concrete mixes tested are presented in Table 3.1 while the mix proportions of the concrete mixes derived by trial mixing are listed in Table 3.2. They ranged from Grades 20 to 40, contained either no or 25% PFA, but were all designed to have a slump of 75 mm. All the concrete mixes were made using local materials. The same sources of materials were used throughout the study so as to maintain consistency.

Table 3.1 - Concrete mixes tested

Mix no.	Grade	Slump (mm)	PFA content
G20/0PFA	20	75	0%
G20/25PFA	20	75	25%
G30/0PFA	30	75	0%
G30/25PFA	30	75	25%
G40/0PFA	40	75	0%
G40/25PFA	40	75	25%

Table 3.2 - Mix proportions of the concrete

Mix no.	Water/binder ratio	OPC content (kg/cu.m.)	PFA content (kg/cu.m.)	Fine to total aggregate ratio	Superplasticizer (litre/cu.m.)
G20/0PFA	0.65	308	-	0.40	0.74
G20/25PFA	0.62	242	81	0.40	0.50
G30/0PFA	0.55	364	-	0.35	1.10
G30/25PFA	0.53	283	94	0.35	0.92
G40/0PFA	0.45	444	-	0.30	1.50
G40/25PFA	0.43	349	116	0.30	1.10

In order to study the effects of the curing temperature, two batches of concrete, one cured at a designated curing temperature of 20ºC and the other at a designated curing temperature of 30ºC, were produced from each concrete mix. The two batches of concrete, though cast of the same mix proportion, are treated as different concretes and assigned different batch numbers as shown in Table 3.3.

Table 3.3 - Concrete batch numbers

Batch no.	Mix no.	Curing temperature
G20/0PFA/20C	G20/0PFA	20ºC
G20/0PFA/30C	G20/0PFA	30ºC
G20/25PFA/20C	G20/25PFA	20ºC
G20/25PFA/30C	G20/25PFA	30ºC
G30/0PFA/20C	G30/0PFA	20ºC
G30/0PFA/30C	G30/0PFA	30ºC
G30/25PFA/20C	G30/25PFA	20ºC
G30/25PFA/30C	G30/25PFA	30ºC
G40/0PFA/20C	G40/0PFA	20ºC
G40/0PFA/30C	G40/0PFA	30ºC
G40/25PFA/20C	G40/25PFA	20ºC
G40/25PFA/30C	G40/25PFA	30ºC

For the sake of quality control, samples were taken from each batch of concrete for workability tests and cube strength tests. The workability tests included slump test, compacting factor test and V-B time test. For the cube strength tests, three cubes were cast. The cubes were cured under a standard temperature of 27ºC (note that this curing temperature is not the same as the designated curing temperature for the other test specimens which were cured at either 20ºC or 30ºC) as required by the Hong Kong Construction Standard CS1:1990, and tested at the age of 28-day.

3.2 Tests Carried Out

For each batch of concrete, the following laboratory tests have been carried out:

(1) Hammering tests - Prisms cast of the concrete mix and cured under the designated curing temperature were subjected to hammer blows of different intensity, and the peak particle velocity required to fracture the concrete at various times after casting were measured.

(2) Stiffening time tests - The penetration resistance of the mortar fraction sieved from the concrete mix at various times after mixing were measured and hence the stiffening times of the concrete mix determined so that the change in vibration resistance of the concrete during the early ages could be related to the stiffening time.

(3) Material property tests - The compressive strength, tensile strength, dynamic modulus, static modulus and longitudinal wave propagation velocity of the concrete at the time

of performing hammering tests were measured if the concrete had hardened sufficiently for the moulds to be removed (normally, the moulds could be removed at 12 hours after casting) so that the vibration resistance of the concrete might be related to these parameters.

(4) Integrity tests - The concrete prisms which had been subjected to different intensity of hammer blows were tested for their 28-day compressive and tensile strengths so as to evaluate the long term effects of the shock vibration on the integrity of the concrete.

The hammering tests on the concrete were carried out at the following ages:

2 hrs., 4 hrs., 6 hrs., 8 hrs., 10 hrs., 12 hrs.,
18 hrs., 24 hrs., 3 days, 7 days, 28 days.

For concrete more than 12 hours old, the moulds of the concrete prisms were first removed before performing any tests. After the moulds were removed, ultrasonic tests were carried out on the concrete prisms to measure the longitudinal wave propagation velocity and dynamic modulus of the concrete. Then the hammering tests were performed by subjecting the concrete prisms (without their moulds) to hammer blows. At the time of performing the hammering tests, the cubes and cylinders cast of the same concrete and cured under the same conditions were also tested to determine the compressive and tensile strengths of the concrete. However, since it was difficult to de-mould concrete specimens within 12 hours after casting, freshly placed concrete with ages of less than 12 hours were tested with their moulds on. Thus hammering tests on concrete placed within 12 hours were carried out by keeping the concrete in their moulds and subjecting the concrete together with their moulds to the hammer blows. As the freshly placed concrete were still colloidal, their stiffness or strength were not measured. Instead, the stiffening time of the concrete was measured so that the change in vibration resistance of the concrete as the concrete hardens could be related to the time when the concrete changed its state.

In order to deal with the situation that concrete with ages of less than 12 hours and those with ages of more than 12 hours had to be treated differently, two separate series of tests were carried out on each batch of concrete. The first series of test was on concrete less than 12 hours old, while the second series was on concrete more than 12 hours old. The tests carried out in each series of test are listed below:

Series 1:

Quality control tests;
Hammering tests (with moulds on) at:
2 hrs., 4 hrs., 6 hrs., 8 hrs., 10 hrs., 12 hrs.;
Stiffening time tests on the fresh concrete;
Integrity tests at 28-day.

Series 2:

Quality control tests;
Hammering tests (with moulds removed) at:
12 hrs., 18 hrs., 24 hrs., 3 days, 7 days, 28 days;
Material property tests at time of performing hammering tests;
Integrity tests at 28-day.

Basically, for each age at which hammering tests were to be carried out, five prisms were cast and cured at their designated curing temperature of either 20ºC or 30ºC until the time for the hammering tests. One of the five prisms was treated as a control specimen that would not be subjected to any hammer blow but would be cured alongside the others for future comparison testing. Among the other four prisms, one was subjected to a number of hammer blows with increasing intensity until the first crack appeared in order to measure the vibration intensity at which the concrete cracked and study the cumulative effects of hammering vibrations on the concrete. The remaining three prisms were each subjected to one hammer blow of different intensity to measure the peak particle velocity required to cause cracking of the concrete. As far as possible, the hammer blows were applied to the three prisms in such a way that at least one but not more than two of the prisms would be cracked. However, due to the erratic nature of vibration resistance, quite often none of the three prisms which had each been subjected to one hammer blow cracked. In order to produce cracking so that the vibration intensity necessary to cause cracking may be determined, some of the prisms were re-hammered (i.e. subjected to more than one hammer blows) until at least one of them cracked. After the hammering tests, the four prisms which had been subjected to the hammering were continued to be cured together with the control specimen at their designated curing temperature. At the age of 28-day, all the prisms were tested for their tensile and compressive strengths so as to evaluate the long term effects of the hammer blows on the integrity of the concrete.

In addition to the above, for Series 1 tests, one fresh concrete sample was taken from each batch of concrete for stiffening time test. The stiffening time test was carried out on the mortar fraction of the concrete which was separated from the fresh concrete specimen by sieving and then kept at the same temperature as the designated curing temperature of the concrete (i.e. either 20ºC or 30ºC) at all times until the test was finished. The penetration resistance of the mortar was measured using a stiffening time tester at half-hourly intervals until the mortar has completely stiffened. The respective stiffening times to reach penetration resistances of 0.5 and 3.5 MPa were determined by plotting the penetration resistance against time and reading the time values corresponding to 0.5 and 3.5 MPa penetration resistance from the graph.

Apart from the prisms required for hammering tests, in the Series 2 tests, three cubes and three cylinders were also cast for each age at which the hammering tests were carried out. The cubes and cylinders were placed alongside the prisms during curing so that they were cured under exactly the same conditions as for the prisms. At the time of performing the hammering tests on the prisms, the three cubes were crushed to determine their compressive strength and the three cylinders split to determine their split cylinder tensile strength. Ultrasonic tests were also carried out on the four concrete prisms prior to applying the hammer blows to them to measure the longitudinal wave propagation velocity and dynamic modulus of the concrete. In addition, for measurement of static modulus, one extra cylinder was cast from each batch of concrete. It was cured under the same condition as for the other specimens and its static modulus measured at the time of performing hammering tests.

More details of the above tests are given in the following sub-sections.

3.2.1 <u>Details of the Hammering Tests</u>

There is no standard test method for measuring the vibration resistance of concrete. Among the existing methods, the simplest and yet most direct way of measuring the vibration resistance of concrete is to subject concrete prisms to hammer blows in the longitudinal direction to induce longitudinal wave stresses along the prisms and then observe the formation of cracks on the concrete surfaces. This hammering test method, with a set up as shown in Fig.3.1, was adopted in this study.

The dimensions of the prisms were 150 mm × 150 mm × 750 mm long which are the same as those of a standard test beam specified in BS1881: Part 118: 1983 for flexural test and equivalent cube test. The reasons for adopting such standard beam size were that this allowed the standard beam moulds to be used in the present study and the prisms to be tested for their integrity in accordance with BS1881: Part 118.

The hammer was mounted on a swing arm for hitting one end of the test prism perpendicularly. Its weight was adjustable between a minimum weight of 7 kg to a

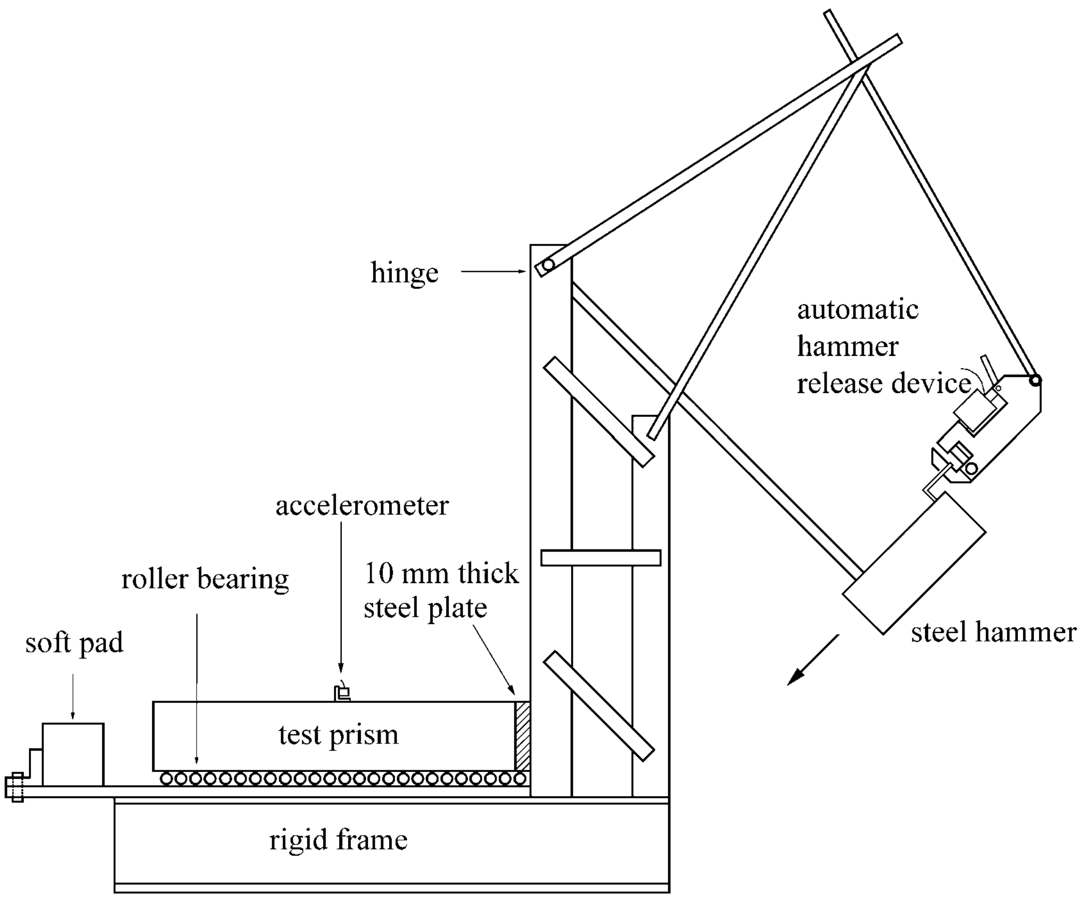


Figure 3.1 - Schematic of set up for hammering test

maximum weight of 36 kg and its height when it was released was also adjustable from 200 mm to 800 mm above the striking point. Hence by varying the weight and/or the height of the hammer when released, the impact energy to be applied to the test prism could be adjusted between 14 to 280 Joules. To ensure that the hammer blow impact was applied uniformly to the cross-section of the test prism, a steel plate of 10 mm thick was attached to the end of the prism to be struck. Concrete cast within 12 hours were tested with their moulds on while concrete cast more than 12 hours were tested with their moulds removed. The concrete prisms were mounted on roller bearings so that they were free to move longitudinally when subjected to hammer blows. Soft rubber pads were fixed at the far end of the testing frame to restrain the prisms from falling off the testing frame after being subjected to hammer blows. The whole hammering operation was controlled by a 486-PC computer which was also used for data acquisition. Before the hammering operation, the hammer was lifted manually and hooked to an automatic hammer release device. At the trigger of an electronic signal from the controlling computer, the hammer was released from the hook of the automatic hammer release device and the data acquisition process started. This ensured that the hammering operation and the data acquisition process were synchronized. Details of the hammer release device are shown in Fig.3.2.

An accelerometer was attached to the test prism to measure the intensity and frequency spectrum of the shock vibration induced by the hammer blow. To fix the accelerometer to the test prism, a perspex seating was first glued onto the top surface of the prism and then the accelerometer attached to the perspex by a small screw. Three different locations for fixing the accelerometer, at the striking end, at mid-length and at the far end, had been tried but it turned out that the mid-length location gave the best results. Added with the consideration that any cracks formed were usually near the centre of the prisms, it was decided to attach the accelerometer at mid-length of the prism, as shown in Fig.3.3. After the application of each hammer blow to the test prism, the prism was inspected for any transverse cracks formed on the concrete.

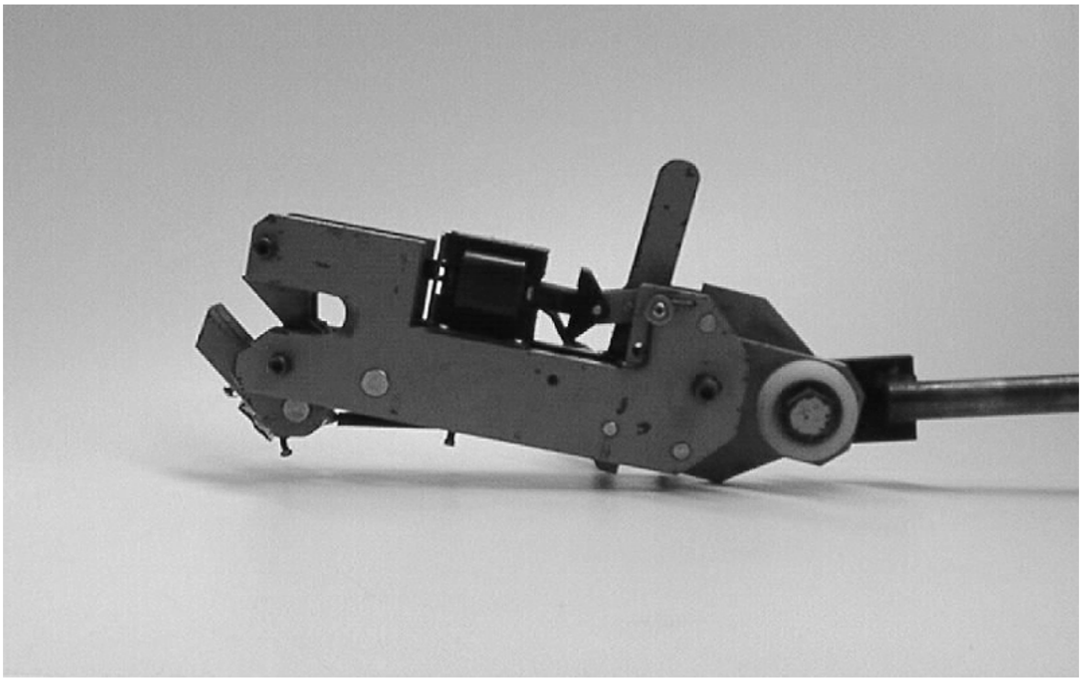


Figure 3.2 - The hammer release device

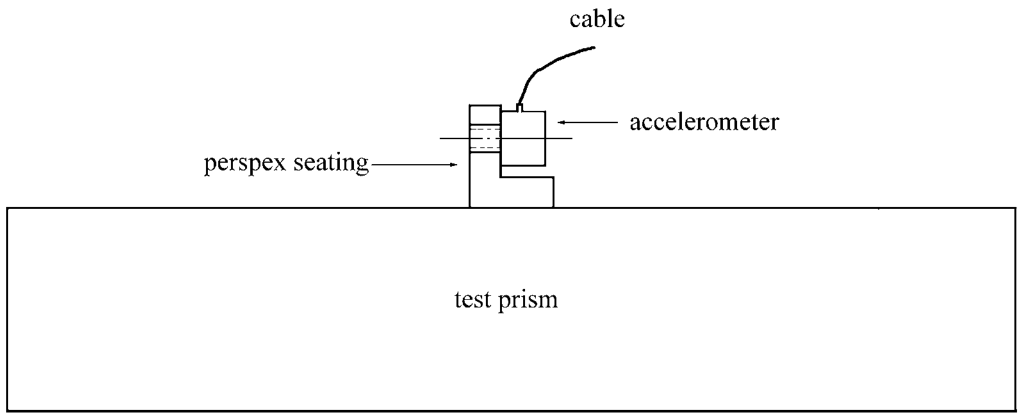


Figure 3.3 - Attached position of the accelerometer

All prisms were continued to be cured under their respective designated curing temperatures after the hammering tests. In order to make sure that any cracks formed by shock vibration during the hammering tests were closed so as to allow for any possible self-healing effects, the prisms were cured with their longitudinal axis oriented vertically so that the cracks were kept closed under the self weight of the prisms. At the age of 28-day, the prisms were each tested for their compressive and tensile strengths to evaluate the long term effects of the shock vibration on the integrity of the concrete (for details of the integrity tests, refer to Section 3.2.3).

The accelerometer used was a Bruel & Kjaer type 8309s piezoelectric high frequency response, high-g (100,000g) shock vibration accelerometer. It was connected to a Bruel & Kjaer type 2626 signal conditioning amplifier which drove the accelerometer and gave analogue output for the acceleration measurement. The analogue output from the conditioning amplifier was fed into a 12-bit A/D converter installed inside the controlling computer which converted the acceleration signal into digital form at a rate of 100 kHz. The digital signals from the A/D converter were first stored into a hard disk and then processed using a software called GLOBAL LAB. In order to obtain the peak particle velocity of the shock vibration, the acceleration results were numerically integrated with respect to time to yield the particle velocity at the point of measurement. The particle velocity so derived consisted of two components: the velocity of the rigid body movement of the test prism and the vibration velocity of the longitudinal shock wave, as illustrated in Fig.3.4. As the rigid body movement of the prism would not induce any stress, it had no effect on the concrete and should thus be removed. A trend removal process was used to remove the rigid body movement component (the trend of the particle velocity results was actually the rigid body movement of the prism) from the particle velocity results so as to yield the pure wave velocity component. It was this pure wave velocity component that induced cyclic tensile-compressive stresses in the concrete prism and caused cracking or damage of the concrete. The pure wave velocity component so obtained generally showed the pattern of a typical attenuating wave with a peak amplitude occurring within the first one or two cycles of vibration and a gradually decreasing wave amplitude with time. From the pure wave velocity component, the peak particle velocity of the shock wave was determined and taken as a measure of the intensity of the applied shock vibration.

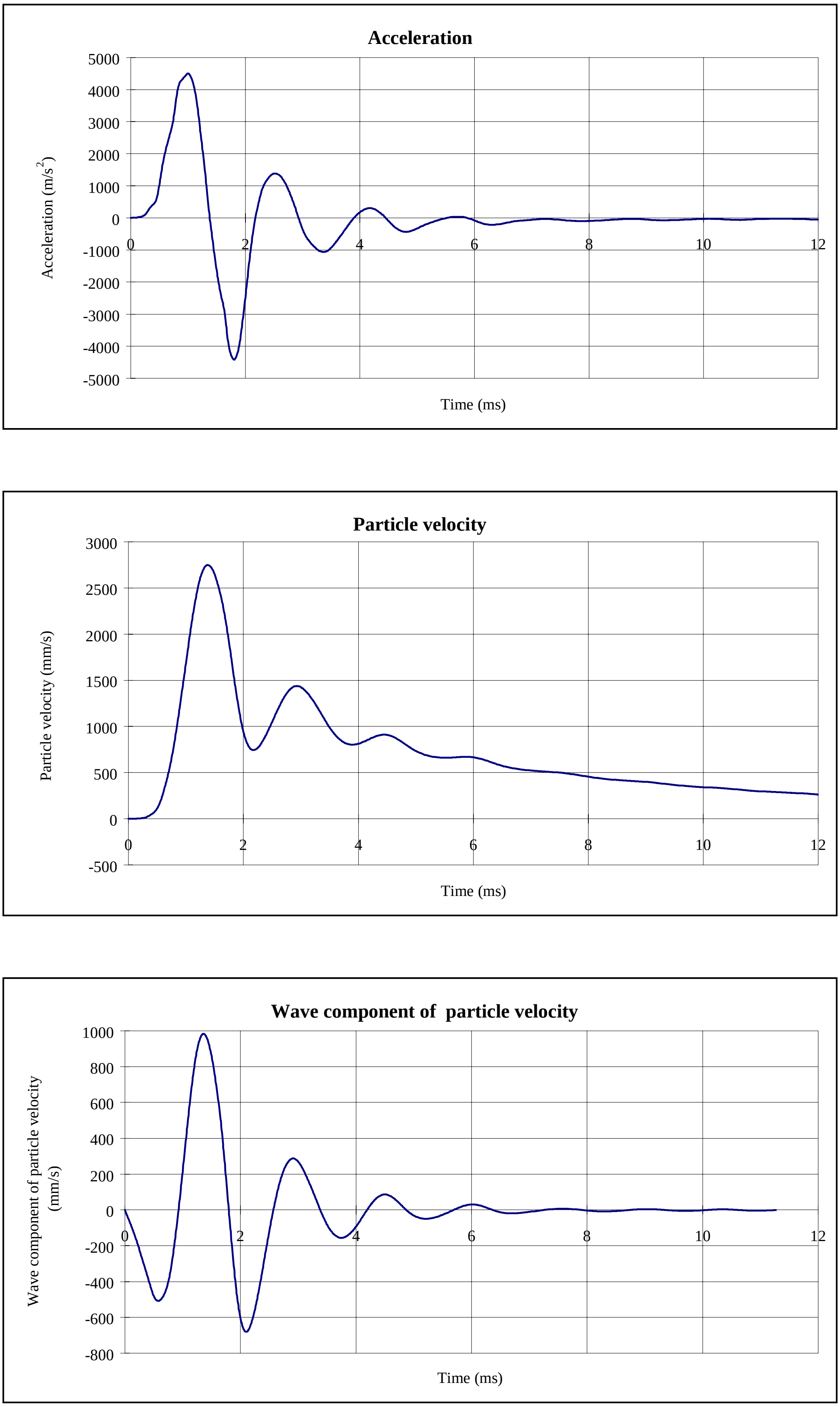


Figure 3.4 - Obtaining particle velocity of shock wave from acceleration result

3.2.2 Details of the Stiffening Time Tests

The stiffening time tests were carried out in accordance with Appendix C of BS5075: Part 1: 1982. They were for the determination of the time after mixing of the concrete for the mortar fraction sieved from the concrete mix to reach penetration resistances of 0.5 MPa and 3.5 MPa. According to BS5075, the time for the mortar fraction to reach a penetration resistance of 0.5 MPa corresponds approximately to the extreme limit for placing and compacting concrete, while the time to reach a penetration resistance of 3.5 MPa gives a guide to the time available for the avoidance of cold joints. Although the stiffening time measurements are originally intended to be used for scheduling of concreting operations, it is felt that they might also be useful for setting vibration limits at various times after placing of the concrete.

As the curing temperature could significantly affect the stiffening time of the concrete, the mortar samples were kept at all times under the designated curing temperature of either 20ºC or 30ºC, i.e., the same curing temperature as for the concrete prisms to be subjected to hammering tests.

The stiffening time tester used consisted of a rig for lowering a pin of dimension specified by the British Standard vertically into a can of mortar and an electronic balance of capacity 50 kg for measuring the penetration resistance of the mortar.

3.2.3 Details of the Integrity Tests

After the hammering tests, all prisms, including those which had been subjected to hammer blows and the control specimens which had not been subjected to any vibration, were continue to be cured at their designated curing temperature until the age of 28-day when they would be tested for their tensile and compressive strengths so as to evaluate the long term effects of the hammer blows on the integrity of the concrete. Theoretically, the tensile and compressive strengths of the concrete prisms could be determined by the flexural test method specified in BS1881: Part 118: 1983 and the equivalent cube test method specified in BS1881: Part 119: 1983. However, since the concrete prisms might have been cracked during the hammering tests and the exact positions of any cracks formed, which could be anywhere along the length, might affect the flexural strength measurements, it was likely that the tensile strength results obtained by the flexural test method would be quite erratic. To avoid this problem, the tensile strength of the prisms was measured by applying direct tension to the ends of the prisms so that the tensile strength results would be independent of the positions of any cracks present in the specimens. After the direct tension test, the equivalent cube method was applied to portions of the prisms broken in tension to determine the equivalent cube strength of the prisms.

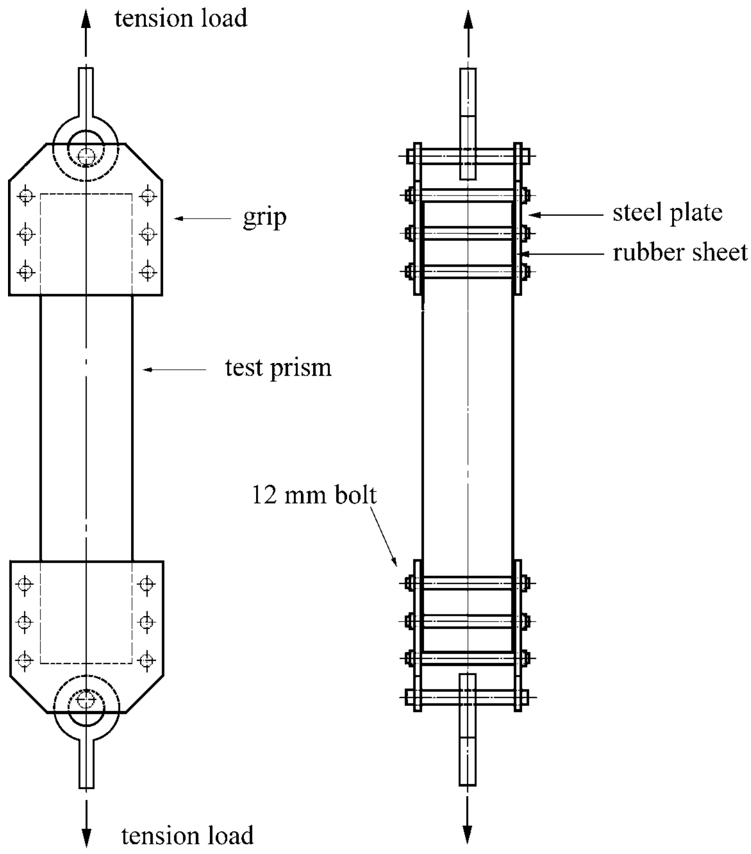


Figure 3.5 - Direct tension testing equipment

A pair of grips has been specifically fabricated for the direct tension test. Each grip consisted of two steel plates and two rubber sheets to be attached to two opposite faces at one end of the prism. Gripping force was provided at each grip by six bolts tightened to press the steel plates and rubber sheets against the prism, as shown in Fig.3.5. A hook whose centre coincided with the centroidal axis of the test prism was provided at each grip to make sure that the force applied would not cause bending of the test specimen. The grips, together with the test prism that they were holding, were installed onto an Avery 1000 kN capacity universal testing machine and then tension load was applied to the specimen under load control until the prism failed. None of the specimens failed at the gripping positions indicating that the gripping stresses applied at the two ends of the prisms had not affected the tension failure of the specimens.

3.2.4 Test Standards Followed

As far as possible, relevant British or local standards were followed in carrying out the tests. The standards followed are listed below:

Quality control tests:

In general accordance with the following standards:

slump test	BS1881: Part 102: 1983
compacting factor test	BS1881: Part 103: 1983
V-B time test	BS1881: Part 104: 1983
cube test	BS1881: Part 116: 1983

except that the curing temperature for the cubes was 27ºC, as required by the Hong Kong Construction Standard CS1: 1990.

Stiffening time test:

In general accordance with Appendix C of BS5075: Part 1: 1982 except that the storage temperature of the mortar specimens for stiffening time tests was the same as the designated curing temperature of the concrete (the standard requires a storage temperature of 20ºC) and that the penetration resistance measurements was performed at half-hourly intervals until 6 hours after mixing of the concrete.

Material property tests:

In general accordance with the following standards:

ultrasonic test	BS1881: Part 203: 1986
cube test	BS1881: Part 116: 1983
split cylinder test	BS1881: Part 117: 1983

except that the curing temperature of the test specimens was the same as the designated curing temperature of the concrete.

Hammering test:

There was no relevant standard that could be followed.

Integrity tests:

The size of the prism specimens was the same as that of the standard 150 mm $\times$ 150 mm $\times$ 750 mm test beam specified in BS1881: Part 118: 1983. Casting of the prism specimens was in accordance with BS1881: Part 109: 1983. The tensile strength of the prisms was, however, measured by applying direct tension instead of by the flexural testing method specified in BS1881: Part 118: 1983. Equivalent cube strength of the prisms was measured in accordance with BS1881: Part 119: 1983. The curing temperature of the prisms was the designated curing temperature of the concrete instead of 20ºC required by the British standards.

4. TEST RESULTS

4.1 Quality Control Tests of Concrete Mixes Cast

From each batch of concrete cast, samples were taken for workability and cube strength tests for quality control purpose.

Three workability tests, namely: slump test, V-B time test and compacting factor test, were carried out. Although the concrete mixes were designed to have a slump of 75 mm, the actual slump values varied from about 65 mm to 85 mm. The corresponding V-B times and compacting factors varied from about 1.0 s to 3.0 s and from about 0.92 to 0.97 respectively. These are considered reasonable variations for the large number of batches of different concrete mixes cast.

Three 150 mm cubes were cast from each batch of concrete for the strength test. The cubes were de-moulded one day after casting and were then cured at 27ºC in a water tank until testing at the age of 28-day. From each set of the three cubes, the cube strength results were averaged to give the mean cube strength value for the batch of concrete cast.

Details of the test results for each batch of concrete cast are tabulated in Tables 4.1 to 4.6. When studying the test results, it should be borne in mind that from each batch of concrete, fifteen prisms were cast. The fifteen prisms were used for hammering tests at three different ages (the number of prisms required for hammering test at each age was five) and thus each set of quality control tests corresponded to concrete cast for hammering tests at three different ages. That is why in Tables 4.1 to 4.6, concretes cast for hammering tests at different ages can have the same quality control test results. It should also be noted that the cubes cast for quality control testing purpose were cured at a temperature of 27ºC as required by Hong Kong Construction Standard CS1 which is not the same as the designated curing temperature for the concrete prisms cast for hammering tests.

Table 4.1 - Quality control test results for G20/0PFA/20C and G20/0PFA/30C

Batch no.	Age at which hammering test carried out	Slump (mm)	V-B time (s)	Compacting factor	Mean 28-day cube strength (MPa)
G20/0PFA/20C Series 1 tests	2 hrs	75	1.5	0.97	32.7
	4 hrs	75	1.5	0.97	32.7
	6 hrs	75	1.5	0.97	32.7
	8 hrs	70	2.8	0.96	31.1
	10 hrs	70	2.8	0.96	31.1
	12 hrs	70	2.8	0.96	31.1
G20/0PFA/20C Series 2 tests	12 hrs	70	1.9	0.97	32.5
	18 hrs	70	1.9	0.97	32.5
	24 hrs	80	1.5	0.97	31.7
	3 days	70	1.9	0.97	32.5
	7 days	80	1.5	0.97	31.7
	28 days	80	1.5	0.97	31.7
G20/0PFA/30C Series 1 tests	2 hrs	80	1.4	0.96	31.8
	4 hrs	85	1.0	0.97	31.6
	6 hrs	85	1.0	0.97	31.6
	8 hrs	80	1.4	0.96	31.8
	10 hrs	70	2.0	0.96	31.3
	12 hrs	65	2.2	0.95	32.2
G20/0PFA/30C Series 2 tests	12 hrs	65	2.2	0.95	32.2
	18 hrs	80	1.4	0.96	31.8
	24 hrs	75	2.3	0.97	31.9
	3 days	70	2.0	0.96	31.3
	7 days	75	2.3	0.97	31.9
	28 days	75	2.3	0.97	31.9

Table 4.2 - Quality control test results for G20/25PFA/20C and G20/25PFA/30C

Batch no.	Age at which hammering test carried out	Slump (mm)	V-B time (s)	Compacting factor	Mean 28-day cube strength (MPa)
G20/25PFA/20C Series 1 tests	2 hrs	70	1.7	0.96	20.5
	4 hrs	70	2.1	0.94	23.5
	6 hrs	70	2.1	0.94	23.5
	8 hrs	75	1.6	0.96	21.9
	10 hrs	75	1.9	0.96	22.8
	12 hrs	75	1.9	0.96	22.8
G20/25PFA/20C Series 2 tests	12 hrs	75	1.9	0.96	22.8
	18 hrs	70	2.1	0.94	23.5
	24 hrs	70	1.7	0.96	20.5
	3 days	75	1.6	0.96	21.9
	7 days	75	1.1	0.95	20.7
	28 days	75	1.1	0.95	20.7
G20/25PFA/30C Series 1 tests	2 hrs	70	2.3	0.96	24.6
	4 hrs	75	2.8	0.96	26.6
	6 hrs	75	2.8	0.96	26.6
	8 hrs	70	2.3	0.96	24.6
	10 hrs	80	1.5	0.96	28.0
	12 hrs	80	1.5	0.96	28.0
G20/25PFA/30C Series 2 tests	12 hrs	80	1.5	0.96	28.0
	18 hrs	75	2.8	0.96	26.6
	24 hrs	70	2.1	0.94	24.3
	3 days	70	2.3	0.96	24.6
	7 days	70	2.1	0.94	24.3
	28 days	70	2.1	0.94	24.3

Table 4.3 - Quality control test results for G30/0PFA/20C and G30/0PFA/30C

Batch no.	Age at which hammering test carried out	Slump (mm)	V-B time (s)	Compacting factor	Mean 28-day cube strength (MPa)
G30/0PFA/20C Series 1 tests	2 hrs	75	2.5	0.94	40.7
	4 hrs	75	2.5	0.94	40.7
	6 hrs	70	2.6	0.95	41.6
	8 hrs	70	2.6	0.95	41.6
	10 hrs	75	2.1	0.95	43.4
	12 hrs	75	2.1	0.95	43.4
G30/0PFA/20C Series 2 tests	12 hrs	75	2.1	0.95	43.4
	18 hrs	75	2.5	0.94	40.7
	24 hrs	70	2.6	0.95	41.6
	3 days	75	2.8	0.95	42.7
	7 days	75	2.8	0.95	42.7
	28 days	75	2.8	0.95	42.7
G30/0PFA/30C Series 1 tests	2 hrs	75	2.3	0.95	41.5
	4 hrs	75	2.3	0.95	41.5
	6 hrs	75	1.8	0.96	40.1
	8 hrs	75	1.8	0.96	40.1
	10 hrs	70	2.6	0.96	43.8
	12 hrs	70	2.6	0.96	43.8
G30/0PFA/30C Series 2 tests	12 hrs	70	2.6	0.96	43.8
	18 hrs	75	2.3	0.95	41.5
	24 hrs	75	1.8	0.96	40.1
	3 days	75	2.0	0.96	43.3
	7 days	75	2.0	0.96	43.3
	28 days	75	2.0	0.96	43.3

Table 4.4 - Quality control test results for G30/25PFA/20C and G30/25PFA/30C

Batch no.	Age at which hammering test carried out	Slump (mm)	V-B time (s)	Compacting factor	Mean 28-day cube strength (MPa)
G30/25PFA/20C Series 1 tests	2 hrs	75	2.6	0.93	29.8
	4 hrs	70	3.0	0.95	35.2
	6 hrs	70	3.0	0.95	35.2
	8 hrs	70	3.0	0.95	35.2
	10 hrs	75	2.6	0.93	29.8
	12 hrs	75	2.6	0.93	29.8
G30/25PFA/20C Series 2 tests	12 hrs	75	2.3	0.95	35.9
	18 hrs	70	3.2	0.95	37.5
	24 hrs	75	2.3	0.95	35.9
	3 days	70	3.2	0.95	37.5
	7 days	75	2.3	0.95	35.9
	28 days	70	3.2	0.95	37.5
G30/25PFA/30C Series 1 tests	2 hrs	65	3.2	0.95	38.5
	4 hrs	70	2.3	0.95	36.8
	6 hrs	70	2.3	0.95	36.8
	8 hrs	70	2.3	0.95	36.8
	10 hrs	65	3.2	0.95	38.5
	12 hrs	65	3.2	0.95	38.5
G30/25PFA/30C Series 2 tests	12 hrs	70	2.3	0.95	38.8
	18 hrs	70	2.3	0.95	38.8
	24 hrs	65	3.2	0.94	37.0
	3 days	70	2.3	0.95	38.8
	7 days	65	3.2	0.94	37.0
	28 days	65	3.2	0.94	37.0

Table 4.5 - Quality control test results for G40/0PFA/20C and G40/0PFA/30C

Batch no.	Age at which hammering test carried out	Slump (mm)	V-B time (s)	Compacting factor	Mean 28-day cube strength (MPa)
G40/0PFA/20C Series 1 tests	2 hrs	65	2.0	0.94	51.7
	4 hrs	75	1.5	0.93	53.8
	6 hrs	75	2.0	0.92	47.6
	8 hrs	65	2.5	0.92	55.7
	10 hrs	75	2.0	0.92	47.6
	12 hrs	75	1.5	0.93	53.8
G40/0PFA/20C Series 2 tests	12 hrs	65	2.5	0.92	55.7
	18 hrs	65	2.0	0.94	51.7
	24 hrs	70	2.5	0.92	55.7
	3 days	65	2.0	0.94	51.7
	7 days	75	2.0	0.92	47.6
	28 days	75	1.5	0.93	53.8
G40/0PFA/30C Series 1 tests	2 hrs	75	1.9	0.97	52.4
	4 hrs	75	2.0	0.97	51.2
	6 hrs	75	1.9	0.93	51.7
	8 hrs	70	2.9	0.97	50.1
	10 hrs	75	1.9	0.93	51.7
	12 hrs	75	2.0	0.97	51.2
G40/0PFA/30C Series 2 tests	12 hrs	70	2.9	0.97	50.1
	18 hrs	75	1.9	0.97	52.4
	24 hrs	70	2.9	0.97	50.1
	3 days	75	1.9	0.97	52.4
	7 days	75	1.9	0.93	51.7
	28 days	70	2.0	0.97	51.2

Table 4.6 - Quality control test results for G40/25PFA/20C and G40/25PFA/30C

Batch no.	Age at which hammering test carried out	Slump (mm)	V-B time (s)	Compacting factor	Mean 28-day cube strength (MPa)
G40/25PFA/20C Series 1 tests	2 hrs	70	2.0	0.94	45.1
	4 hrs	70	2.0	0.94	45.1
	6 hrs	75	2.9	0.95	43.8
	8 hrs	65	3.3	0.90	44.1
	10 hrs	75	1.9	0.95	45.2
	12 hrs	75	1.9	0.95	45.2
G40/25PFA/20C Series 2 tests	12 hrs	75	1.9	0.95	45.2
	18 hrs	65	3.3	0.90	44.1
	24 hrs	70	2.0	0.94	45.1
	3 days	75	2.9	0.95	43.8
	7 days	65	3.3	0.90	44.1
	28 days	75	2.9	0.95	43.8
G40/25PFA/30C Series 1 tests	2 hrs	70	2.1	0.95	46.2
	4 hrs	70	2.1	0.95	46.2
	6 hrs	70	1.9	0.95	47.5
	8 hrs	70	1.9	0.95	47.5
	10 hrs	75	1.8	0.95	48.9
	12 hrs	75	1.8	0.95	48.9
G40/25PFA/30C Series 2 tests	12 hrs	75	1.8	0.95	48.9
	18 hrs	75	2.0	0.95	45.3
	24 hrs	70	1.9	0.95	47.5
	3 days	70	2.1	0.95	46.2
	7 days	75	2.0	0.95	45.3
	28 days	75	2.0	0.95	45.3

A summary of the quality control test results is presented in Table 4.7 wherein the average values, standard deviations and coefficients of variation of the 28-day cube strengths of the concrete mixes studied are listed. It is seen that the largest standard deviation of the various concrete mixes was only 2.8 MPa. This indicates that the production of the concrete mixes was in generally under good quality control. The concrete mixes without PFA generally have their respective mean strengths more than 10 MPa higher than their specified characteristic strengths. However, the concrete mixes with PFA added were of somewhat inferior quality; their mean strengths were generally less than 10 MPa higher than their specified characteristic strengths. Thus when interpreting the test results, reference should be made to their actual mean strengths rather their specified grades.

Table 4.7 - Statistical analysis of the 28-day cube strength results

Mix no.	Average value of 28-day cube strength (MPa)	Standard deviation of 28-day cube strength (MPa)	Coefficient of variation (%)
G20/0PFA	31.9	0.6	1.9
G20/25PFA	23.7	2.5	10.5
G30/0PFA	42.1	1.4	3.3
G30/25PFA	36.2	2.8	7.7
G40/0PFA	51.8	2.4	4.6
G40/25PFA	45.8	1.7	3.7

4.2 Stiffening Times of the Concrete Mixes

The stiffening time tests were conducted on the mortar fractions of the concrete mixes which were obtained by sieving the coarse aggregate particles away from the fresh concretes. Although the British Standard requires only the times to reach penetration resistances of 0.5 MPa and 3.5 MPa to be measured, the penetration resistance of the mortar was measured at 30 minute intervals to see how the penetration resistance actually varied with time. The results of such measurements are plotted in Figs. 4.1 to 4.6. It can be seen from these figures that the penetration resistance generally increases very slowly within the first two hours after mixing. At about four to five hours after mixing, however, the penetration resistance starts to increase fairly rapidly indicating that the mortar being tested has then began to solidify and behave more like a solid.

From the graphs presented in Figs. 4.1 to 4.6, the times taken to reach penetration resistances of 0.5 MPa and 3.5 MPa are determined and the results are tabulated in Table 4.8. It is evident from these results that the stiffening time of a concrete depends on the grade of the concrete, whether PFA has been added and the curing temperature.

Table 4.8 - Stiffening times of the concrete mixes

Batch no.	Time to reach penetration resistance of 0.5 MPa (hour:minute)	Time to reach penetration resistance of 3.5 MPa (hour:minute)
G20/0PFA/20C	3:15	4:45
G20/0PFA/30C	3:05	4:10
G20/25PFA/20C	4:45	6:45
G20/25PFA/30C	4:35	6:20
G30/0PFA/20C	3:30	5:10
G30/0PFA/30C	2:40	4:00
G30/25PFA/20C	4:30	6:15
G30/25PFA/30C	3:20	4:50
G40/0PFA/20C	4:15	6:05
G40/0PFA/30C	2:45	4:45
G40/25PFA/20C	4:30	6:30
G40/25PFA/30C	4:15	5:50

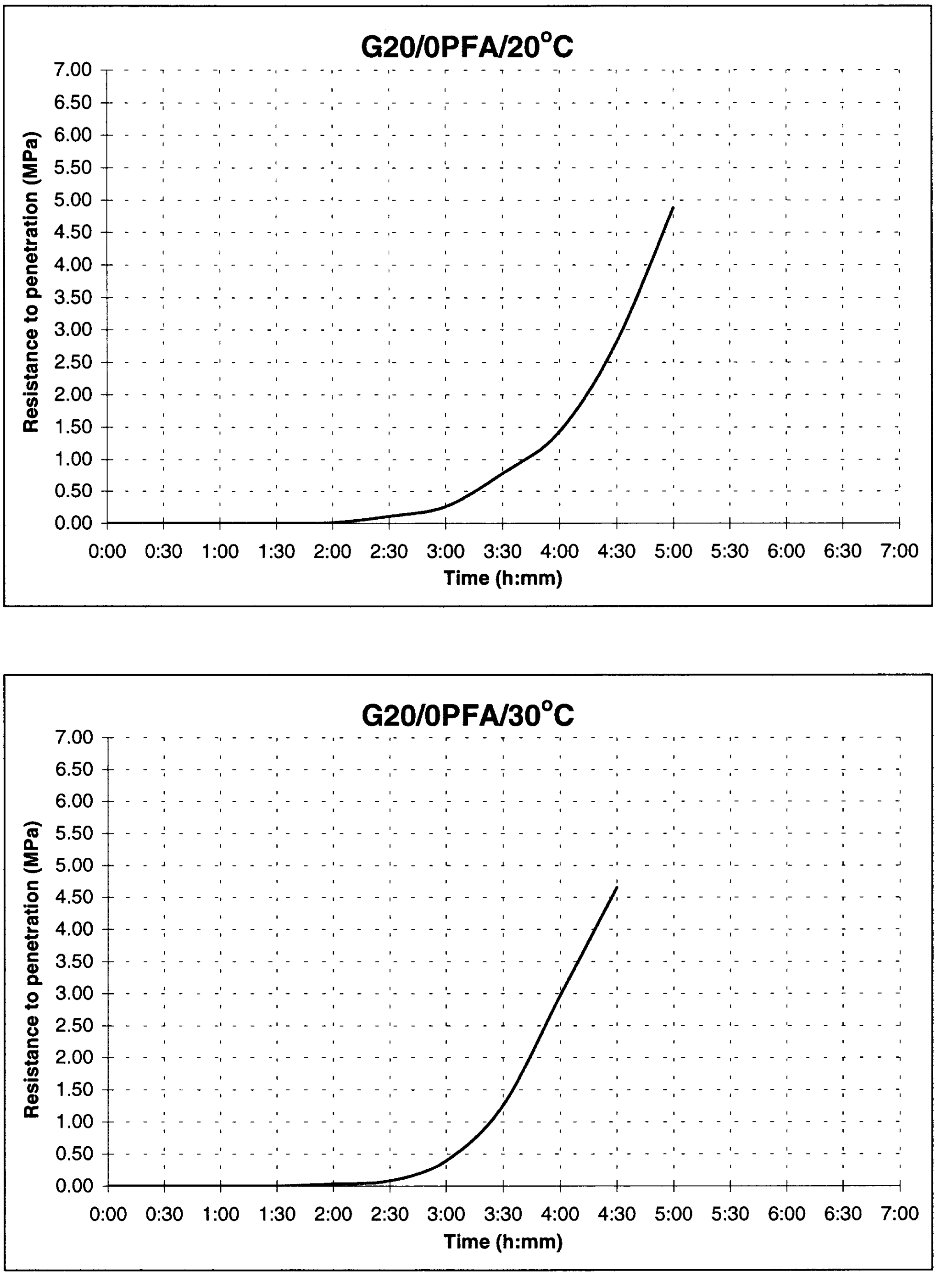
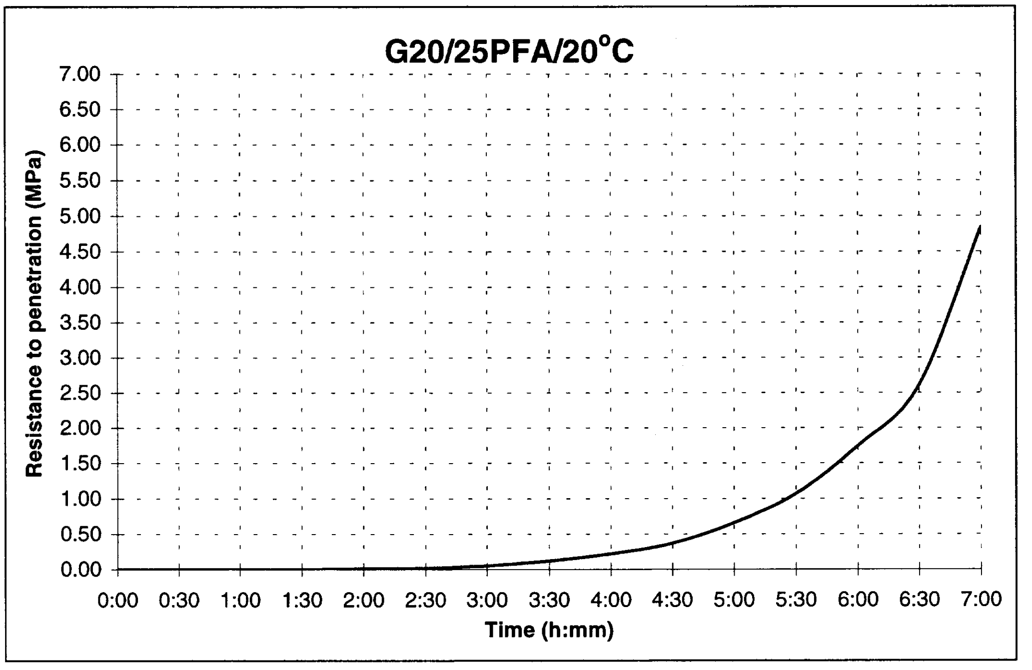


Figure 4.1 - The results of the stiffening time tests of G20/0PFA



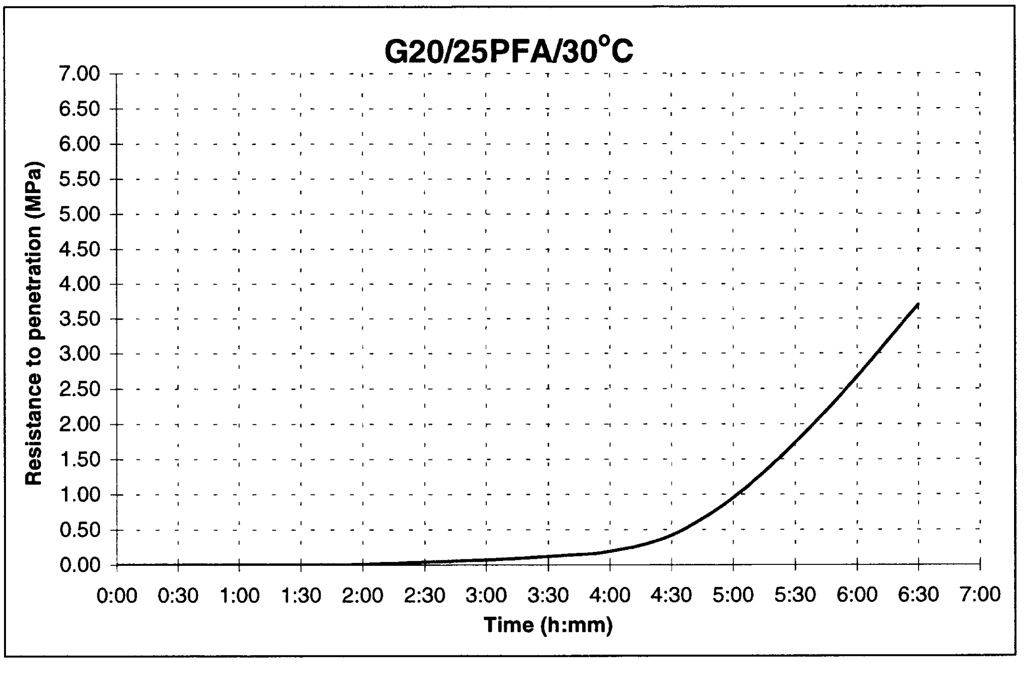
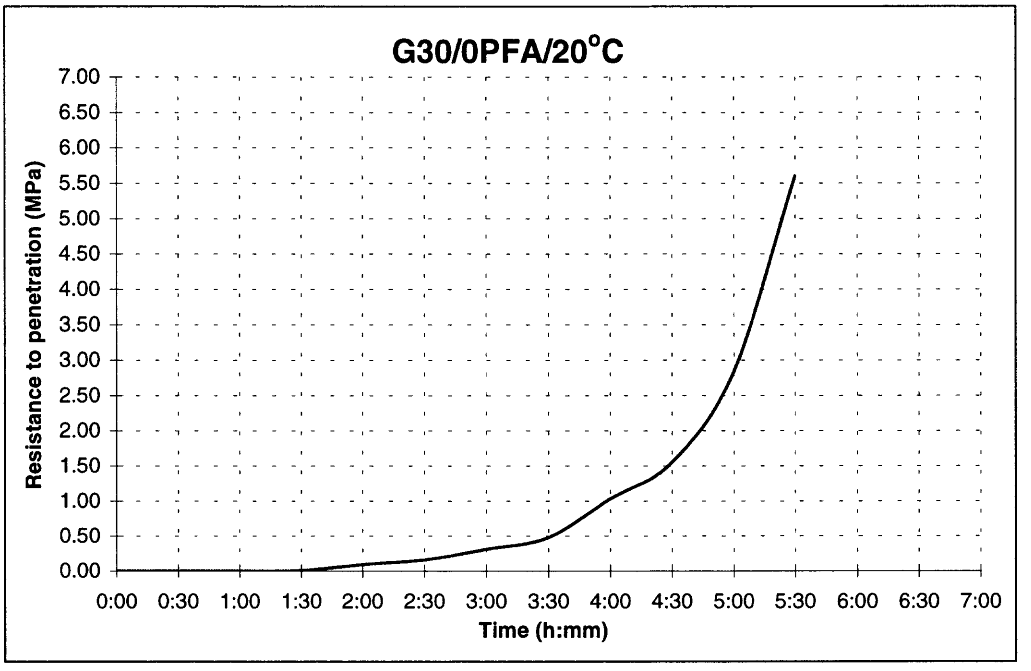


Figure 4.2 - The results of the stiffening time tests of G20/25PFA



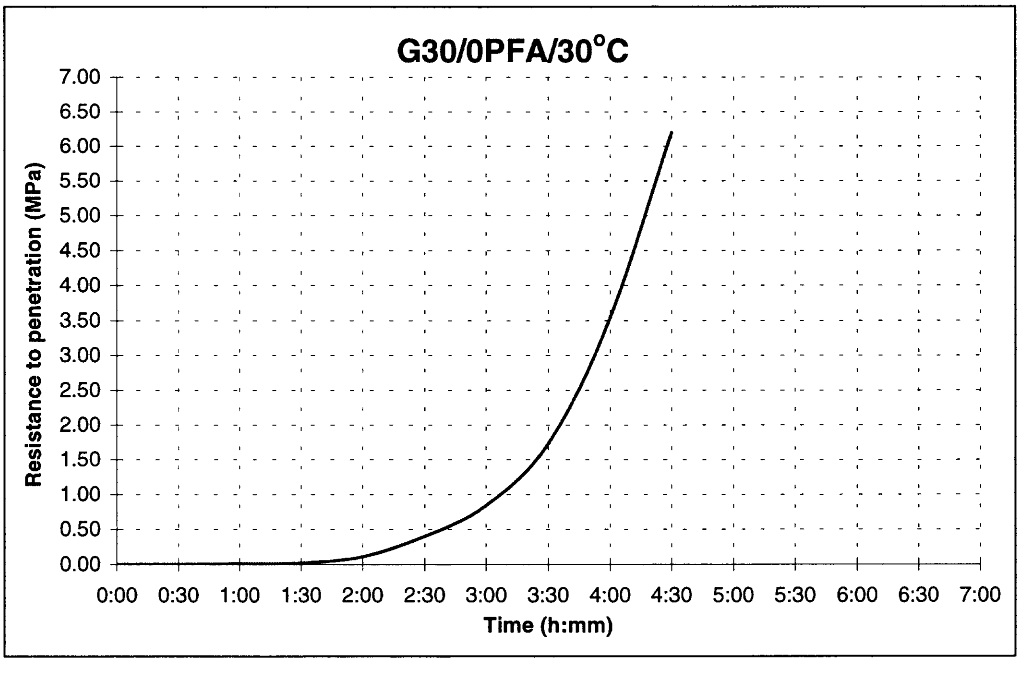
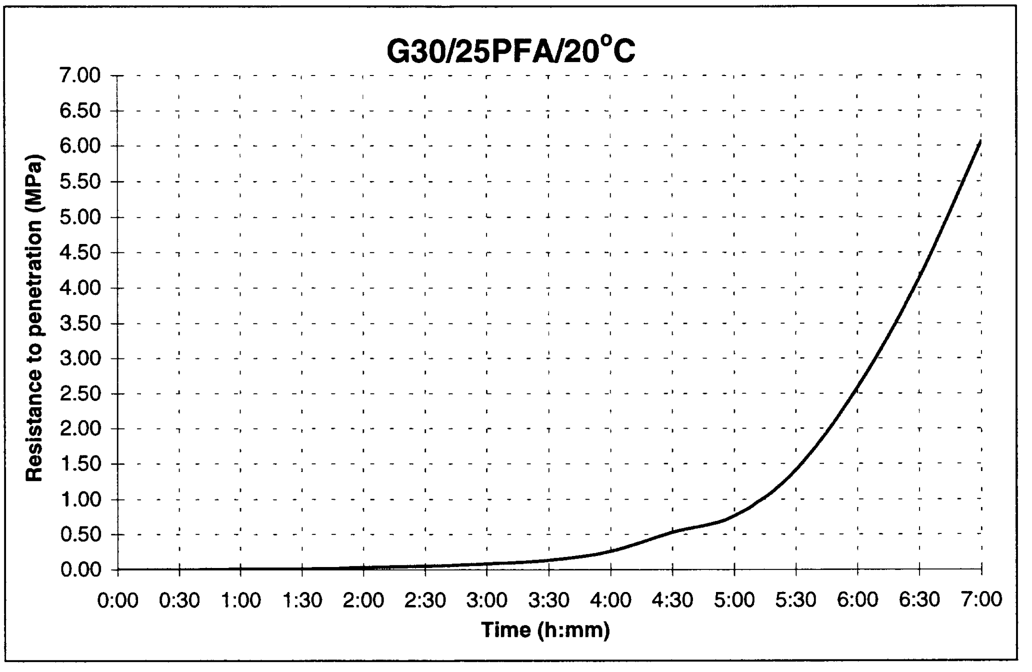


Figure 4.3 - The results of the stiffening time tests of G30/0PFA



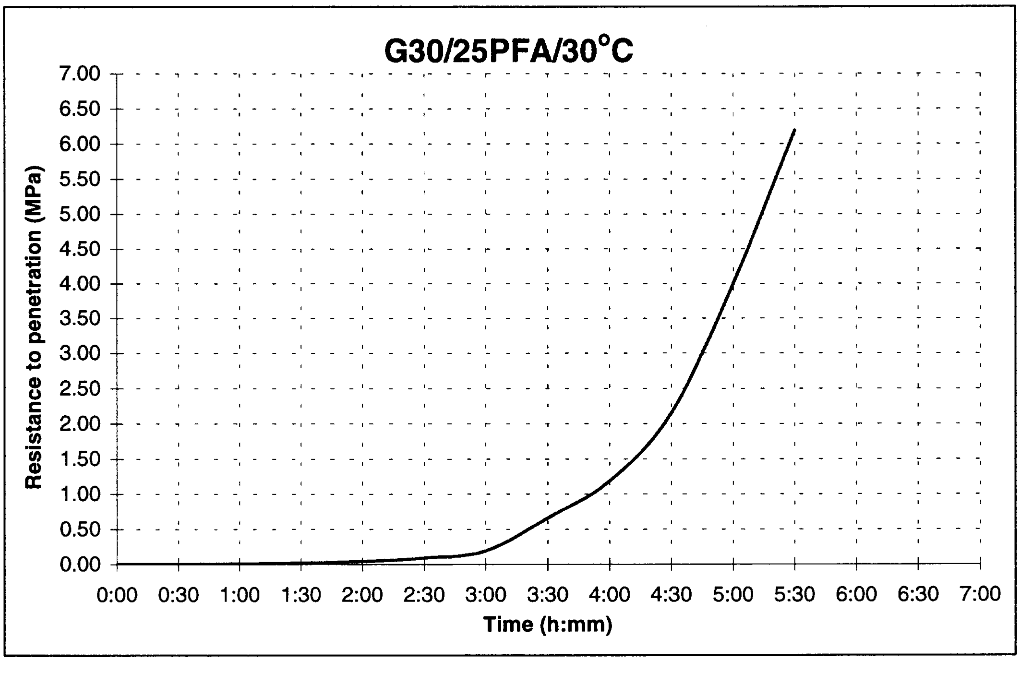
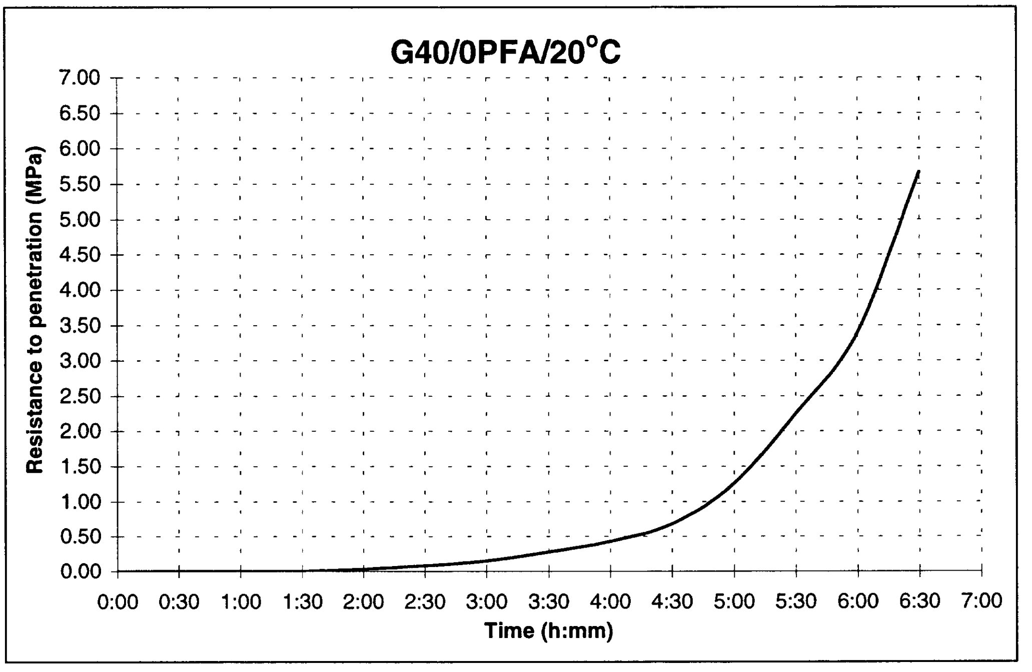


Figure 4.4 - The results of the stiffening time tests of G30/25PFA



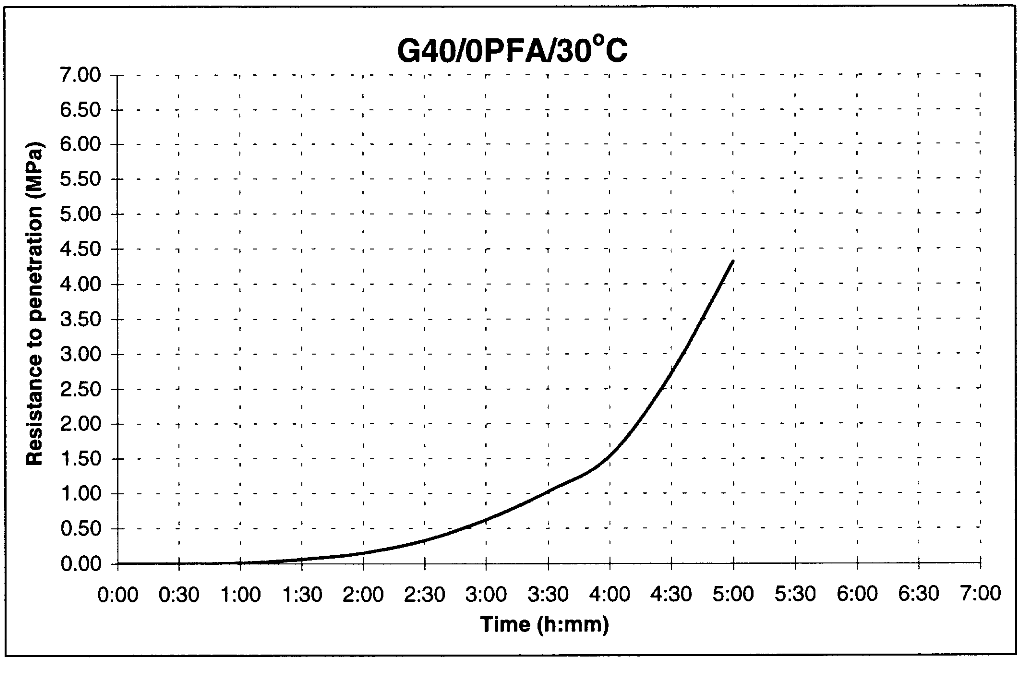


Figure 4.5 - The results of the stiffening time tests of G40/0PFA

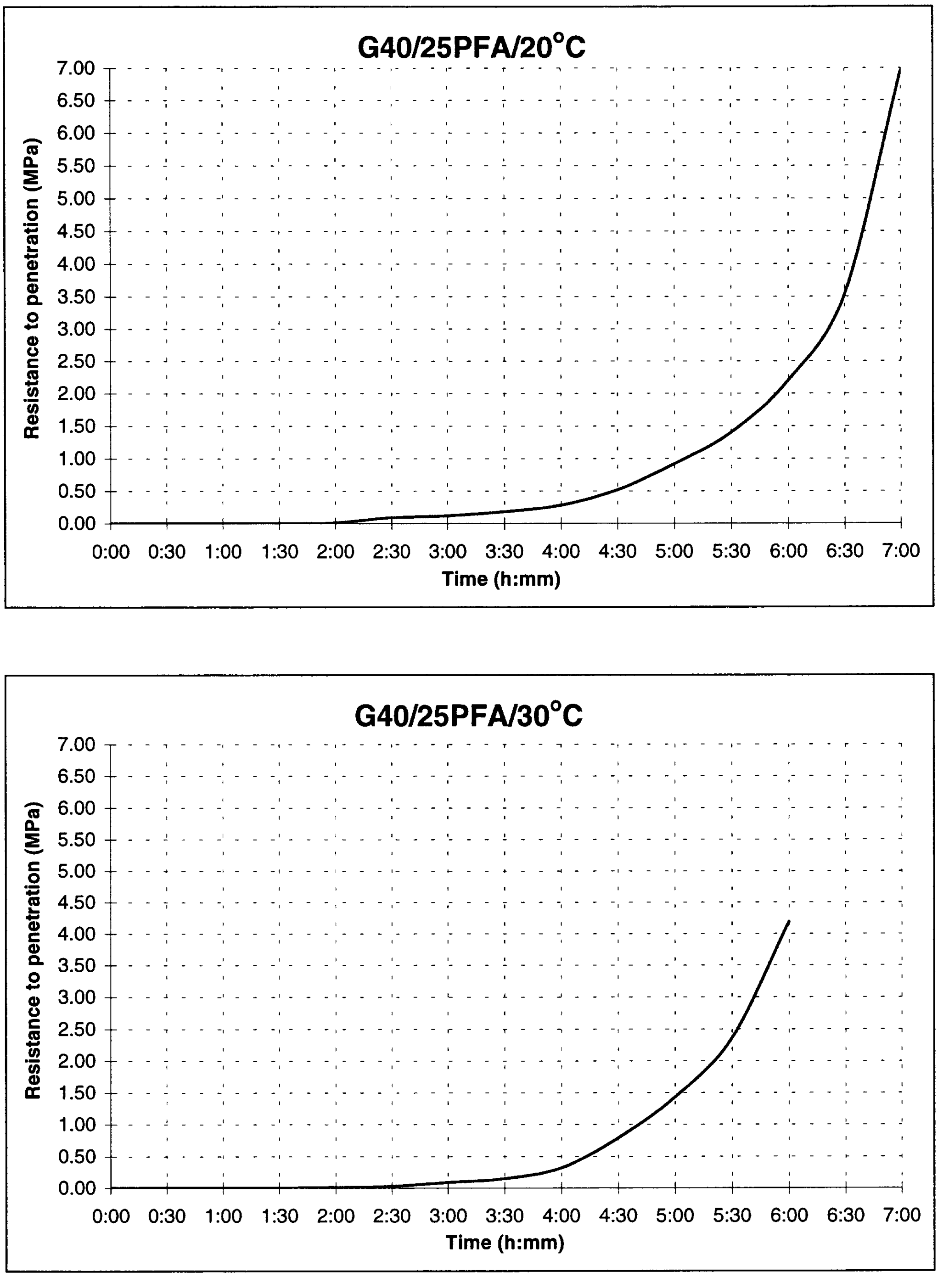


Figure 4.6 - The results of the stiffening time tests of G40/25PFA

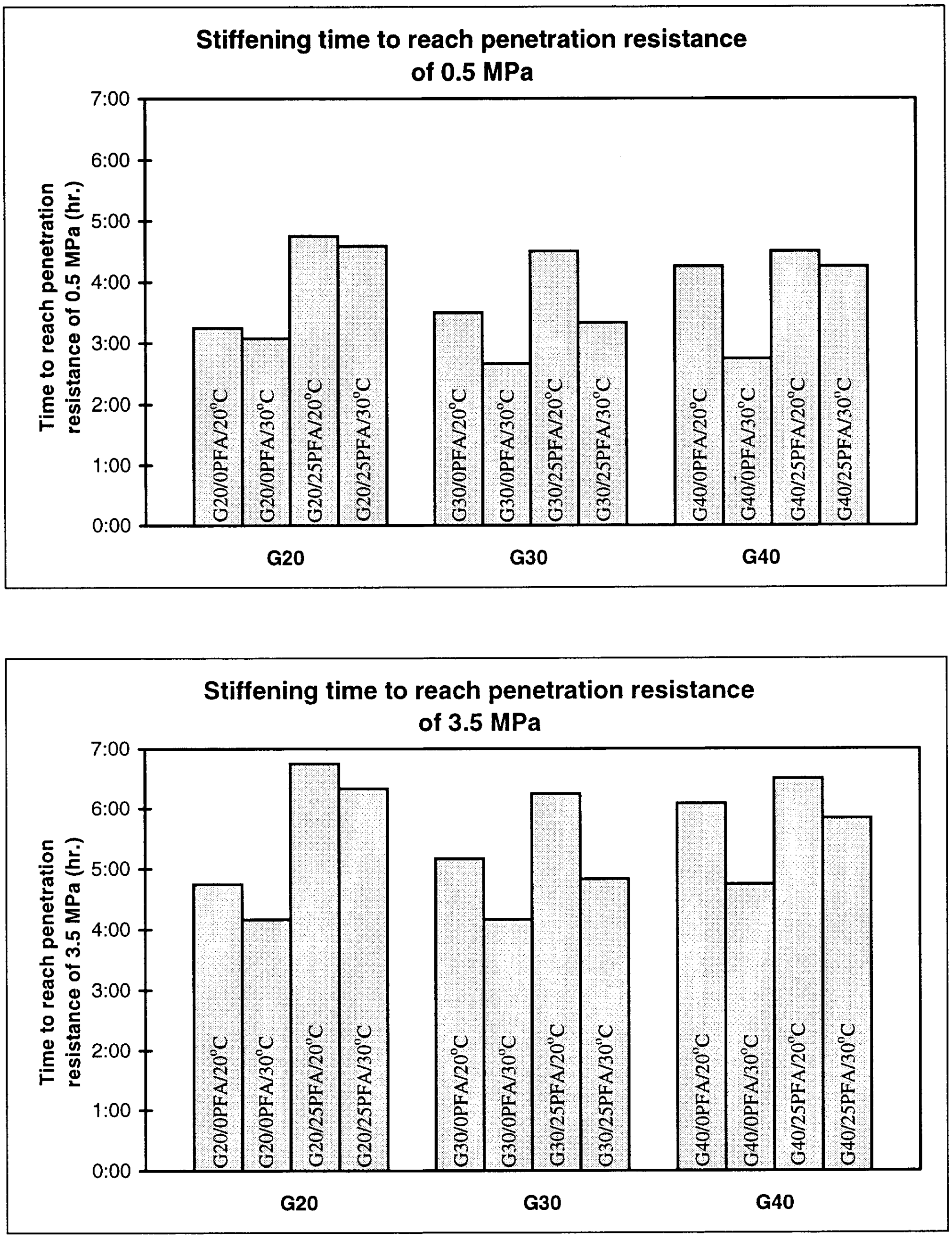


Figure 4.7 - Stiffening times of the concrete mixes

In order to allow easier visualization, the stiffening time results are plotted in the form of a bar chart in Fig. 4.7. Comparing concretes having the same amount of PFA and cured at the same temperature but of different grades, it can be seen that generally speaking, the higher the concrete grade is, the longer is the stiffening time. It is, however, believed that the concrete grade does not really have a direct influence on the stiffening time. The longer stiffening times of the higher grade concretes are due to the larger amounts of superplasticizer used (see Table 3.2 for the amounts of superplasticizer added to the various concrete mixes) which from past experience is known to have a slight retarding effect. Comparing concretes of the same grade and cured at the same temperature but having different amounts of PFA, it is obvious that the use of PFA can lead to a significant increase in stiffening time. This is expected as the use of PFA to replace part of the cement would generally slow than the strength development process. Comparing concrete of the same mix composition but cured at different temperatures, it is evident that a higher curing temperature leads to a shorter stiffening time. This can be explained by the fact that cement hydration is generally faster at a higher temperature.

4.3 Material Properties at Time of Hammering Test

For concretes which had hardened sufficiently for the moulds to be removed before the hammering tests, their material properties at the time of conducting hammering test were measured. The material properties measured were the cube compressive strength, split cylinder tensile strength, dynamic modulus, static modulus and longitudinal wave propagation velocity. Specimens for material properties tests were cast from the same batch of concrete out of which the prisms for hammering test were produced and were cured alongside the prisms so that they were cured under exactly the same conditions. They were tested at the same time of conducting hammering test and thus the material properties measured may be taken as those of the concrete at the time of hammering. Since the moulds could not be removed at less than 12 hours after casting, the material properties tests were carried out only for concretes in Series 2 tests, not for concretes in Series 1 tests.

For the determination of cube compressive strength and split cylinder tensile strength at each designated curing temperature and at each age, three cubes and three cylinders were cast from the concrete. From the three cubes, the cube strength results are averaged to give the mean compressive strength of the concrete at the time of hammering test. Similarly, from the three cylinders, the split cylinder tensile strength results are averaged to give the mean tensile strength of the concrete.

On the other hand, the dynamic modulus and longitudinal wave propagation velocity of the concrete were measured by ultrasonic testing of the concrete prisms cast for the hammering tests. The ultrasonic tests were carried out on the concrete prisms in their longitudinal direction just before they were subjected to the hammering impact. Since in each set of five prisms to be tested for their vibration resistance, only four were subjected to hammer blows, the ultrasonic tests were carried out on the four prisms which would be hammered. The four ultrasonic test results from each set of prisms are averaged to give the dynamic modulus and longitudinal wave velocity of the concrete at the time of hammering test. Occasionally, especially when the concrete to be tested was at the age of 12 hours, the signal received from the ultrasonic transducer was too weak for accurate measurement to be made and consequently no ultrasonic test result was obtained.

The results of the material properties tests are listed in Tables 4.9 to 4.11. For easier interpretation, the compressive and tensile strengths of the concretes are plotted against age in Figs. 4.8 to 4.13. From these results, it is evident that the curing temperature has a significant effect on the rate of strength development. Generally speaking, the compressive and tensile strengths attained by a concrete at a higher curing temperature of 30ºC are higher than the corresponding strengths attained at a curing temperature of 20ºC at all ages. Moreover, this effect of the curing temperature is somewhat greater for concretes containing PFA.

Apart from the three cubes and three cylinders for the determination of compressive and tensile strengths of the concrete, one additional cylinder had also been cast for the measurement of static modulus. This additional cylinder was cured under the same conditions as for the concrete prisms and was tested at the same time of hammering test. The static modulus was measured by gluing electrical strain gauges onto the surface of the cylinders after they were de-moulded, applying axial compression loads to the cylinders and measuring the axial strains as the compression loads were applied. The results obtained are tabulated in Tables 4.12 to 4.14. Since it took several hours for the cylinders to dry up, the strain gauges to be glued onto the cylinder surface and the ends of the cylinders to be capped before the static moduli could be measured, it was found difficult to measure the static moduli of the concretes at ages of less than 24 hours and thus the static moduli of the concretes were occasionally not measured at such early ages.

With the large amount of experimental data available, the opportunity is taken here to study how the different material properties are inter-related to each other.

For this purpose, the split cylinder tensile strength is plotted against the cube compressive strength in Fig.4.14. As cited by Neville (1995), Raphael (1984) has proposed the following equation for the split cylinder tensile strength:

$$f_t = 0.30 \cdot (f_c')^{2/3} \qquad \text{...(4.1)}$$

where f_t and f_c' are the split cylinder tensile strength and cylinder compressive strength respectively in MPa. On the other hand, based on some other test results, Gardner (1990) has proposed the following equation instead:

$$f_t = 0.33 \cdot (f_c')^{2/3} \qquad \text{...(4.2)}$$

It is, however, found that the experimental results do not agree well with any of the above equations (during the comparison, the cylinder compressive strength is taken to be equal to 0.8 of the cube compressive strength, as recommended by BS1881: Part 120: 1983). Assuming that the split cylinder tensile strength is related to the cube compressive strength f_{cu} by $f_t = k(f_{cu})^{2/3}$ where k is a coefficient, the best fit equation for the experimental results is found by regression analysis to be:

$$f_t = 0.24 \cdot (f_{cu})^{2/3} \qquad \text{...(4.3)}$$

It is plotted in Fig.4.14 to demonstrate how it fits the experimental results. The correlation index R^2 of the above equation is 0.97 which is regarded as very good for concrete test results. Converting the above equation to be in terms of cylinder compressive strength, the following equation is derived:

$$f_t = 0.28 \cdot (f_c')^{2/3} \qquad \text{...(4.4)}$$

Hence, the tensile strength of the concretes tested is on average slightly lower than those of the concretes studied by Raphael and by Gardner.

The relation between the static modulus E_c and the dynamic modulus E_d is studied by plotting one against the other in Fig.4.15. Assuming that they are proportional to each other, the best fit equation relating them together is found to be:

$$E_c = 0.68 \cdot E_d \qquad ...(4.5)$$

The R^2 value between this equation and the experimental results is 0.84. It is noteworthy that the ratio of static to dynamic moduli of 0.68 in the above equation is somewhat lower than the value of 0.83 obtained by Lydon and Balendran (1986).

Likewise, to study how the static and dynamic moduli vary with the concrete strength, the static modulus and the dynamic modulus are plotted against the cube compressive strength in Figs. 4.16 and 4.17 respectively. The American code ACI 318-89 Revised 1992 recommended the following equation for the static modulus:

$$E_c = 4.73 \cdot (f_c')^{1/2} \qquad ...(4.6)$$

in which E_c is expressed in GPa and f_c' in MPa. On the other hand, as cited by Neville (1995), Kakizaki *et al* (1992) have proposed the following equation:

$$E_c = 3.65 \cdot (f_c')^{1/2} \qquad ...(4.7)$$

which is quite different from the one given by the ACI code. Assuming that the static modulus and the compressive strength are related by $E_c = k(f_{cu})^{1/2}$ where k is an unknown coefficient to be determined, the best fit equation for the experimental results is determined by regression analysis as:

$$E_c = 4.17 \cdot (f_{cu})^{1/2} \qquad ...(4.8)$$

The corresponding equation in terms of cylinder compressive strength is obtained as:

$$E_c = 4.66 \cdot (f_c')^{1/2} \qquad ...(4.9)$$

Static modulus predicted by this equation is about 1.5% lower than that predicted by the ACI equation but is about 28% higher than that predicted by Kakizaki's equation. Similarly, assuming that the dynamic modulus and the compressive strength are also related by $E_d = k(f_{cu})^{1/2}$, the following best fit equation is derived:

$$E_d = 6.13 \cdot (f_{cu})^{1/2} \qquad ...(4.10)$$

Converting the cube compressive strength to cylinder compressive strength, the following equation is derived:

$$E_d = 6.85 \cdot (f_c')^{1/2} \qquad ...(4.11)$$

The above two equations for static and dynamic moduli are plotted alongside the experimental results in Figs. 4.16 and 4.17, from which it can be seen that they fit the experimental results quite well. The correlation index R^2 for Eqn.4.10 is 0.85, which is regarded as good. However, the correlation index for Eqn.4.8 is only 0.68. A possible cause for this is that there are more errors in the static modulus measurement. The static modulus measurement involved the measurement of both load and strain. Any error in either the load or strain measurement would contribute to error in the static modulus. Moreover, since the concrete surfaces had to be dried for strain gauges to be glued on before the static modulus measurement, the moisture condition of the concrete was quite variable.

The longitudinal wave propagation velocity c is plotted against the cube compressive strength in Fig.4.18. Assuming that they are inter-related by a power equation of the form $c = k(f_{cu})^n$ where both k and n are unknown coefficients and applying regression analysis, the following best fit equation is obtained:

$$c = 2041 \cdot (f_{cu})^{0.2} \qquad \text{...(4.12)}$$

in which the units of c is m/s. This equation has a R^2 value of 0.91 and, as plotted in Fig.4.18, agrees closely with the experimental results. These results reveal that for most hardened concretes, the value of c ranges from about 2000 to 4000 m/s.

The static tensile strain capacity ε_c', evaluated as f_t / E_c, is plotted against the cube compressive strength in Fig.4.19. Curve fitting yields the following equation:

$$\varepsilon_c' = 24.1 \cdot (f_{cu})^{0.44} \qquad \text{...(4.13)}$$

in which the unit for ε_c' is micro-strain. From this equation which has a R^2 value of 0.67, it is seen that the static tensile strain capacity of concrete increases from about 40 micro-strain at a cube strength of 3 MPa to 110 micro-strain at a cube strength of about 30 MPa.

Table 4.9 - Material properties of G20 concretes at the time of hammering test

Batch no.	Age at which hammering test carried out	Mean compressive strength (MPa)	Mean tensile strength (MPa)	Dynamic modulus (GPa)	Longitudinal wave velocity (m/s)
G20/0PFA/20C	12 hrs	3.9	0.43	-	-
	18 hrs	5.8	0.74	19.4	3140
	24 hrs	8.9	1.00	22.9	3410
	3 days	12.7	1.31	26.9	3700
	7 days	17.7	1.54	30.6	3940
	28 days	31.9	2.46	34.7	4200
G20/0PFA/30C	12 hrs	4.8	0.48	15.2	2780
	18 hrs	6.3	0.81	19.5	3150
	24 hrs	11.0	1.21	22.5	3380
	3 days	14.7	1.56	23.6	3460
	7 days	21.5	1.99	28.1	3780
	28 days	32.3	2.68	35.1	4220
G20/25PFA/20C	12 hrs	1.3	0.13	8.4	2070
	18 hrs	3.1	0.28	14.3	2700
	24 hrs	4.9	0.49	16.4	2890
	3 days	8.9	1.01	21.6	3310
	7 days	10.5	1.37	25.9	3630
	28 days	22.2	1.76	31.8	4020
G20/25PFA/30C	12 hrs	3.0	0.28	7.8	1990
	18 hrs	3.4	0.32	12.9	2560
	24 hrs	5.0	0.61	16.8	2920
	3 days	9.9	1.09	21.6	3310
	7 days	12.2	1.08	26.8	3690
	28 days	24.5	2.18	31.3	3990

Table 4.10 - Material properties of G30 concretes at the time of hammering test

Batch no.	Age at which hammering test carried out	Mean compressive strength (MPa)	Mean tensile strength (MPa)	Dynamic modulus (GPa)	Longitudinal wave velocity (m/s)
G30/0PFA/20C	12 hrs	4.1	0.40	13.9	2640
	18 hrs	7.5	0.76	18.9	3080
	24 hrs	10.2	0.94	20.4	3200
	3 days	18.6	1.82	28.6	3790
	7 days	28.7	2.09	32.1	4010
	28 days	39.3	2.87	36.5	4280
G30/0PFA/30C	12 hrs	5.9	0.52	16.3	2860
	18 hrs	9.0	0.81	20.2	3180
	24 hrs	10.4	0.99	20.9	3240
	3 days	21.9	1.79	27.7	3730
	7 days	37.1	2.72	33.8	4120
	28 days	41.9	3.07	35.9	4240
G30/25PFA/20C	12 hrs	2.6	0.27	12.2	2490
	18 hrs	4.8	0.41	15.9	2840
	24 hrs	7.4	0.75	19.0	3110
	3 days	10.4	1.09	24.7	3540
	7 days	18.9	1.62	29.3	3860
	28 days	33.9	2.54	34.9	4210
G30/25PFA/30C	12 hrs	3.6	0.37	12.6	2530
	18 hrs	7.2	0.62	18.8	3090
	24 hrs	8.6	0.77	20.1	3200
	3 days	14.8	1.30	27.2	3720
	7 days	24.4	2.20	30.1	3910
	28 days	43.1	3.04	35.7	4260

Table 4.11 - Material properties of G40 concretes at the time of hammering test

Batch no.	Age at which hammering test carried out	Mean compressive strength (MPa)	Mean tensile strength (MPa)	Dynamic modulus (GPa)	Longitudinal wave velocity (m/s)
G40/0PFA/20C	12 hrs	2.0	0.29	10.3	2290
	18 hrs	5.6	0.66	21.5	3310
	24 hrs	7.1	0.73	23.1	3430
	3 days	23.5	1.79	29.0	3840
	7 days	31.2	2.40	32.6	4070
	28 days	46.9	3.17	38.2	4410
G40/0PFA/30C	12 hrs	1.6	0.09	-	-
	18 hrs	13.0	1.06	22.3	3370
	24 hrs	13.4	1.12	23.0	3420
	3 days	27.0	2.08	28.4	3800
	7 days	38.9	2.60	31.3	3990
	28 days	50.8	3.26	36.0	4280
G40/25PFA/20C	12 hrs	0.7	0.03	-	-
	18 hrs	2.0	0.33	8.8	2110
	24 hrs	3.9	0.64	16.1	2860
	3 days	18.0	1.83	27.1	3710
	7 days	26.5	2.31	31.3	3990
	28 days	40.5	2.65	32.9	4090
G40/25PFA/30C	12 hrs	2.2	0.20	8.1	2030
	18 hrs	4.3	0.37	18.1	3030
	24 hrs	8.0	0.65	19.8	3170
	3 days	18.3	1.52	25.8	3620
	7 days	28.0	1.96	29.6	3880
	28 days	48.0	3.03	36.6	4310

Table 4.12 - Static moduli of G20 concretes

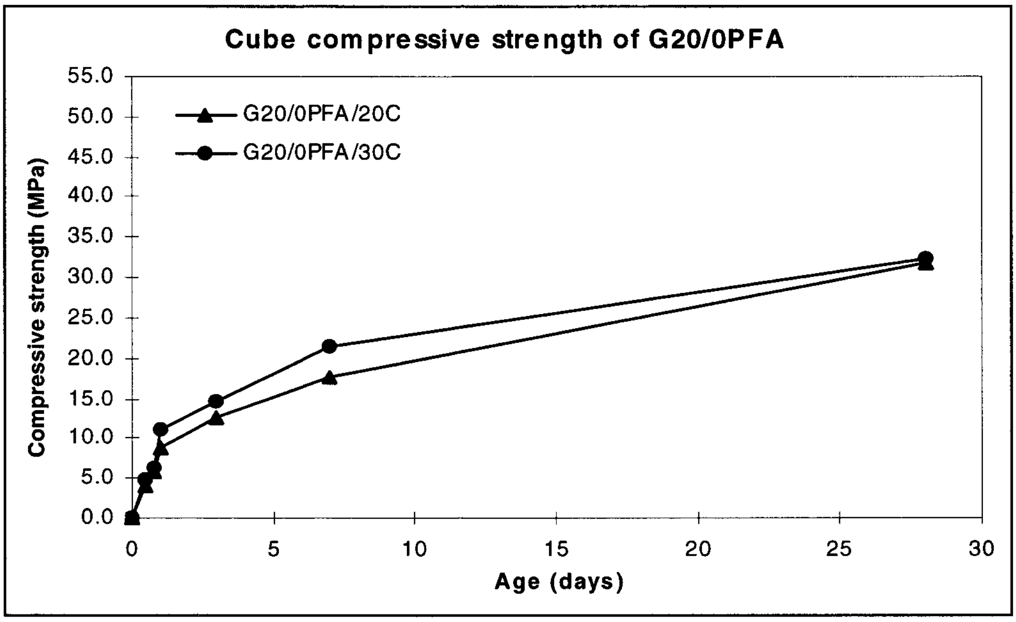
Batch no.	Age at which hammering test carried out	Static modulus (GPa)
G20/0PFA/20C	12 hrs	(not measured)
	18 hrs	8.5
	24 hrs	10.3
	3 days	15.4
	7 days	18.1
	28 days	23.1
G20/0PFA/30C	12 hrs	(not measured)
	18 hrs	8.5
	24 hrs	10.3
	3 days	17.4
	7 days	16.2
	28 days	23.7
G20/25PFA/20C	12 hrs	(not measured)
	18 hrs	(not measured)
	24 hrs	11.7
	3 days	14.2
	7 days	15.9
	28 days	20.4
G20/25PFA/30C	12 hrs	(not measured)
	18 hrs	10.3
	24 hrs	9.4
	3 days	13.3
	7 days	19.2
	28 days	23.8

Table 4.13 - Static moduli of G30 concretes

Batch no.	Age at which hammering test carried out	Static modulus (GPa)
G30/0PFA/20C	12 hrs	(not measured)
	18 hrs	14.7
	24 hrs	13.4
	3 days	19.0
	7 days	21.5
	28 days	23.6
G30/0PFA/30C	12 hrs	(not measured)
	18 hrs	15.2
	24 hrs	18.0
	3 days	19.4
	7 days	24.3
	28 days	25.7
G30/25PFA/20C	12 hrs	(not measured)
	18 hrs	13.4
	24 hrs	15.6
	3 days	17.6
	7 days	19.0
	28 days	22.8
G30/25PFA/30C	12 hrs	(not measured)
	18 hrs	14.4
	24 hrs	17.7
	3 days	19.5
	7 days	21.6
	28 days	25.3

Table 4.14 - Static moduli of G40 concretes

Batch no.	Age at which hammering test carried out	Static modulus (GPa)
G40/0PFA/20C	12 hrs	(not measured)
	18 hrs	14.5
	24 hrs	17.6
	3 days	17.2
	7 days	19.5
	28 days	26.5
G40/0PFA/30C	12 hrs	(not measured)
	18 hrs	16.4
	24 hrs	16.8
	3 days	18.1
	7 days	19.4
	28 days	25.6
G40/25PFA/20C	12 hrs	(not measured)
	18 hrs	6.5
	24 hrs	10.9
	3 days	17.9
	7 days	23.9
	28 days	24.0
G40/25PFA/30C	12 hrs	(not measured)
	18 hrs	15.1
	24 hrs	18.4
	3 days	18.4
	7 days	21.2
	28 days	25.0



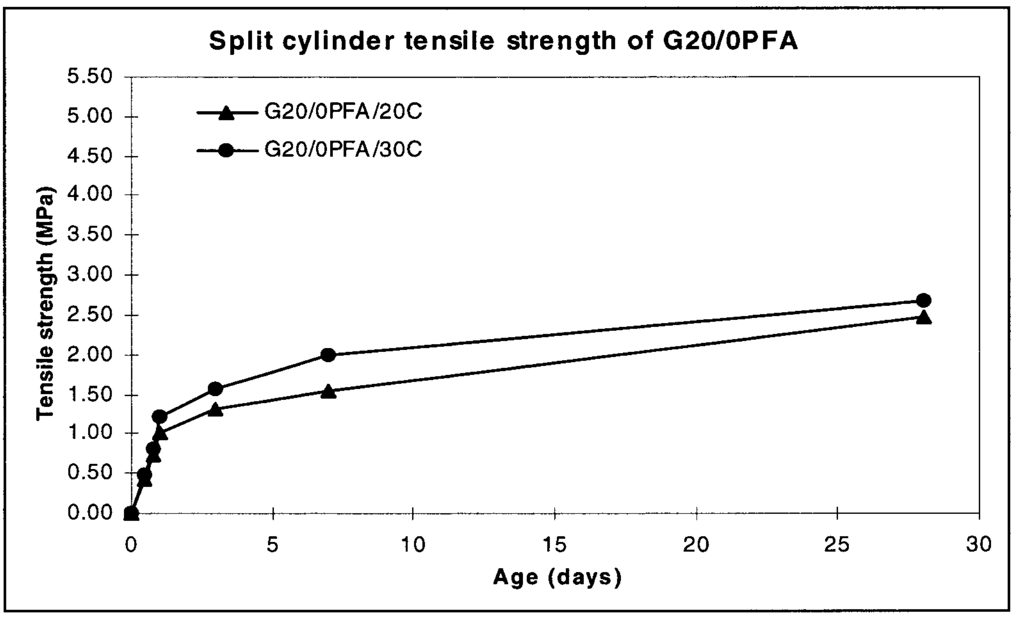


Figure 4.8 - Compressive and tensile strength of G20/0PFA

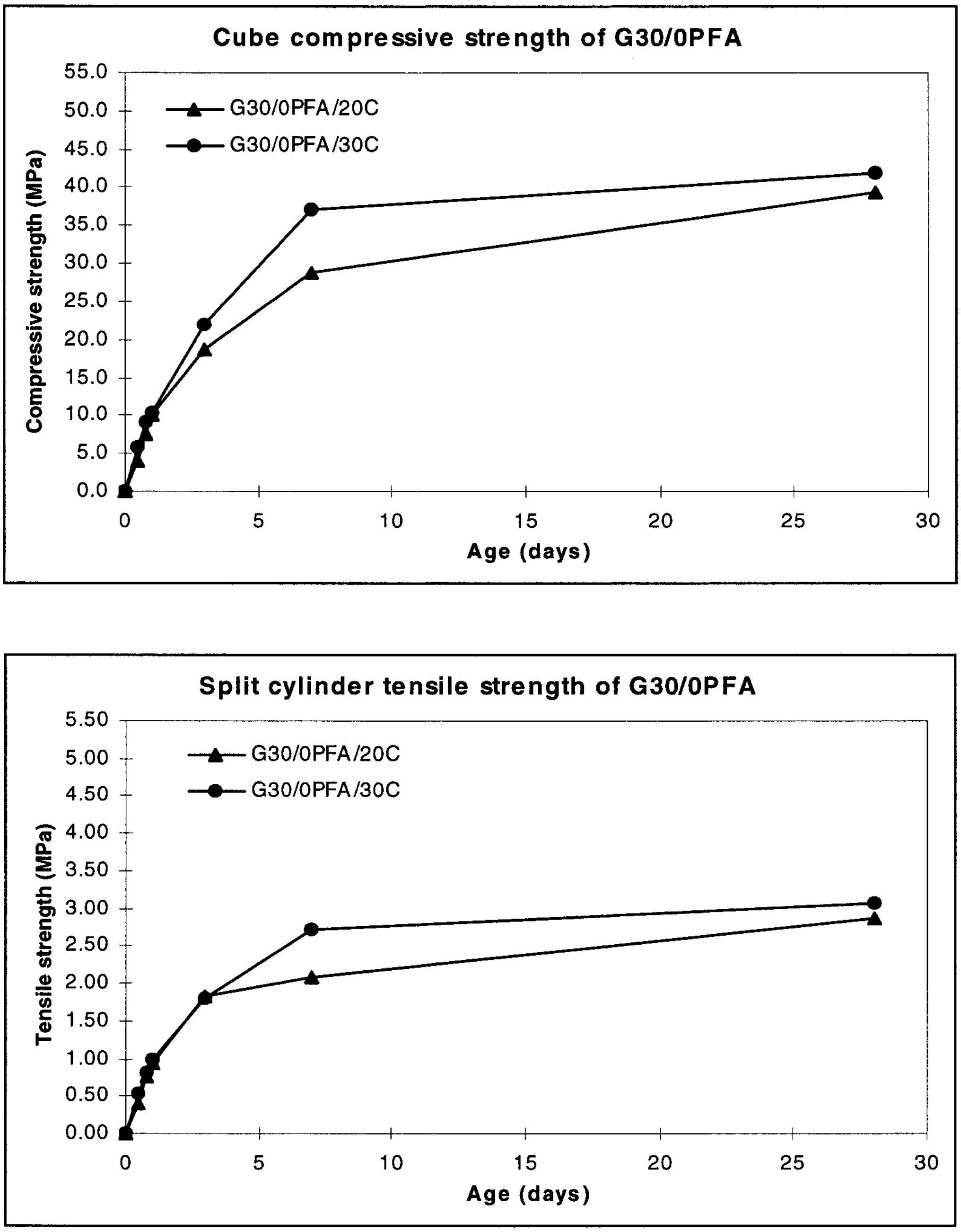
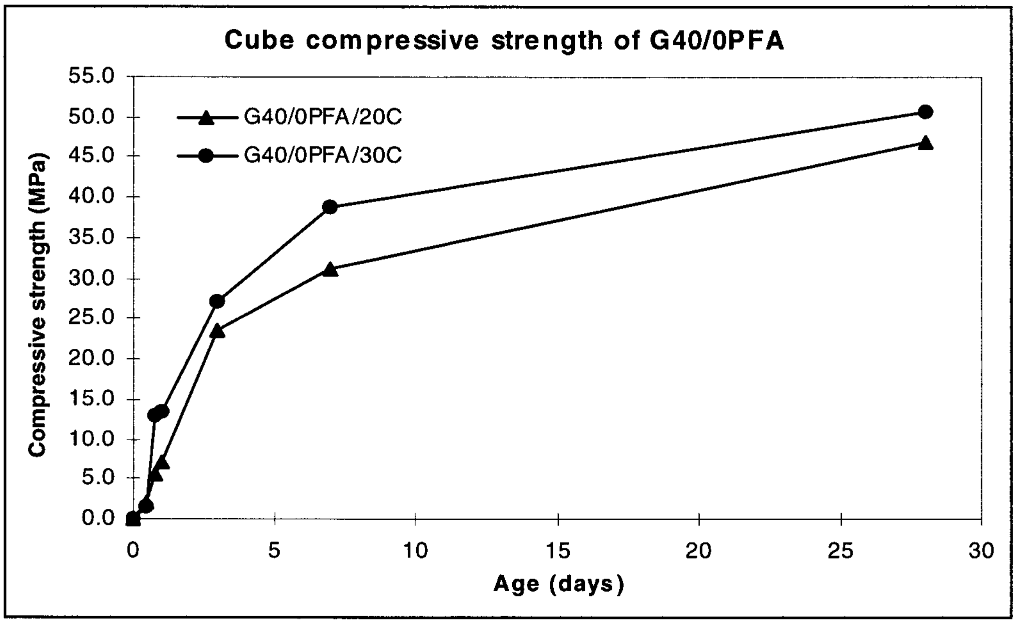


Figure 4.9 - Compressive and tensile strength of G30/0PFA



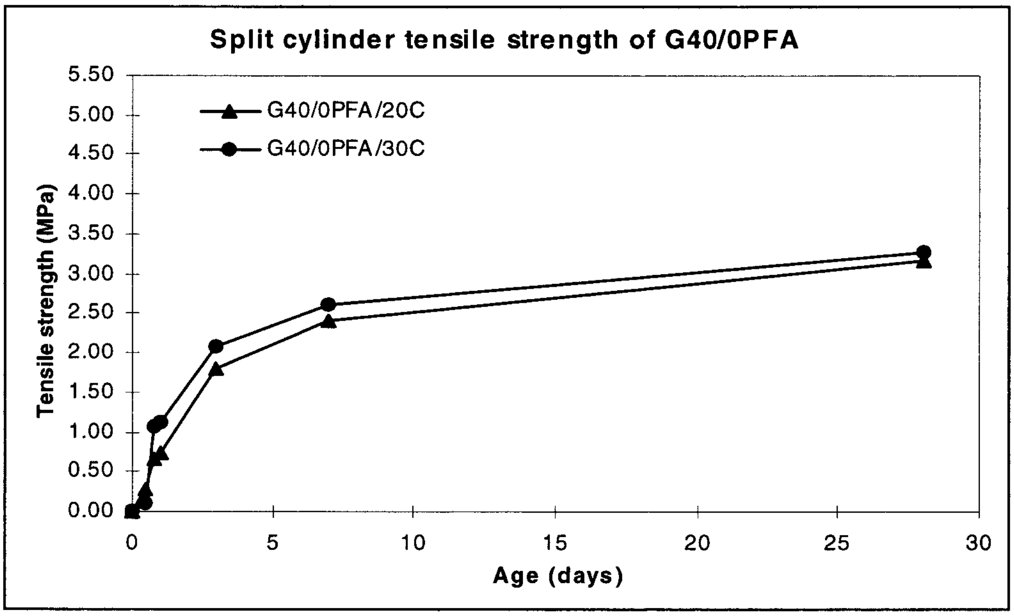
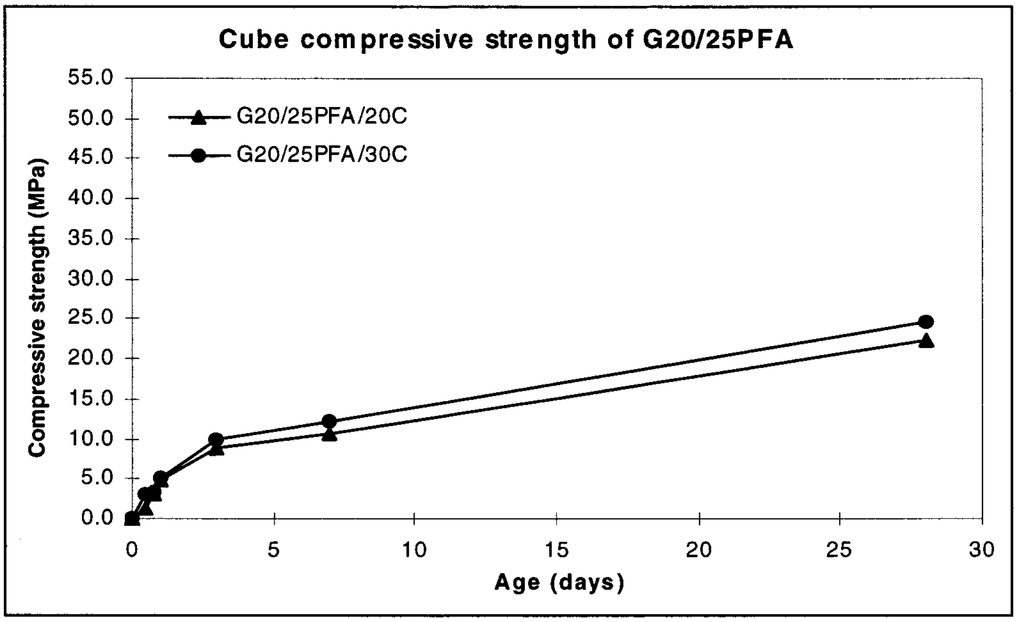


Figure 4.10 - Compressive and tensile strength of G40/0PFA



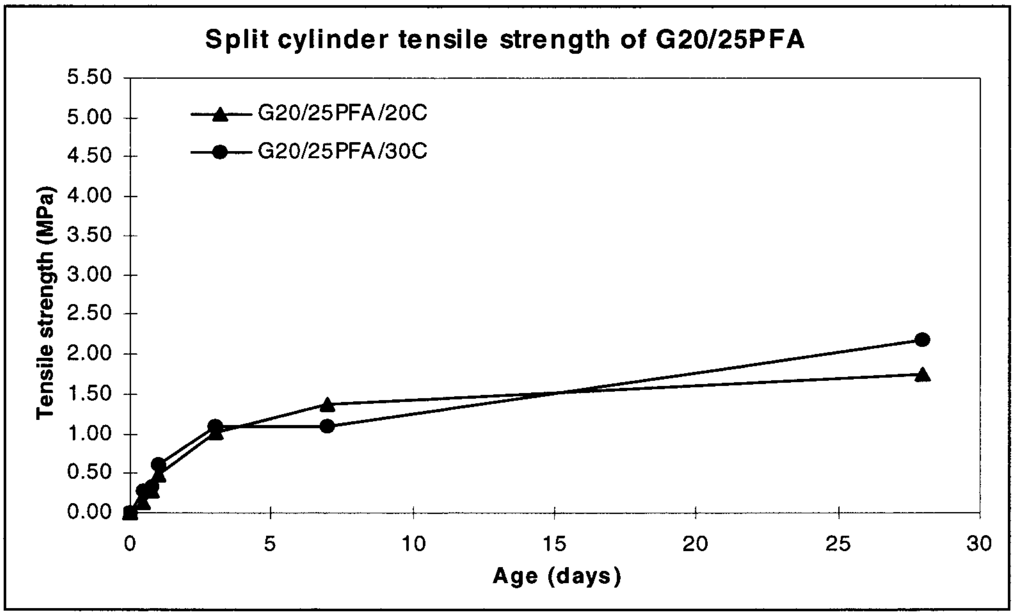
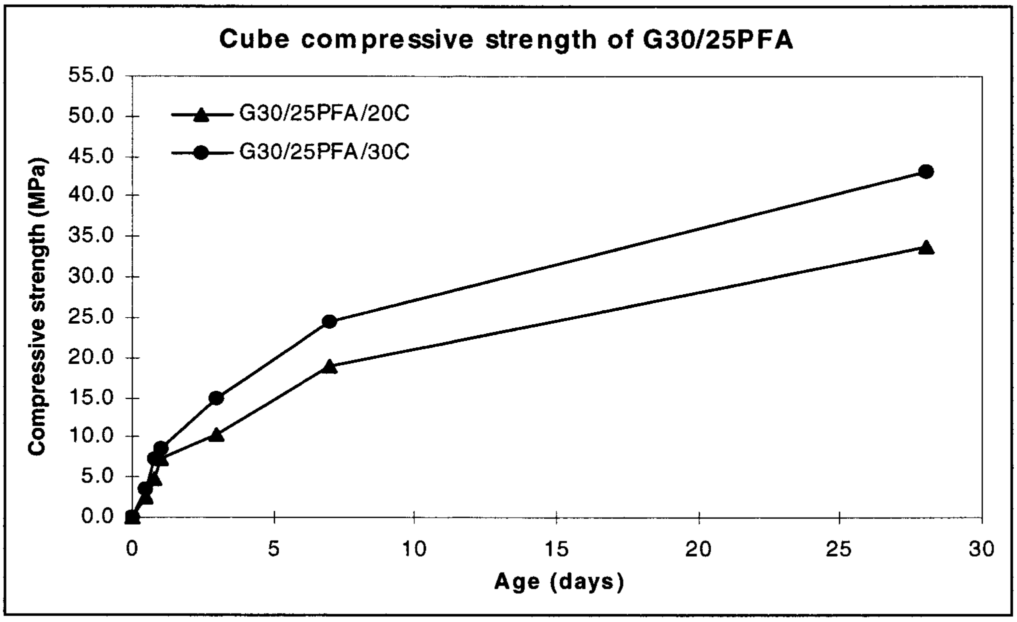


Figure 4.11 - Compressive and tensile strength of G20/25PFA



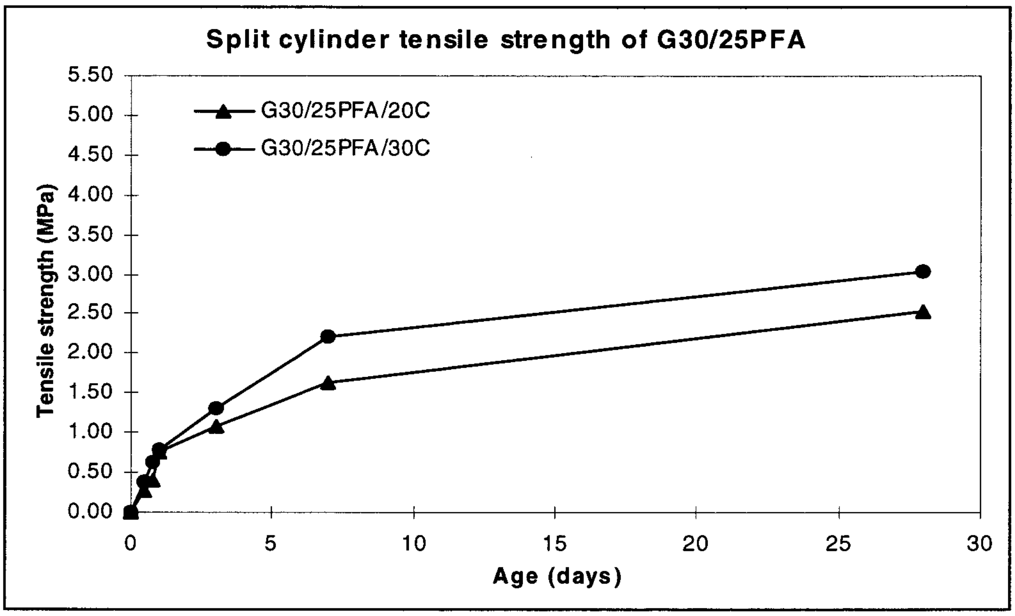
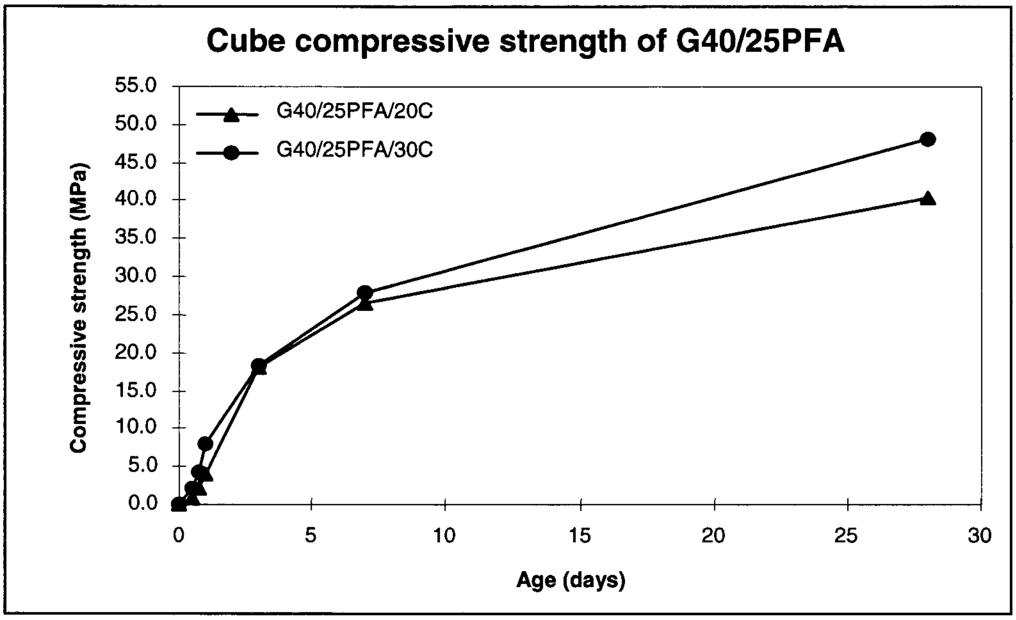


Figure 4.12 - Compressive and tensile strength of G30/25PFA



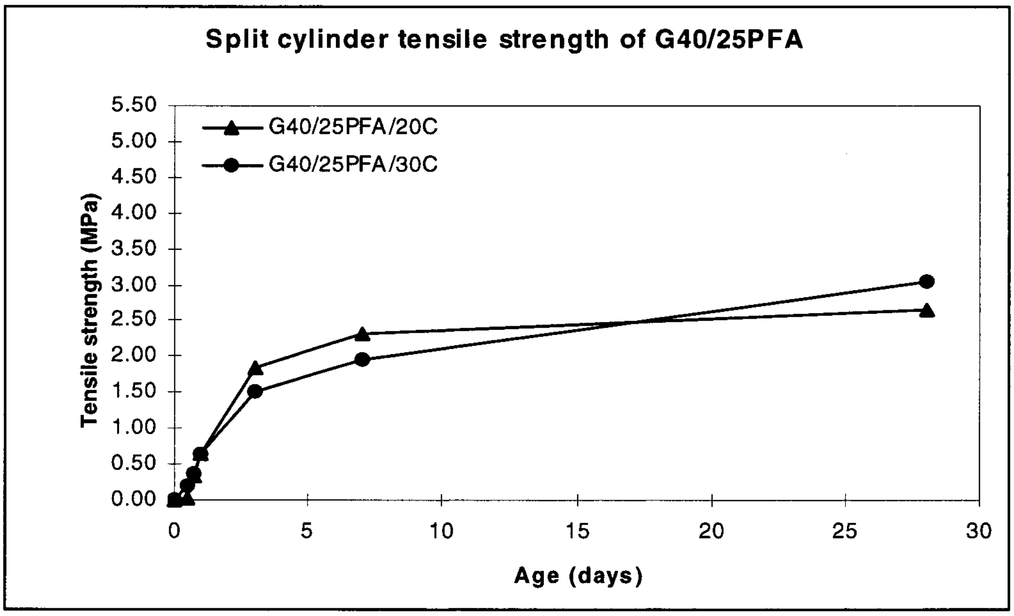


Figure 4.13 - Compressive and tensile strength of G40/25PFA

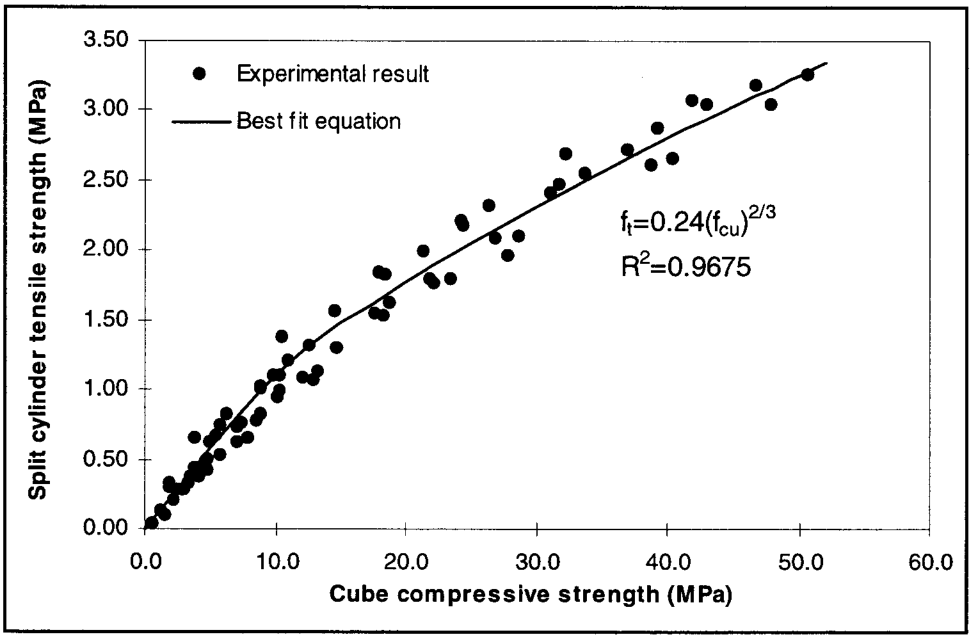


Figure 4.14 - Split cylinder tensile strength against cube compressive strength

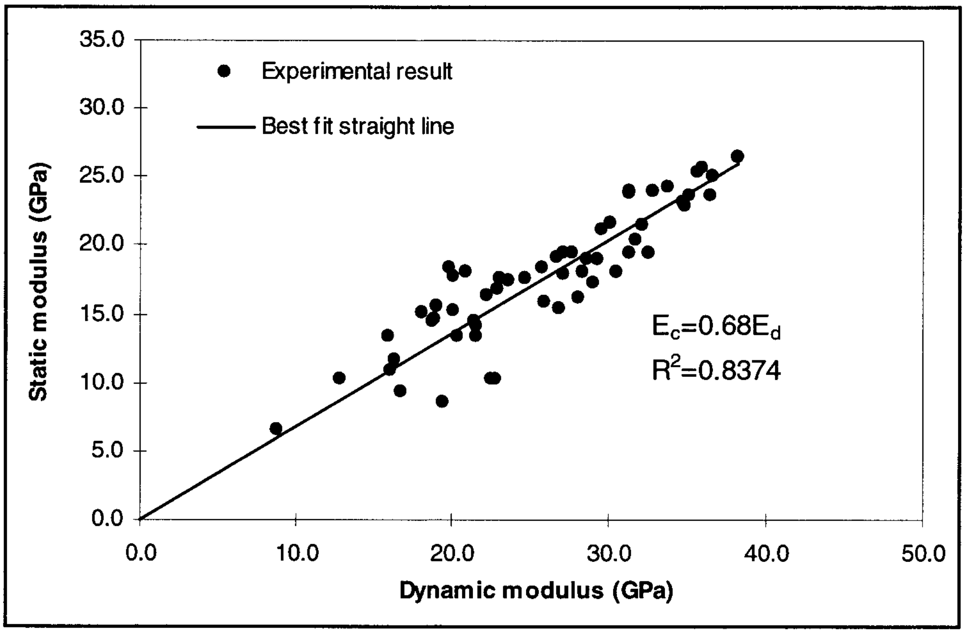


Figure 4.15 - Static modulus against dynamic modulus

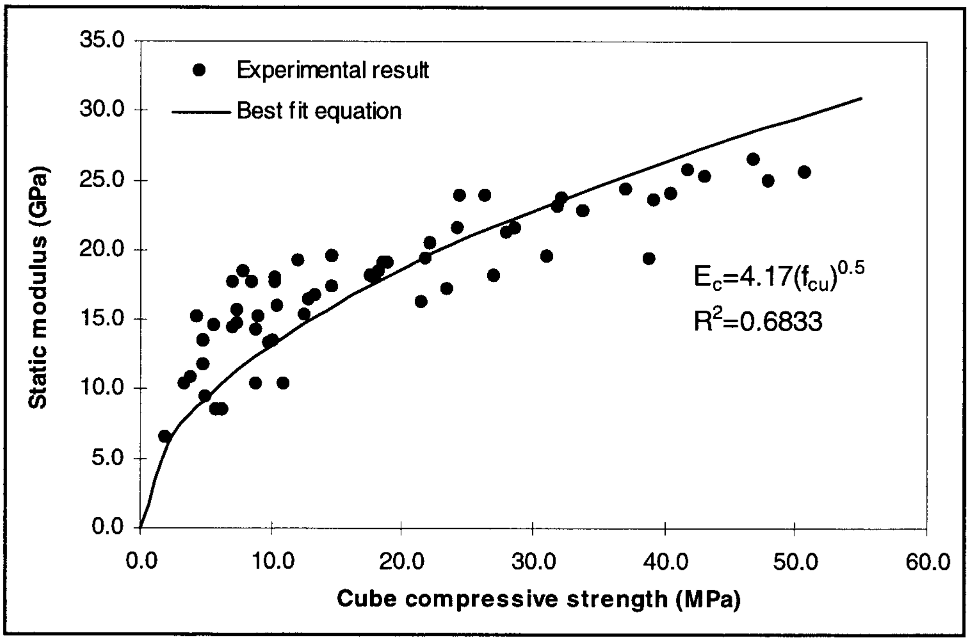


Figure 4.16 - Static modulus against cube compressive strength

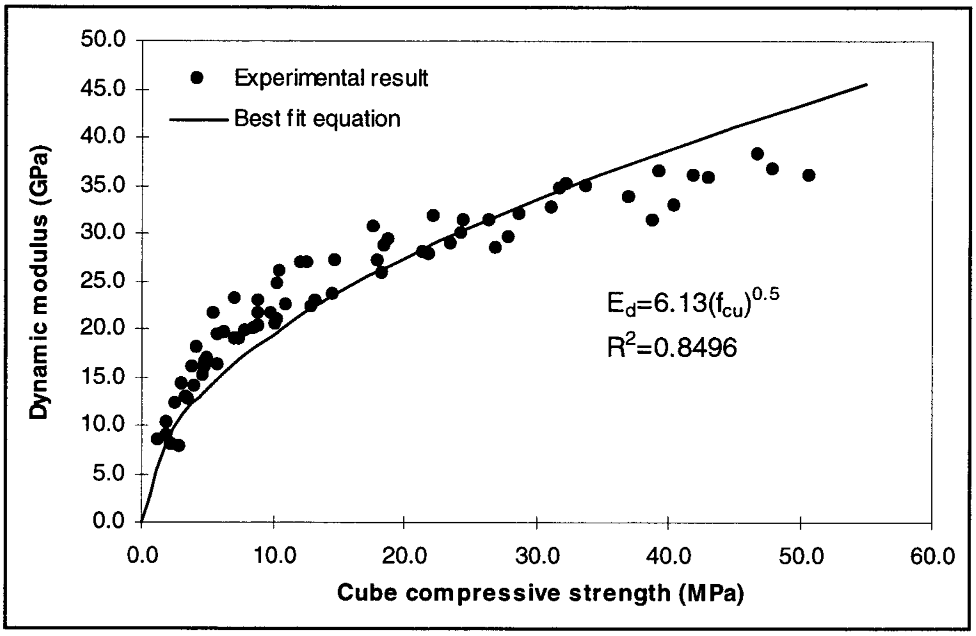


Figure 4.17 - Dynamic modulus against cube compressive strength

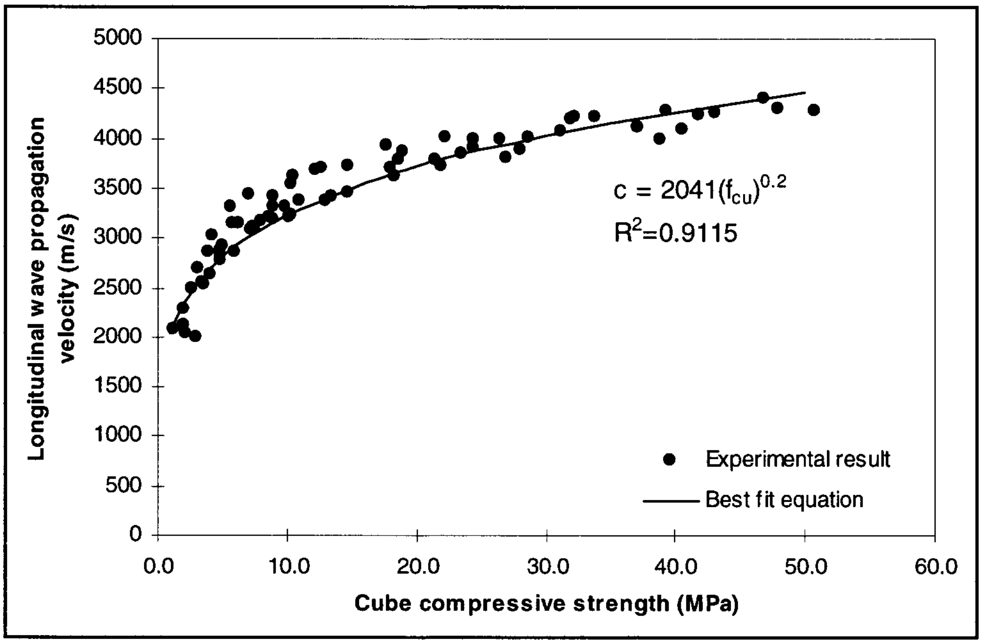


Figure 4.18 - Longitudinal wave propagation velocity against cube compressive strength

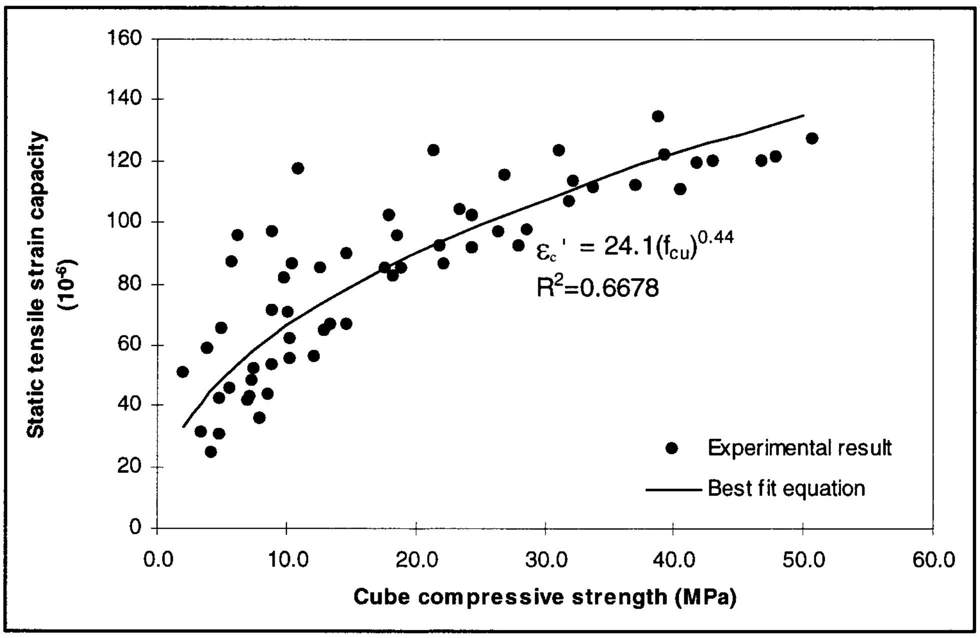


Figure 4.19 - Static tensile strain capacity against cube compressive strength

4.4 Results of Hammering Tests and Integrity Tests

Each set of specimens for the hammering tests consisted of five prisms which were cast from the same batch of concrete and cured under the same designated curing temperature. One of the five prisms was treated as a control specimen and was not subjected to any hammer blow or vibration. Among the other four prisms, one was subjected to a number of hammer blows with increasing intensity until at least one crack appeared to determine the vibration intensity required to cause cracking. The remaining three prisms were then each subjected to one hammer blow in such a way that at least one prism but not all would be cracked. In some cases, however, none of the three prisms was cracked after the hammering test. In such case, the three prisms were re-hammered until at least one of them was cracked. In some cases, therefore, among the four prisms subjected to hammering, there could be two or more prisms subjected to more than one hammer blows.

After the hammering tests, the prisms, including the one which acted as control specimen, were continued to be cured at their designated curing temperature until the age of 28-day by which time they were tested for their integrity in order to evaluate the effects of the shock vibration applied to them. Any possible self-healing was allowed for by placing the prisms with their longitudinal axis vertical during curing so that any cracks formed by the shock vibration were kept closed.

The results of the hammering and integrity tests are presented in Tables 4.15 to 4.22, Tables 4.23 to 4.30 and Tables 4.31 to 4.38 for the G20, G30 and G40 concretes respectively.

When studying the test results, it should be borne in mind that from each batch of concrete, fifteen prisms for hammering tests at three different ages (five prisms for each age) were cast (since there were only fifteen moulds, only fifteen prism could be cast from one batch of concrete). As one control prism was cast for hammering test at each age, there were all together three control specimens cast from each batch of concrete. Although each control specimen corresponded to hammering test at a certain age, the three control specimens cast from the same batch of concrete were actually identical as they were cured under the same conditions and were not subjected to any hammering. However, due to random variation of the strength properties, the three control specimens yielded somewhat slightly different integrity test results. Since the three control specimens were cast from the same batch of concrete, it was found difficult to assign any particular control specimen to hammering test at any particular age. To overcome this difficulty, the integrity test results of the three control specimens cast from the same batch of concrete were averaged and the results taken as those for the hammering tests at the three different ages. That is why in the integrity test results tabulated, prisms cast for hammering tests at different ages can have the same control specimen results (the averaged results of the three control specimens). In fact, this averaging of the control specimen results has the advantage of reducing the statistical variation of the test results of the control specimens which would be used as a datum when assessing the effects of the shock vibration applied to the other prisms.

In Tables 4.15 to 4.38, several peak particle velocity results separated by semi-colons are sometimes given for the same prism. This is because the prism had been subjected to more than one hammer blow during the hammering test; in the results presented, each peak particle velocity given is the intensity of the shock vibration applied at one hammer blow. Occasionally, a hyphen is given as the peak particle velocity of the shock vibration applied to the prism. This means that no result on vibration intensity was obtained for that particular prism during the hammering test. This was due mainly to the tight schedule of the testing programme and the fact that many of the tests had to be carried out at night time (sometimes after mid-night) when there was no technician to help handling the heavy specimens or operating the equipment. Accidents and/or mistakes were therefore prone to occur. Fortunately, none of the Research Assistants had any of their toes broken but some mistakes had been made during the hammering tests. In several cases, it was found after the hammering test was completed that the cable linking the accelerometer to the conditioning amplifier had not been properly connected and consequently, no signal was received by the data acquisition system during the hammering. In one other case, the data file storing the hammering test results was accidentally deleted before it was backed up. However, the worst thing happened in day time when the accelerometer which cost about HK$10,000 was broken during one hammering test (hammering test of G40/0PFA/20C at the age of 28 days). As a result, no peak particle velocity result was obtained for that particular whole set of prisms (see Table 4.33) and the accelerometer had to be replaced by a new one. In another case of G30/25PFA/20C subjected to hammering test at the age of 8 hours, one prism (prism no.4) was damaged accidentally during the direct tensile test and no result for its tensile strength was thus obtained; this is indicated by a question mark in the corresponding data entry in Table 4.27. Nevertheless, the number of prisms affected is actually quite small, compared to the total number of prisms tested.

After the application of each hammer blow to the test prism, the prism was inspected for any transverse cracks formed on the concrete. Transverse cracks observed are denoted either as "*" or "#" against the peak particle velocity of the shock wave applied in Tables 4.15 - 4.38. A "*" indicates that a transverse crack cutting through the whole section of the concrete prism had been formed, i.e. the concrete prism had been broken into two pieces. A "#" indicates that one or more transverse cracks on the surface of the concrete prism had been observed but the concrete prism still remained as one piece, i.e. the transverse cracks formed had not cut through the whole section of the prism. Although the exact locations of the transverse cracks were not measured, they were all found to be located within the middle-third of the length of the prisms. This reveals that the tensile stresses induced by the shock wave applied were highest near the middle of the prisms.

The hammering test results are quite erratic and therefore not easy to interpret. Since it was not possible to continuously adjust the vibration intensity until the concrete failed, the vibration resistance of the concrete could not be directly obtained. Nevertheless, from the intensity of the shock vibration applied which had not caused any damage to the concrete and the intensity of the shock vibration applied which had caused damage to the concrete, the vibration resistance of the concrete may be indirectly determined. Detailed analysis of the test results is presented in the next two chapters.

Table 4.15 - Hammering and integrity test results for G20/0PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	150	2.13	1.99	34.0	30.9
	2	230	1.44		36.4	
	3	40	2.04		34.3	
	4	330	1.96		32.9	
4 hrs	1	240	1.70	1.99	28.8	30.9
	2	170	2.20		25.0	
	3	380	1.76		32.4	
	4	95	1.90		32.9	
6 hrs	1	110	2.10	1.99	33.2	30.9
	2	290	1.30		32.2	
	3	300	2.02		32.0	
	4	140	2.16		32.8	
8 hrs	1	420	1.82	1.67	29.3	27.0
	2	410	2.11		32.5	
	3	140	2.27		31.6	
	4	180	2.02		29.8	
10 hrs	1	480	1.96	1.67	32.1	27.0
	2	250	2.04		29.6	
	3	160	1.89		28.4	
	4	290	1.93		29.0	
12 hrs	1	150	2.11	1.67	29.7	27.0
	2	230	2.02		29.7	
	3	410	2.11		32.1	
	4	200	2.09		31.0	

Table 4.16 - Hammering and integrity test results for G20/0PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	21;90	1.76	1.82	31.1	28.8
	2	340	2.22		30.9	
	3	240	1.78		28.0	
	4	550	1.91		30.2	
4 hrs	1	86;120;160	1.49	1.96	30.2	28.3
	2	91	0.56		29.0	
	3	79	1.11		29.1	
	4	56	1.87		29.9	
6 hrs	1	74	1.93	1.96	29.9	28.3
	2	180	1.67		32.9	
	3	110	1.87		30.8	
	4	79	2.13		31.1	
8 hrs	1	170	1.80	1.82	28.8	28.8
	2	140	1.91		27.9	
	3	140	1.93		27.7	
	4	180	1.78		27.8	
10 hrs	1	820	1.69	1.99	29.5	28.2
	2	460;800	2.13		28.7	
	3	750	2.24		29.9	
	4	680	1.87		29.1	
12 hrs	1	740	2.04	1.92	28.0	27.8
	2	650	2.07		29.2	
	3	760;860	1.91		31.0	
	4	510;640	1.64		29.6	

Table 4.17 - Hammering and integrity test results for G20/0PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	460;600;680#	0.44	2.07	29.7	28.6
	2	640	1.89		29.6	
	3	610*	0.00		28.9	
	4	570	0.51		30.4	
18 hrs	1	670;910;960*	0.00	2.07	26.7	28.6
	2	590	1.67		26.5	
	3	920	1.89		27.8	
	4	890#	0.18		25.5	
24 hrs	1	120;170;420*	0.00	2.00	34.4	33.9
	2	670	2.36		33.2	
	3	350*	0.00		32.9	
	4	410;700	1.93		33.2	
3 days	1	1500	1.24	2.07	28.7	28.6
	2	840;1700#	0.13		28.3	
	3	1500	1.87		29.3	
	4	700;1100;1200*	0.00		30.2	
7 days	1	1200	2.44	2.00	32.6	33.9
	2	1100	2.04		31.1	
	3	1100	2.29		33.2	
	4	1100;1300;1600#	0.00		33.1	
28 days	1	1800;1900;3600*	0.00	2.00	33.0	33.9
	2	1500;1700	2.11		32.3	
	3	1800;2000;2300	2.11		33.8	
	4	2300#	1.33		32.7	

Table 4.18 - Hammering and integrity test results for G20/0PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	780;1100;1200#	0.00	1.92	29.0	27.8
	2	-	1.60		27.2	
	3	510	1.80		30.1	
	4	580;600;910	1.80		29.8	
18 hrs	1	660;730;990*	0.00	1.82	30.1	28.8
	2	780	1.89		29.8	
	3	810;830;1600*	0.00		31.0	
	4	1300	2.02		29.7	
24 hrs	1	710	2.27	2.19	33.4	29.7
	2	740;990;1300*	0.00		33.2	
	3	840	1.91		32.2	
	4	700;1100;1200*	0.00		33.3	
3 days	1	2300;2500;2900*	0.00	1.99	29.6	28.3
	2	900;1000;2100	2.00		29.7	
	3	1700;2100;2700*	0.00		28.8	
	4	1100;2100;2500	1.58		28.5	
7 days	1	600;1800;2900*	0.00	2.19	30.7	29.7
	2	1200	2.11		28.7	
	3	2300	2.00		29.3	
	4	3100	2.04		31.1	
28 days	1	3400	1.89	2.19	30.1	29.7
	2	2100	1.89		30.3	
	3	1800;2100;2900*	0.00		31.7	
	4	3300	1.73		31.3	

Table 4.19 - Hammering and integrity test results for G20/25PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	-	1.60	1.55	25.3	20.9
	2	28	1.10		24.8	
	3	67	1.60		25.3	
	4	35	1.70		25.2	
4 hrs	1	150	1.56	1.67	26.1	21.2
	2	70	1.67		21.7	
	3	300	1.47		24.3	
	4	700	1.60		24.5	
6 hrs	1	130	1.13	1.67	22.7	21.2
	2	60	1.62		22.2	
	3	270	1.47		19.7	
	4	210	0.71		22.4	
8 hrs	1	200	1.64	1.60	22.2	24.4
	2	150	1.78		20.1	
	3	1700	0.53		20.7	
	4	1700	1.38		22.9	
10 hrs	1	560	1.73	1.71	22.5	24.6
	2	520	0.27		21.3	
	3	1800	0.98		18.4	
	4	2100	1.60		20.5	
12 hrs	1	660	0.93	1.71	20.7	24.6
	2	560	1.38		21.2	
	3	1600	1.11		22.9	
	4	3400	1.22		21.6	

Table 4.20 - Hammering and integrity test results for G20/25PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	23	1.96	1.57	27.6	24.4
	2	31	1.69		27.3	
	3	25	1.82		24.2	
	4	56	1.76		24.8	
4 hrs	1	36;38	1.98	1.73	31.6	29.0
	2	36	1.91		34.2	
	3	150	1.87		33.3	
	4	140	1.76		34.2	
6 hrs	1	210;310	1.89	1.73	32.2	29.0
	2	180;210	2.00		33.7	
	3	170	1.93		32.8	
	4	250	1.87		32.7	
8 hrs	1	230	1.44	1.57	26.3	24.4
	2	140	1.84		25.9	
	3	580	1.89		26.9	
	4	1100	1.69		26.9	
10 hrs	1	290;300;380	0.93	1.91	29.6	28.1
	2	360;410	1.67		29.9	
	3	370	1.42		28.9	
	4	310	0.51		28.3	
12 hrs	1	300;310	1.67	1.91	27.8	28.1
	2	310;370;400	1.13		29.6	
	3	250	1.76		30.2	
	4	360;460	0.22		30.8	

Table 4.21 - Hammering and integrity test results for G20/25PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	340#	0.00	1.71	20.6	24.6
	2	980	1.09		20.6	
	3	-*	0.00		20.6	
	4	-*	0.00		21.4	
18 hrs	1	510;1200#	0.98	1.67	20.2	21.2
	2	520;920	1.24		20.1	
	3	2000;2300	1.18		20.6	
	4	1600	1.38		21.6	
24 hrs	1	550;750#	0.90	1.55	22.4	20.9
	2	510;620	1.60		22.0	
	3	480;1200	0.90		21.0	
	4	760;1200;1500#	0.60		21.9	
3 days	1	1400	1.47	1.60	18.7	24.4
	2	1100	1.33		21.9	
	3	2800*	0.00		22.6	
	4	2300#	0.76		21.3	
7 days	1	2800*	0.00	1.70	23.3	21.7
	2	2200*	0.00		22.9	
	3	1000	0.93		22.9	
	4	2600	1.70		21.7	
28 days	1	3100	1.60	1.70	23.7	21.7
	2	1400	1.60		24.1	
	3	2300#	1.10		23.0	
	4	2200	1.60		23.4	

Table 4.22 - Hammering and integrity test results for G20/25PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	450;570;700	1.42	1.91	30.3	28.1
	2	440	1.67		28.2	
	3	390	1.76		29.8	
	4	450	1.56		29.5	
18 hrs	1	700	1.62	1.73	29.8	29.0
	2	590;650;820	1.29		28.2	
	3	550;880	1.18		29.3	
	4	930	1.87		28.6	
24 hrs	1	640;710;1500	1.40	1.66	23.1	24.3
	2	860	1.20		23.7	
	3	740	1.36		23.6	
	4	1100;1400;1700	1.60		24.2	
3 days	1	2600#	0.71	1.57	24.9	24.4
	2	1200	0.62		24.0	
	3	2200*	0.00		22.5	
	4	2500*	0.00		24.8	
7 days	1	3500#	0.33	1.66	26.5	24.3
	2	2300	1.27		26.3	
	3	1800	1.91		24.4	
	4	1700	1.62		26.4	
28 days	1	3100*	0.00	1.66	26.1	24.3
	2	2900	1.02		25.7	
	3	3200#	0.56		25.6	
	4	2900*	0.00		26.0	

Table 4.23 - Hammering and integrity test results for G30/0PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	24;100	2.31	2.13	40.8	39.7
	2	100	2.18		41.9	
	3	92	2.16		42.1	
	4	-	2.02		40.4	
4 hrs	1	74;88	2.07	2.13	39.8	39.7
	2	320	1.82		35.4	
	3	410	2.33		34.0	
	4	500	2.00		37.5	
6 hrs	1	100;180	1.69	1.89	32.2	39.3
	2	160;240	1.98		35.8	
	3	600	1.89		36.3	
	4	670	1.80		33.5	
8 hrs	1	180;280	2.11	1.93	34.6	39.6
	2	340;390	1.80		38.7	
	3	500	1.13		38.4	
	4	490	1.89		37.5	
10 hrs	1	220;330;400	1.89	1.89	39.4	41.2
	2	410	1.22		40.8	
	3	370;540	2.24		41.0	
	4	350	2.16		40.2	
12 hrs	1	490;560	1.44	1.93	41.5	41.5
	2	500;820	2.20		38.5	
	3	300	2.02		40.0	
	4	530	1.67		41.0	

Table 4.24 - Hammering and integrity test results for G30/0PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	170;180	2.07	2.09	44.0	42.3
	2	150	2.18		43.3	
	3	46	2.02		45.0	
	4	29	1.78		41.5	
4 hrs	1	120;260;280	2.04	2.09	44.6	42.3
	2	170	1.80		38.6	
	3	230	1.89		40.4	
	4	300	1.96		41.1	
6 hrs	1	340;350;440	1.98	2.11	37.5	40.8
	2	210;230	2.18		42.7	
	3	210	2.07		39.1	
	4	310	2.20		40.9	
8 hrs	1	340;410;430	2.11	2.11	41.3	40.8
	2	600	1.84		40.5	
	3	350	1.89		40.6	
	4	390	2.16		38.5	
10 hrs	1	570;700;770	1.24	1.98	37.7	42.8
	2	560	1.89		38.2	
	3	610;830	1.69		39.4	
	4	710	2.36		39.9	
12 hrs	1	1100;1200	2.20	1.98	37.8	42.8
	2	430;780	2.00		39.9	
	3	780	1.67		39.0	
	4	980	2.07		38.2	

Table 4.25 - Hammering and integrity test results for G30/0PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	310;460;800#	0.71	1.93	41.2	41.2
	2	760	2.00		40.8	
	3	510	1.87		40.0	
	4	730	1.80		40.4	
18 hrs	1	810;1000*	0.00	2.13	37.5	39.7
	2	430	1.89		38.7	
	3	700	2.02		38.6	
	4	730;1300#	0.38		37.2	
24 hrs	1	980;1400;1600#	0.53	1.93	38.7	40.0
	2	1600	1.36		37.5	
	3	1000	1.71		38.2	
	4	990;1100#	0.36		37.2	
3 days	1	890;1100;1100*	0.00	2.02	36.4	39.9
	2	1000#	0.36		38.1	
	3	620	1.89		40.4	
	4	740	2.58		38.7	
7 days	1	1300;1400;1600*	0.00	2.02	37.4	39.9
	2	1700#	0.47		36.6	
	3	1100	1.89		38.3	
	4	950	2.33		39.9	
28 days	1	1500;1800;1900*	0.00	2.02	38.6	39.9
	2	1400*	0.00		38.6	
	3	1200	1.80		38.5	
	4	1100;1500	0.47		38.1	

Table 4.26 - Hammering and integrity test results for G30/0PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	590;800;1100#	0.36	1.98	37.8	42.8
	2	1600	2.09		41.0	
	3	810	2.00		39.8	
	4	1900*	0.00		36.5	
18 hrs	1	990	1.67	2.09	39.2	42.3
	2	-*	0.00		38.6	
	3	900;1200#	0.38		36.8	
	4	690	1.80		40.3	
24 hrs	1	1500;1600*	0.00	2.11	40.5	40.8
	2	1500	1.87		39.5	
	3	940	1.56		39.7	
	4	1400;1500#	0.22		39.4	
3 days	1	1300;1400*	0.00	2.04	40.0	41.0
	2	1000;1500*	0.00		39.5	
	3	980	2.11		40.5	
	4	1100	1.93		37.7	
7 days	1	1500;1700#	0.42	2.04	38.1	41.0
	2	1500;1600*	0.00		35.7	
	3	720	1.71		39.6	
	4	1700	1.20		38.0	
28 days	1	1100;1500;1800*	0.00	2.04	38.6	41.0
	2	1400#	0.38		39.2	
	3	1500#	0.67		39.7	
	4	1900*	0.00		39.8	

Table 4.27 - Hammering and integrity test results for G30/25PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	22;90	1.60	1.49	28.8	29.5
	2	22	1.33		27.6	
	3	30	1.51		29.7	
	4	32	1.13		26.7	
4 hrs	1	230;440	1.62	1.71	31.4	32.7
	2	490	1.69		31.9	
	3	280	1.80		33.5	
	4	620	1.62		32.0	
6 hrs	1	200	1.64	1.71	32.8	32.7
	2	84;190;260	2.16		33.6	
	3	290	1.78		31.6	
	4	410	1.89		32.8	
8 hrs	1	210;250;280	1.89	1.71	34.0	32.7
	2	380	1.60		33.6	
	3	320	1.78		31.7	
	4	400;480	?		33.9	
10 hrs	1	340;400;540	1.42	1.49	28.7	29.5
	2	620	1.33		27.9	
	3	460	1.58		30.5	
	4	650;790	1.31		29.2	
12 hrs	1	260;480;560	1.42	1.49	29.1	29.5
	2	370	1.56		29.3	
	3	470	1.29		27.4	
	4	860	1.44		28.9	

Table 4.28 - Hammering and integrity test results for G30/25PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	20;47	1.76	1.71	40.8	38.8
	2	36	1.93		43.1	
	3	29	2.02		40.1	
	4	69	1.80		41.4	
4 hrs	1	180;240;350	1.93	1.71	40.7	39.0
	2	460	1.84		39.4	
	3	240	1.73		38.4	
	4	370	1.84		39.6	
6 hrs	1	410;490;510	1.42	1.71	37.8	39.0
	2	670	1.40		38.7	
	3	630	1.80		39.4	
	4	660	1.11		37.7	
8 hrs	1	520;960;1100	2.11	1.71	40.4	39.0
	2	860	1.82		37.5	
	3	790	1.64		38.4	
	4	860	0.71		39.2	
10 hrs	1	440;640;660	1.89	1.71	40.0	38.8
	2	760	1.71		39.2	
	3	450	1.60		38.6	
	4	730	1.78		38.6	
12 hrs	1	550;560;830	1.76	1.71	34.2	38.8
	2	580	1.82		38.7	
	3	500	1.78		37.3	
	4	670	1.96		35.5	

Table 4.29 - Hammering and integrity test results for G30/25PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	550;610;1100#	0.22	1.87	31.2	33.1
	2	680	1.71		31.5	
	3	580	1.78		33.8	
	4	1000	1.56		31.8	
18 hrs	1	750;850#	0.40	1.62	37.7	34.8
	2	760	1.76		36.6	
	3	1000	1.02		34.1	
	4	1000	1.82		35.2	
24 hrs	1	870;1300#	0.47	1.87	32.9	33.1
	2	1300	1.62		31.0	
	3	890	1.69		31.8	
	4	1500*	0.00		31.2	
3 days	1	1100;1300;2000*	0.00	1.62	32.4	34.8
	2	1400	1.76		30.7	
	3	940	1.44		34.5	
	4	1700*	0.00		34.0	
7 days	1	1300;1400*	0.00	1.87	32.2	33.1
	2	1200#	0.40		31.1	
	3	810	1.71		32.7	
	4	1400;1500;1700*	0.00		32.8	
28 days	1	1600;1900;1900	1.27	1.62	34.0	34.8
	2	1400;1600;1700#	0.00		33.7	
	3	1400	1.44		33.4	
	4	1500#	0.56		33.4	

Table 4.30 - Hammering and integrity test results for G30/25PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	680;700	1.38	1.78	35.5	38.5
	2	430;640*	0.00		33.8	
	3	450	1.67		36.8	
	4	410	1.44		35.1	
18 hrs	1	910;1100;1400#	0.38	1.78	38.1	38.5
	2	980	1.04		40.5	
	3	780	1.71		39.1	
	4	1300#	0.33		39.2	
24 hrs	1	1300;1600;1700#	0.71	1.80	39.5	39.1
	2	1200	1.09		38.4	
	3	1300	1.82		40.2	
	4	1700#	0.36		38.4	
3 days	1	1300;1400#	0.33	1.78	33.3	38.5
	2	1700*	0.00		31.6	
	3	900	1.44		38.7	
	4	1500;1600#	0.33		42.9	
7 days	1	1300;1400;2000#	0.56	1.80	38.7	39.1
	2	1300	0.40		35.9	
	3	1300	1.64		37.4	
	4	1500;1600*	0.00		38.2	
28 days	1	1700;2000;2200*	0.00	1.80	39.7	39.1
	2	1600	0.78		39.5	
	3	1400*	0.00		39.1	
	4	980	1.71		39.1	

Table 4.31 - Hammering and integrity test results for G40/0PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	49;98;110	2.36	2.84	46.8	47.0
	2	38;45;53	2.53		49.1	
	3	28;88	2.18		50.0	
	4	100;130	2.36		51.2	
4 hrs	1	74;80	2.49	2.40	52.5	50.0
	2	50;110	2.13		51.6	
	3	74;98;270	2.36		54.9	
	4	150;180;190	1.47		51.9	
6 hrs	1	290;350;420	2.58	2.11	51.1	46.6
	2	-	1.78		49.2	
	3	100;110;130	1.89		44.1	
	4	110	2.33		47.1	
8 hrs	1	130;150;330	2.44	2.47	52.1	49.9
	2	150;210;220	2.56		50.1	
	3	160;180;180	2.42		53.3	
	4	430;450	2.44		50.3	
10 hrs	1	230;290;300	2.42	2.11	49.5	46.6
	2	210;250	2.51		54.5	
	3	170;380	2.53		58.3	
	4	240;390	2.38		58.4	
12 hrs	1	210;210;240	2.44	2.40	52.4	50.0
	2	230;390	2.27		51.9	
	3	200;200;210	2.67		51.2	
	4	320;420	1.96		51.4	

Table 4.32 - Hammering and integrity test results for G40/0PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	55;68;97	1.87	2.36	46.2	48.3
	2	91	2.58		47.7	
	3	27;42	2.13		47.6	
	4	61	2.16		43.0	
4 hrs	1	74;100	2.02	2.31	46.7	47.1
	2	45;95	2.31		53.4	
	3	110	2.16		52.6	
	4	94;110	2.24		55.4	
6 hrs	1	82;160;190	2.56	2.44	41.6	50.3
	2	45	2.13		45.3	
	3	96;150	1.98		44.3	
	4	87;140	2.67		43.0	
8 hrs	1	100;250	1.58	2.22	47.4	46.9
	2	180	1.89		50.7	
	3	42;110	1.60		44.1	
	4	41;200	2.22		52.3	
10 hrs	1	180;260	2.42	2.44	45.3	50.3
	2	160	2.44		46.0	
	3	190	1.13		50.1	
	4	200	2.00		44.4	
12 hrs	1	170;290;320	1.67	2.31	52.9	47.1
	2	370	1.82		48.3	
	3	250;390	1.89		47.2	
	4	240	2.24		51.2	

Table 4.33 - Hammering and integrity test results for G40/0PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	380;430;520	1.49	2.47	46.8	49.9
	2	1500*	0.00		45.7	
	3	680;880#	0.87		49.0	
	4	580;970;1100	1.00		49.9	
18 hrs	1	840;930;1300	2.31	2.84	47.9	47.0
	2	420;560;890	2.58		47.4	
	3	870;1000*	0.00		48.6	
	4	320	2.22		47.5	
24 hrs	1	1400;1900	1.22	2.47	50.9	49.9
	2	1300	1.40		52.1	
	3	2100*	0.00		50.1	
	4	720	2.00		50.5	
3 days	1	270;590;610*	0.00	2.84	48.6	47.0
	2	230;400;500	2.44		47.5	
	3	350;710*	0.00		47.9	
	4	210;300	2.31		48.8	
7 days	1	240	1.89	2.11	45.9	46.6
	2	940#	1.09		49.5	
	3	500;1100;1400*	0.00		41.3	
	4	910	2.13		47.2	
28 days	1	-*	0.00	2.40	50.2	50.0
	2	-	2.04		50.1	
	3	-*	0.00		47.7	
	4	-	1.38		49.7	

Table 4.34 - Hammering and integrity test results for G40/0PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	370;380;500*	0.00	2.22	45.2	46.9
	2	380*	0.00		45.1	
	3	510	1.60		44.1	
	4	280	1.91		47.3	
18 hrs	1	490;550*	0.00	2.36	46.3	48.3
	2	330;560*	0.00		47.0	
	3	330	1.60		43.9	
	4	180	1.91		46.0	
24 hrs	1	750;920#	0.93	2.22	43.8	46.9
	2	1300*	0.00		50.2	
	3	1200	1.60		46.2	
	4	1100	2.22		47.5	
3 days	1	-	2.18	2.36	50.0	48.3
	2	250;850;1800*	0.00		43.0	
	3	-	2.16		47.6	
	4	-*	0.00		46.3	
7 days	1	800;990;1000*	0.00	2.44	47.4	50.3
	2	1100;1600#	0.00		43.5	
	3	690	2.33		48.2	
	4	2200*	0.00		47.8	
28 days	1	570;830	2.02	2.31	46.7	47.1
	2	790*	0.00		53.4	
	3	-*	0.00		52.6	
	4	-	2.24		55.3	

Table 4.35 - Hammering and integrity test results for G40/25PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	76;78	1.98	2.00	42.2	41.5
	2	110;200;320	2.36		42.2	
	3	74	1.36		42.6	
	4	79	2.00		42.3	
4 hrs	1	63;70	2.60	2.00	42.8	41.5
	2	73;120	0.36		44.1	
	3	52	0.91		43.3	
	4	50	1.78		43.3	
6 hrs	1	60;67	1.91	2.04	46.6	43.9
	2	170;200	2.40		46.4	
	3	96	1.60		48.0	
	4	120	1.96		47.0	
8 hrs	1	81;81	2.27	2.11	47.3	45.3
	2	62;79	1.56		49.2	
	3	150	1.82		44.2	
	4	180	1.87		46.9	
10 hrs	1	81;120	2.27	2.49	41.6	40.6
	2	150;170	2.53		40.4	
	3	110	2.51		40.2	
	4	120	2.42		40.7	
12 hrs	1	170;170	2.44	2.49	39.1	40.6
	2	160;160	2.22		39.7	
	3	260	2.20		41.1	
	4	350#	0.00		44.5	

Table 4.36 - Hammering and integrity test results for G40/25PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
2 hrs	1	50;88	2.53	2.27	46.3	47.4
	2	46	1.44		48.8	
	3	30	1.91		47.5	
	4	35	1.73		47.6	
4 hrs	1	170;250	2.36	2.27	52.3	47.4
	2	69	2.60		51.7	
	3	95	1.62		44.3	
	4	210	2.24		49.4	
6 hrs	1	74;150	2.78	2.26	46.8	48.7
	2	130;140	2.44		45.8	
	3	200	2.16		46.8	
	4	200	2.47		48.1	
8 hrs	1	210	2.29	2.26	50.7	48.7
	2	170	1.62		48.7	
	3	220	2.08		49.5	
	4	230	2.24		49.1	
10 hrs	1	390;420	2.49	2.47	52.3	49.0
	2	530	1.38		47.9	
	3	530	2.07		50.8	
	4	340	2.71		49.9	
12 hrs	1	410;550	2.44	2.47	49.1	49.0
	2	550	2.13		51.6	
	3	700	2.18		48.8	
	4	240	1.84		50.3	

Table 4.37 - Hammering and integrity test results for G40/25PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	110;300;520	1.98	2.49	41.7	40.6
	2	180;180;400	1.69		39.8	
	3	73	2.22		41.3	
	4	250	1.69		39.9	
18 hrs	1	220;250;320*	0.00	2.11	44.1	45.3
	2	480	1.80		42.9	
	3	310#	0.53		43.1	
	4	510	1.87		43.3	
24 hrs	1	410	1.98	2.00	44.0	41.5
	2	230;360*	0.00		41.9	
	3	550;570#	0.91		40.4	
	4	560;620	2.44		38.1	
3 days	1	1300	1.47	2.04	42.1	43.9
	2	690;750;920	1.58		43.5	
	3	750;840;1100#	0.36		40.8	
	4	860	1.56		43.7	
7 days	1	830;1000;1300*	0.00	2.11	43.6	45.3
	2	810;1200	0.58		42.8	
	3	1200	0.67		44.2	
	4	1800*	0.00		43.6	
28 days	1	2100*	0.00	2.04	39.5	43.9
	2	1500	0.67		39.0	
	3	1400*	0.00		40.5	
	4	750	1.80		39.6	

Table 4.38 - Hammering and integrity test results for G40/25PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Direct tensile strength of hammered prism at 28-day (MPa)	Direct tensile strength of control prism at 28-day (MPa)	Equivalent cube strength of hammered prism at 28-day (MPa)	Equivalent cube strength of control prism at 28-day (MPa)
12 hrs	1	590;600	1.84	2.47	50.0	49.0
	2	560	2.22		46.1	
	3	480#	0.98		47.7	
	4	580	1.87		47.9	
18 hrs	1	810;960;1000#	0.71	2.20	50.8	45.2
	2	850;970;1100	1.09		47.5	
	3	1000	2.36		47.4	
	4	1000	2.36		51.1	
24 hrs	1	1100;1100#	0.38	2.26	48.4	48.7
	2	860#	0.67		42.6	
	3	990	2.11		45.7	
	4	1000#	1.29		43.5	
3 days	1	1100;1800*	0.00	2.27	46.1	47.4
	2	1700#	0.60		49.3	
	3	1400	1.13		47.6	
	4	-*	0.00		47.2	
7 days	1	1600;1700#	0.49	2.20	47.3	45.2
	2	1600	1.22		45.9	
	3	1500*	0.00		47.3	
	4	1400#	0.09		48.8	
28 days	1	1400;2000;2000*	0.00	2.20	48.8	45.2
	2	2000	1.96		48.2	
	3	1400	2.27		48.1	
	4	770;1600*	0.00		47.5	

5. VIBRATION RESISTANCE OF THE CONCRETE MIXES TESTED

5.1 Effects of Shock Vibration on Concrete

The effects of the shock vibrations applied to the concrete prisms during the hammering test are evaluated by comparing the tensile and compressive strengths of the hammered prisms to those of the control prisms which had not been subjected to any hammering. For such comparison purpose, the following tensile and compressive strength ratios are defined:

tensile strength ratio:

$$R_1 = \frac{P_t'}{P_t} \qquad \ldots(5.1)$$

compressive strength ratio:

$$R_2 = \frac{P_c'}{P_c} \qquad \ldots(5.2)$$

in which P_t' and P_c' are the tensile and compressive strengths of the hammered prism at 28-day, and P_t and P_c are the tensile and compressive strengths of the corresponding control prism at 28-day. A strength ratio smaller/greater than 1.0 indicates that the shock vibration had reduced/increased the strength of the concrete. On the other hand, a strength ratio close to 1.0 indicates that the shock vibration had no significant effect on the strength of the concrete.

The tensile and compressive strength ratios of the hammered prisms are tabulated in Tables 5.1 - 5.24 alongside the peak particle velocities of the shock vibrations applied to the prisms during the hammering tests. From the values of R_1 and R_2 presented, it can be clearly seen whether the vibrations applied had caused cracking and/or significant change in the strength of the concrete.

The results in Tables 5.1 - 5.24 reveal that whilst the high intensity shock vibrations induced onto the concrete prisms during the hammering tests had caused cracking or substantial reduction of the tensile strength of the concrete, they had in general little effect on the compressive strength of the concrete. Among the 576 concrete prisms which had been subjected to hammering, only two (Prism No.3 of G20/25PFA/20C tested at 10 hrs. and Prism No.1 of G20/25PFA/20C tested at 3 days) had their compressive strengths lower than those of their respective control prisms by more than 20%. Apart from these two prisms, all the other hammered prisms, representing more than 99% of the total number, had their compressive strengths remaining at above 80% of those of the respective control prisms. In fact, some of the hammered prisms even have higher compressive strength than the respective control prisms, i.e. values of R_2 higher than 1.0. It is not believed that the shock vibrations could increase the compressive strength of the concrete, especially after the concrete had hardened. The somewhat higher compressive strength of the hammered prisms than the control prisms was most likely just the result of experimental errors and random variation of concrete strength. For the same reason, it is suspected that the low R_2 values of the two prisms with R_2 smaller than 0.8 were partly due to experimental errors and random variation of concrete strength, rather than entirely the effect of the shock vibrations applied. Paying particular attention to those prisms which had been broken into pieces by the shock vibrations applied, it can be seen that even when vibrations strong enough to break the concrete into pieces had been applied, the compressive strength of the broken pieces of the prisms did not seem to have been significantly affected.

It is obvious from the test results that the major effect of the shock vibrations applied to the concrete prisms was on the tensile strength of the concrete. Depending on the intensity of the vibration applied, the shock vibrations had caused the following different degrees of damage to the prisms:

(1) the concrete prism was completely cracked and thus broken into two pieces, i.e. the concrete prism had cracks formed cutting through a whole section of it;
(2) the concrete prism was partially cracked in such a way that hairline cracks observable on the surface had been formed but the concrete prism had remained in one piece, i.e. the cracks formed had not cut through any whole section of the prism;
(3) the concrete prism had remained un-cracked (to be more specific, the concrete prism did not have any observable cracks formed on its surfaces) but its tensile strength had been significant reduced, as reflected by a relatively low R_1 value;
(4) the concrete prism had not been affected in any way, i.e. there was no observable crack formed and the concrete prism behaved just like the control prism when its tensile and compressive strengths were measured.

In Tables 5.1 - 5.24, prisms which had been completely or partially cracked are marked with "*" or "#" respectively against the shock vibration which caused the cracking.

From the results on cracking, it is seen that the effect of the shock vibrations applied was quite erratic. In many cases, two prisms cast from the same concrete mix and cured side by side under exactly the same conditions were found to be affected to very different extent when subjected to shock vibrations of similar intensity (e.g. Prisms No.2 and No.3 of G20/0PFA/20C tested at 6 hrs.). Quite often, one prism was broken into pieces when subjected to shock vibration of a certain intensity while another prism, which should be identical, withstood a much higher intensity vibration without cracking or any deterioration in strength (e.g. Prisms No.3 and No.4 of G20/0PFA/20C tested at 24 hrs.).

In order to allow for any possible self-healing effects, the hammered prisms were continued to be cured with their longitudinal axis oriented vertically so that any transverse cracks formed were kept closed under the self weight of the prisms. Prisms, which were completely cracked and broken into pieces, were reassembled by putting the broken pieces back together before curing. It is, however, found from the results presented in Tables 5.1 - 5.24 that all the prisms which had been completely cracked (those with a "*" marked) had negligible tensile strength when tested at 28-day, despite continuous curing with the cracks closed. This indicates that once the concrete has been completely cracked, there is little hope that any significant tensile strength can be recovered, i.e. there is practically no self-healing that can be relied on to reinstate the cracks. Regarding the prisms which had only been partially cracked (those with a "#" marked), their tensile strengths at 28-day ranged from 0.00 to 0.67 of those of the respectively control prisms. It is hard to say whether significant self-healing had occurred because the tensile strengths of the partially cracked prisms at 28-day might just be constituted largely of the residual strength of the cracked concrete. In any case, even with the cracks kept closed and the concrete continued to be cured so that any possible self-healing should have taken place, the tensile strengths of the partially cracked concrete prisms were found to be all lower than 70% of those of the respective control prisms. Thus, it may be said that once the concrete has observable cracks formed, it is unlikely its tensile strength can be recovered to more than 70% of that of the original concrete.

In each set of four prisms cast from the same concrete and subjected to hammering test at the same age, some prisms were each subjected to only one hammer blow while the other prisms were each subjected to two or three hammer blows. However, since no direct comparison can be made between prisms subjected to only one hammer blow and prisms subjected to two or three hammer blows (the intensities of the hammer blows applied to them were quite different), it is difficult to single out the cumulative effect of multiple hammer blows. Added with the uncertainties due to the erratic nature of the effects of shock vibrations on concrete, no definite conclusion can be drawn on the cumulative effect from the present set of data. To be on the conservative side, the prisms which had been subjected to more than one hammer blow are treated as if they had been each subjected to just one hammer blow - the hammer blow of the highest intensity among the hammer blows that the prism had been subjected to. As without the cumulative effect, the values of R_1 and R_2 should have been higher, treating the prisms as if they were each subjected to just one hammer blow would tend to underestimate the values of R_1 and R_2 for the effect of one single hammer blow.

Since it is not possible to continuously adjust the shock vibration intensity until the concrete fails, the vibration resistance of concrete cannot be directly measured. Nevertheless, for this particular study in which shock vibrations of different intensity were applied in such a way that some of the test prisms were cracked while the others remained un-cracked, the vibration resistances of the concretes tested may be indirectly determined from the following two peak particle velocity (ppv) results: (1) the lowest ppv applied that had caused failure of the concrete; and (2) the highest ppv applied that had not caused any failure of the concrete. Since shock vibrations have little effect on the compressive strength, failure in this case means either cracking or significant reduction (say, more than 30% reduction) in tensile strength. For easy reference, values of R_1 lower than 0.70 which imply more than 30% reduction in tensile strength are underlined in Tables 5.1 - 5.24. A reduction in tensile strength of less than 30% is not regarded as significant in the present context because of the following reasons:

(1) the tensile strength of concrete is by itself a variable property and when its intrinsic variation is added with the experimental errors involved in its measurement, the measured tensile strength can vary by as much as 20% or probably more in some extreme cases; and

(2) appropriate safety margins should have been included in any structural design to allow for possible variations (either expected or unexpected) in the strength of the materials used and a known reduction in the tensile strength of concrete of less than 30% should be within the normal safety margin provided.

The above two indirect measurements of the vibration resistances of the concrete mixes tested are abbreviated as ppv1 and ppv2, which are defined below:

ppv1 = lowest ppv applied that had caused failure …(5.3)

ppv2 = highest ppv applied that had not caused failure …(5.4)

These two ppv values are tabulated in the last two columns of Tables 5.1 - 5.24. Since theoretically ppv1 should be higher than ppv2, in cases where none of the hammered prisms in the same set had failed, ppv1 is taken to have a value of ">ppv2", while in cases where all of the hammered prisms in the same set had failed, ppv2 is taken to be equal to "<ppv1". However, from those sets of prisms in which at least one but not all of the hammered prisms in the same set had failed, it can be seen that due to the erratic nature of the vibration resistance of concrete, ppv1 is not always greater than ppv2.

Table 5.1 - Effects of shock vibrations on G20/0PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	150	1.07	1.10	>330	330
	2	230	0.72	1.18		
	3	40	1.03	1.11		
	4	330	0.98	1.06		
4 hrs	1	240	0.85	0.93	>380	380
	2	170	1.11	0.81		
	3	380	0.88	1.05		
	4	95	0.95	1.06		
6 hrs	1	110	1.06	1.07	290	300
	2	290	<u>0.65</u>	1.04		
	3	300	1.02	1.04		
	4	140	1.09	1.06		
8 hrs	1	420	1.09	1.09	>420	420
	2	410	1.26	1.20		
	3	140	1.36	1.17		
	4	180	1.21	1.10		
10 hrs	1	480	1.17	1.19	>480	480
	2	250	1.22	1.10		
	3	160	1.13	1.05		
	4	290	1.16	1.07		
12 hrs	1	150	1.26	1.10	>410	410
	2	230	1.21	1.10		
	3	410	1.26	1.19		
	4	200	1.25	1.15		

Table 5.2 - Effects of shock vibrations on G20/0PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	21;90	0.97	1.08	>550	550
	2	340	1.22	1.07		
	3	240	0.98	0.97		
	4	550	1.05	1.05		
4 hrs	1	86;120;160	0.76	1.07	79	160
	2	91	0.29	1.02		
	3	79	0.57	1.03		
	4	56	0.95	1.06		
6 hrs	1	74	0.98	1.06	>180	180
	2	180	0.85	1.16		
	3	110	0.95	1.09		
	4	79	1.09	1.10		
8 hrs	1	170	0.99	1.00	>180	180
	2	140	1.05	0.97		
	3	140	1.06	0.96		
	4	180	0.98	0.97		
10 hrs	1	820	0.85	1.05	>820	820
	2	460;800	1.07	1.02		
	3	750	1.13	1.06		
	4	680	0.94	1.03		
12 hrs	1	740	1.06	1.01	>860	860
	2	650	1.08	1.05		
	3	760;860	0.99	1.12		
	4	510;640	0.85	1.06		

Table 5.3 - Effects of shock vibrations on G20/0PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	460;600;680#	0.21	1.04	570	640
	2	640	0.91	1.03		
	3	610*	0.00	1.01		
	4	570	0.25	1.06		
18 hrs	1	670;910;960*	0.00	0.93	890	920
	2	590	0.81	0.93		
	3	920	0.91	0.97		
	4	890#	0.09	0.89		
24 hrs	1	120;170;420*	0.00	1.01	350	700
	2	670	1.18	0.98		
	3	350*	0.00	0.97		
	4	410;700	0.97	0.98		
3 days	1	1500	0.60	1.00	1200	1500
	2	840;1700#	0.06	0.99		
	3	1500	0.90	1.02		
	4	700;1100;1200*	0.00	1.06		
7 days	1	1200	1.22	0.96	1600	1200
	2	1100	1.02	0.92		
	3	1100	1.15	0.98		
	4	1100;1300;1600#	0.00	0.98		
28 days	1	1800;1900;3600*	0.00	0.97	2300	2300
	2	1500;1700	1.06	0.95		
	3	1800;2000;2300	1.06	1.00		
	4	2300#	0.67	0.96		

Table 5.4 - Effects of shock vibrations on G20/0PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	780;1100;1200#	0.00	1.04	1200	910
	2	-	0.83	0.98		
	3	510	0.94	1.08		
	4	580;600;910	0.94	1.07		
18 hrs	1	660;730;990*	0.00	1.05	990	1300
	2	780	1.04	1.03		
	3	810;830;1600*	0.00	1.08		
	4	1300	1.11	1.03		
24 hrs	1	710	1.04	1.12	1200	840
	2	740;990;1300*	0.00	1.12		
	3	840	0.87	1.08		
	4	700;1100;1200*	0.00	1.12		
3 days	1	2300;2500;2900*	0.00	1.05	2700	2500
	2	900;1000;2100	1.01	1.05		
	3	1700;2100;2700*	0.00	1.02		
	4	1100;2100;2500	0.79	1.01		
7 days	1	600;1800;2900*	0.00	1.03	2900	3100
	2	1200	0.96	0.97		
	3	2300	0.91	0.99		
	4	3100	0.93	1.05		
28 days	1	3400	0.86	1.01	2900	3400
	2	2100	0.86	1.02		
	3	1800;2100;2900*	0.00	1.07		
	4	3300	0.79	1.05		

Table 5.5 - Effects of shock vibrations on G20/25PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	-	1.03	1.21	>67	67
	2	28	0.71	1.19		
	3	67	1.03	1.21		
	4	35	1.10	1.21		
4 hrs	1	150	0.93	1.23	>700	700
	2	70	1.00	1.02		
	3	300	0.88	1.15		
	4	700	0.96	1.16		
6 hrs	1	130	0.68	1.07	130	270
	2	60	0.97	1.05		
	3	270	0.88	0.93		
	4	210	0.43	1.06		
8 hrs	1	200	1.03	0.91	1700	1700
	2	150	1.11	0.82		
	3	1700	0.33	0.85		
	4	1700	0.86	0.94		
10 hrs	1	560	1.01	0.91	520	2100
	2	520	0.16	0.87		
	3	1800	0.57	0.75		
	4	2100	0.94	0.83		
12 hrs	1	660	0.54	0.84	660	3400
	2	560	0.81	0.86		
	3	1600	0.65	0.93		
	4	3400	0.71	0.88		

Table 5.6 - Effects of shock vibrations on G20/25PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	23	1.25	1.13	>56	56
	2	31	1.08	1.12		
	3	25	1.16	0.99		
	4	56	1.12	1.02		
4 hrs	1	36;38	1.14	1.09	>150	150
	2	36	1.10	1.18		
	3	150	1.08	1.15		
	4	140	1.02	1.18		
6 hrs	1	210;310	1.09	1.11	>310	310
	2	180;210	1.16	1.16		
	3	170	1.12	1.13		
	4	250	1.08	1.13		
8 hrs	1	230	0.92	1.08	>1100	1100
	2	140	1.17	1.06		
	3	580	1.20	1.10		
	4	1100	1.08	1.10		
10 hrs	1	290;300;380	0.49	1.05	310	410
	2	360;410	0.87	1.06		
	3	370	0.74	1.03		
	4	310	0.27	1.01		
12 hrs	1	300;310	0.87	0.99	400	310
	2	310;370;400	0.59	1.05		
	3	250	0.92	1.07		
	4	360;460	0.12	1.10		

Table 5.7 - Effects of shock vibrations on G20/25PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	340#	0.00	0.84	340	<340
	2	980	0.64	0.84		
	3	-*	0.00	0.84		
	4	-*	0.00	0.87		
18 hrs	1	510;1200#	0.59	0.95	1200	2300
	2	520;920	0.74	0.95		
	3	2000;2300	0.71	0.97		
	4	1600	0.83	1.02		
24 hrs	1	550;750#	0.58	1.07	750	620
	2	510;620	1.03	1.05		
	3	480;1200	0.58	1.00		
	4	760;1200;1500#	0.39	1.05		
3 days	1	1400	0.92	0.77	2300	1400
	2	1100	0.83	0.90		
	3	2800*	0.00	0.93		
	4	2300#	0.48	0.87		
7 days	1	2800*	0.00	1.07	1000	2600
	2	2200*	0.00	1.06		
	3	1000	0.55	1.06		
	4	2600	1.00	1.00		
28 days	1	3100	0.94	1.09	2300	3100
	2	1400	0.94	1.11		
	3	2300#	0.65	1.06		
	4	2200	0.94	1.08		

Table 5.8 - Effects of shock vibrations on G20/25PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	450;570;700	0.74	1.08	>700	700
	2	440	0.87	1.00		
	3	390	0.92	1.06		
	4	450	0.82	1.05		
18 hrs	1	700	0.94	1.03	880	930
	2	590;650;820	0.75	0.97		
	3	550;880	0.68	1.01		
	4	930	1.08	0.99		
24 hrs	1	640;710;1500	0.84	0.95	>1700	1700
	2	860	0.72	0.98		
	3	740	0.82	0.97		
	4	1100;1400;1700	0.96	1.00		
3 days	1	2600#	0.45	1.02	1200	<1200
	2	1200	0.39	0.98		
	3	2200*	0.00	0.92		
	4	2500*	0.00	1.02		
7 days	1	3500#	0.20	1.09	3500	2300
	2	2300	0.77	1.08		
	3	1800	1.15	1.00		
	4	1700	0.98	1.09		
28 days	1	3100*	0.00	1.07	2900	<2900
	2	2900	0.61	1.06		
	3	3200#	0.34	1.05		
	4	2900*	0.00	1.07		

Table 5.9 - Effects of shock vibrations on G30/0PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	24;100	1.08	1.03	>100	100
	2	100	1.02	1.06		
	3	92	1.01	1.06		
	4	-	0.95	1.02		
4 hrs	1	74;88	0.97	1.00	>500	500
	2	320	0.85	0.89		
	3	410	1.09	0.86		
	4	500	0.94	0.94		
6 hrs	1	100;180	0.89	0.82	>670	670
	2	160;240	1.05	0.91		
	3	600	1.00	0.92		
	4	670	0.95	0.85		
8 hrs	1	180;280	1.09	0.87	500	490
	2	340;390	0.93	0.98		
	3	500	<u>0.59</u>	0.97		
	4	490	0.98	0.95		
10 hrs	1	220;330;400	1.00	0.96	410	540
	2	410	<u>0.65</u>	0.99		
	3	370;540	1.19	1.00		
	4	350	1.14	0.98		
12 hrs	1	490;560	0.75	1.00	>820	820
	2	500;820	1.14	0.93		
	3	300	1.05	0.96		
	4	530	0.87	0.99		

Table 5.10 - Effects of shock vibrations on G30/0PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	170;180	0.99	1.04	>180	180
	2	150	1.04	1.02		
	3	46	0.97	1.06		
	4	29	0.85	0.98		
4 hrs	1	120;260;280	0.98	1.05	>300	300
	2	170	0.86	0.91		
	3	230	0.90	0.96		
	4	300	0.94	0.97		
6 hrs	1	340;350;440	0.94	0.92	>440	440
	2	210;230	1.03	1.05		
	3	210	0.98	0.96		
	4	310	1.04	1.00		
8 hrs	1	340;410;430	1.00	1.01	>600	600
	2	600	0.87	0.99		
	3	350	0.90	1.00		
	4	390	1.02	0.94		
10 hrs	1	570;700;770	0.63	0.88	770	830
	2	560	0.95	0.89		
	3	610;830	0.85	0.92		
	4	710	1.19	0.93		
12 hrs	1	1100;1200	1.11	0.88	>1200	1200
	2	430;780	1.01	0.93		
	3	780	0.84	0.91		
	4	980	1.05	0.89		

Table 5.11 - Effects of shock vibrations on G30/0PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	310;460;800#	0.37	1.00	800	760
	2	760	1.04	0.99		
	3	510	0.97	0.97		
	4	730	0.93	0.98		
18 hrs	1	810;1000*	0.00	0.94	1000	700
	2	430	0.89	0.97		
	3	700	0.95	0.97		
	4	730;1300#	0.18	0.94		
24 hrs	1	980;1400;1600#	0.27	0.97	1100	1600
	2	1600	0.70	0.94		
	3	1000	0.89	0.96		
	4	990;1100#	0.19	0.93		
3 days	1	890;1100;1100*	0.00	0.91	1000	740
	2	1000#	0.18	0.95		
	3	620	0.94	1.01		
	4	740	1.28	0.97		
7 days	1	1300;1400;1600*	0.00	0.94	1600	1100
	2	1700#	0.23	0.92		
	3	1100	0.94	0.96		
	4	950	1.15	1.00		
28 days	1	1500;1800;1900*	0.00	0.97	1400	1200
	2	1400*	0.00	0.97		
	3	1200	0.89	0.96		
	4	1100;1500	0.23	0.95		

Table 5.12 - Effects of shock vibrations on G30/0PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	590;800;1100#	0.18	0.88	1100	1600
	2	1600	1.06	0.96		
	3	810	1.01	0.93		
	4	1900*	0.00	0.85		
18 hrs	1	990	0.80	0.93	1200	990
	2	-*	0.00	0.91		
	3	900;1200#	0.18	0.87		
	4	690	0.86	0.95		
24 hrs	1	1500;1600*	0.00	0.99	1500	1500
	2	1500	0.89	0.97		
	3	940	0.74	0.97		
	4	1400;1500#	0.10	0.97		
3 days	1	1300;1400*	0.00	0.98	1400	1100
	2	1000;1500*	0.00	0.96		
	3	980	1.03	0.99		
	4	1100	0.95	0.92		
7 days	1	1500;1700#	0.21	0.93	1600	720
	2	1500;1600*	0.00	0.87		
	3	720	0.84	0.97		
	4	1700	0.59	0.93		
28 days	1	1100;1500;1800*	0.00	0.94	1400	<1400
	2	1400#	0.19	0.96		
	3	1500#	0.33	0.97		
	4	1900*	0.00	0.97		

Table 5.13 - Effects of shock vibrations on G30/25PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	22;90	1.07	0.98	>90	90
	2	22	0.89	0.94		
	3	30	1.01	1.01		
	4	32	0.76	0.91		
4 hrs	1	230;440	0.95	0.96	>620	620
	2	490	0.99	0.98		
	3	280	1.05	1.02		
	4	620	0.95	0.98		
6 hrs	1	200	0.96	1.00	>410	410
	2	84;190;260	1.26	1.03		
	3	290	1.04	0.97		
	4	410	1.11	1.00		
8 hrs	1	210;250;280	1.11	1.04	>380	380
	2	380	0.94	1.03		
	3	320	1.04	0.97		
	4	400;480	?	1.04		
10 hrs	1	340;400;540	0.95	0.97	>790	790
	2	620	0.89	0.95		
	3	460	1.06	1.03		
	4	650;790	0.88	0.99		
12 hrs	1	260;480;560	0.95	0.99	>860	860
	2	370	1.05	0.99		
	3	470	0.87	0.93		
	4	860	0.97	0.98		

Table 5.14 - Effects of shock vibrations on G30/25PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	20;47	1.03	1.05	>69	69
	2	36	1.13	1.11		
	3	29	1.18	1.03		
	4	69	1.05	1.07		
4 hrs	1	180;240;350	1.13	1.04	>460	460
	2	460	1.08	1.01		
	3	240	1.01	0.98		
	4	370	1.08	1.02		
6 hrs	1	410;490;510	0.83	0.97	660	670
	2	670	0.82	0.99		
	3	630	1.05	1.01		
	4	660	0.65	0.97		
8 hrs	1	520;960;1100	1.23	1.04	860	1100
	2	860	1.06	0.96		
	3	790	0.96	0.98		
	4	860	0.42	1.01		
10 hrs	1	440;640;660	1.11	1.03	>760	760
	2	760	1.00	1.01		
	3	450	0.94	0.99		
	4	730	1.04	0.99		
12 hrs	1	550;560;830	1.03	0.88	>830	830
	2	580	1.06	1.00		
	3	500	1.04	0.96		
	4	670	1.15	0.91		

Table 5.15 - Effects of shock vibrations on G30/25PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	550;610;1100#	0.12	0.94	1100	1000
	2	680	0.91	0.95		
	3	580	0.95	1.02		
	4	1000	0.83	0.96		
18 hrs	1	750;850#	0.25	1.08	850	1000
	2	760	1.09	1.05		
	3	1000	0.63	0.98		
	4	1000	1.12	1.01		
24 hrs	1	870;1300#	0.25	0.99	1300	1300
	2	1300	0.87	0.94		
	3	890	0.90	0.96		
	4	1500*	0.00	0.94		
3 days	1	1100;1300;2000*	0.00	0.93	1700	1400
	2	1400	1.09	0.88		
	3	940	0.89	0.99		
	4	1700*	0.00	0.98		
7 days	1	1300;1400*	0.00	0.97	1200	810
	2	1200#	0.21	0.94		
	3	810	0.91	0.99		
	4	1400;1500;1700*	0.00	0.99		
28 days	1	1600;1900;1900	0.78	0.98	1500	1900
	2	1400;1600;1700#	0.00	0.97		
	3	1400	0.89	0.96		
	4	1500#	0.35	0.96		

Table 5.16 - Effects of shock vibrations on G30/25PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	680;700	0.78	0.92	640	700
	2	430;640*	0.00	0.88		
	3	450	0.94	0.96		
	4	410	0.81	0.91		
18 hrs	1	910;1100;1400#	0.21	0.99	980	780
	2	980	0.58	1.05		
	3	780	0.96	1.02		
	4	1300#	0.19	1.02		
24 hrs	1	1300;1600;1700#	0.39	1.01	1200	1300
	2	1200	0.61	0.98		
	3	1300	1.01	1.03		
	4	1700#	0.20	0.98		
3 days	1	1300;1400#	0.19	0.86	1400	900
	2	1700*	0.00	0.82		
	3	900	0.81	1.01		
	4	1500;1600#	0.19	1.11		
7 days	1	1300;1400;2000#	0.31	0.99	1300	1300
	2	1300	0.22	0.92		
	3	1300	0.91	0.96		
	4	1500;1600*	0.00	0.98		
28 days	1	1700;2000;2200*	0.00	1.02	1400	980
	2	1600	0.43	1.01		
	3	1400*	0.00	1.00		
	4	980	0.95	1.00		

Table 5.17 - Effects of shock vibrations on G40/0PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	49;98;110	0.83	1.00	>130	130
	2	38;45;53	0.89	1.04		
	3	28;88	0.77	1.06		
	4	100;130	0.83	1.09		
4 hrs	1	74;80	1.04	1.05	190	270
	2	50;110	0.89	1.03		
	3	74;98;270	0.98	1.10		
	4	150;180;190	<u>0.61</u>	1.04		
6 hrs	1	290;350;420	1.22	1.10	>420	420
	2	-	0.84	1.06		
	3	100;110;130	0.90	0.95		
	4	110	1.10	1.01		
8 hrs	1	130;150;330	0.99	1.04	>450	450
	2	150;210;220	1.04	1.00		
	3	160;180;180	0.98	1.07		
	4	430;450	0.99	1.01		
10 hrs	1	230;290;300	1.15	1.06	>390	390
	2	210;250	1.19	1.17		
	3	170;380	1.20	1.25		
	4	240;390	1.13	1.25		
12 hrs	1	210;210;240	1.02	1.05	>420	420
	2	230;390	0.95	1.04		
	3	200;200;210	1.11	1.02		
	4	320;420	0.82	1.03		

Table 5.18 - Effects of shock vibrations on G40/0PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	55;68;97	0.79	0.96	>97	97
	2	91	1.09	0.99		
	3	27;42	0.90	0.99		
	4	61	0.92	0.89		
4 hrs	1	74;100	0.87	0.99	>110	110
	2	45;95	1.00	1.13		
	3	110	0.94	1.12		
	4	94;110	0.97	1.18		
6 hrs	1	82;160;190	1.05	0.83	>190	190
	2	45	0.87	0.90		
	3	96;150	0.81	0.88		
	4	87;140	1.09	0.85		
8 hrs	1	100;250	0.71	1.01	>250	250
	2	180	0.85	1.08		
	3	42;110	0.72	0.94		
	4	41;200	1.00	1.12		
10 hrs	1	180;260	0.99	0.90	190	260
	2	160	1.00	0.91		
	3	190	<u>0.46</u>	1.00		
	4	200	0.82	0.88		
12 hrs	1	170;290;320	0.72	1.12	>390	390
	2	370	0.79	1.03		
	3	250;390	0.82	1.00		
	4	240	0.97	1.09		

Table 5.19 - Effects of shock vibrations on G40/0PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
	1	380;430;520	0.60	0.94		
12 hrs	2	1500*	0.00	0.92	520	<520
	3	680;880#	0.35	0.98		
	4	580;970;1100	0.40	1.00		
	1	840;930;1300	0.81	1.02		
18 hrs	2	420;560;890	0.91	1.01	1000	1300
	3	870;1000*	0.00	1.03		
	4	320	0.78	1.01		
	1	1400;1900	0.49	1.02		
24 hrs	2	1300	0.57	1.04	1300	720
	3	2100*	0.00	1.00		
	4	720	0.81	1.01		
	1	270;590;610*	0.00	1.03		
3 days	2	230;400;500	0.86	1.01	610	500
	3	350;710*	0.00	1.02		
	4	210;300	0.81	1.04		
	1	240	0.90	0.98		
7 days	2	940#	0.52	1.06	940	910
	3	500;1100;1400*	0.00	0.89		
	4	910	1.01	1.01		
	1	-*	0.00	1.00		
28 days	2	-	0.85	1.00	?	?
	3	-*	0.00	0.95		
	4	-	0.58	0.99		

Table 5.20 - Effects of shock vibrations on G40/0PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	370;380;500*	0.00	0.96	380	510
	2	380*	0.00	0.96		
	3	510	0.72	0.94		
	4	280	0.86	1.01		
18 hrs	1	490;550*	0.00	0.96	330	180
	2	330;560*	0.00	0.97		
	3	330	0.68	0.91		
	4	180	0.81	0.95		
24 hrs	1	750;920#	0.42	0.93	920	1200
	2	1300*	0.00	1.07		
	3	1200	0.72	0.99		
	4	1100	1.00	1.01		
3 days	1	-	0.92	1.04	1800	<1800
	2	250;850;1800*	0.00	0.89		
	3	-	0.92	0.99		
	4	-*	0.00	0.96		
7 days	1	800;990;1000*	0.00	0.94	1000	690
	2	1100;1600#	0.00	0.86		
	3	690	0.95	0.96		
	4	2200*	0.00	0.95		
28 days	1	570;830	0.87	0.99	790	830
	2	790*	0.00	1.13		
	3	-*	0.00	1.12		
	4	-	0.97	1.17		

Table 5.21 - Effects of shock vibrations on G40/25PFA/20C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	76;78	0.99	1.02	74	320
	2	110;200;320	1.18	1.02		
	3	74	<u>0.68</u>	1.03		
	4	79	1.00	1.02		
4 hrs	1	63;70	1.30	1.03	52	70
	2	73;120	<u>0.18</u>	1.06		
	3	52	<u>0.46</u>	1.04		
	4	50	0.89	1.04		
6 hrs	1	60;67	0.94	1.06	>200	200
	2	170;200	1.18	1.06		
	3	96	0.78	1.09		
	4	120	0.96	1.07		
8 hrs	1	81;81	1.08	1.04	>180	180
	2	62;79	0.74	1.09		
	3	150	0.86	0.98		
	4	180	0.89	1.04		
10 hrs	1	81;120	0.91	1.02	>170	170
	2	150;170	1.02	1.00		
	3	110	1.01	0.99		
	4	120	0.97	1.00		
12 hrs	1	170;170	0.98	0.96	350	260
	2	160;160	0.89	0.98		
	3	260	0.88	1.01		
	4	350#	<u>0.00</u>	1.10		

Table 5.22 - Effects of shock vibrations on G40/25PFA/30C - Series 1 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
2 hrs	1	50;88	1.11	0.98	46	88
	2	46	0.63	1.03		
	3	30	0.84	1.00		
	4	35	0.76	1.00		
4 hrs	1	170;250	1.04	1.10	>250	250
	2	69	1.15	1.09		
	3	95	0.71	0.93		
	4	210	0.99	1.04		
6 hrs	1	74;150	1.23	0.96	>200	200
	2	130;140	1.08	0.94		
	3	200	0.96	0.96		
	4	200	1.09	0.99		
8 hrs	1	210	1.01	1.04	>230	230
	2	170	0.72	1.00		
	3	220	0.92	1.02		
	4	230	0.99	1.01		
10 hrs	1	390;420	1.01	1.07	530	530
	2	530	0.56	0.98		
	3	530	0.84	1.04		
	4	340	1.10	1.02		
12 hrs	1	410;550	0.99	1.00	>700	700
	2	550	0.86	1.05		
	3	700	0.88	1.00		
	4	240	0.74	1.03		

Table 5.23 - Effects of shock vibrations on G40/25PFA/20C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	110;300;520	0.80	1.03	250	520
	2	180;180;400	0.68	0.98		
	3	73	0.89	1.02		
	4	250	0.68	0.98		
18 hrs	1	220;250;320*	0.00	0.97	310	510
	2	480	0.85	0.95		
	3	310#	0.25	0.95		
	4	510	0.89	0.96		
24 hrs	1	410	0.99	1.06	360	620
	2	230;360*	0.00	1.01		
	3	550;570#	0.46	0.97		
	4	560;620	1.22	0.92		
3 days	1	1300	0.72	0.96	1100	1300
	2	690;750;920	0.77	0.99		
	3	750;840;1100#	0.18	0.93		
	4	860	0.76	1.00		
7 days	1	830;1000;1300*	0.00	0.96	1200	<1200
	2	810;1200	0.27	0.94		
	3	1200	0.32	0.98		
	4	1800*	0.00	0.96		
28 days	1	2100*	0.00	0.90	1400	750
	2	1500	0.33	0.89		
	3	1400*	0.00	0.92		
	4	750	0.88	0.90		

Table 5.24 - Effects of shock vibrations on G40/25PFA/30C - Series 2 tests

Age at hammer test	Prism no.	Peak particle velocity of shock wave applied (mm/s)	Tensile strength ratio, R_1	Compressive strength ratio, R_2	ppv1 (mm/s)	ppv2 (mm/s)
12 hrs	1	590;600	0.74	1.02	480	600
	2	560	0.90	0.94		
	3	480#	0.40	0.97		
	4	580	0.76	0.98		
18 hrs	1	810;960;1000#	0.32	1.12	1000	1000
	2	850;970;1100	0.50	1.05		
	3	1000	1.07	1.05		
	4	1000	1.07	1.13		
24 hrs	1	1100;1100#	0.17	0.99	860	990
	2	860#	0.30	0.87		
	3	990	0.93	0.94		
	4	1000#	0.57	0.89		
3 days	1	1100;1800*	0.00	0.97	1400	<1400
	2	1700#	0.26	1.04		
	3	1400	0.50	1.00		
	4	-*	0.00	1.00		
7 days	1	1600;1700#	0.22	1.05	1400	<1400
	2	1600	0.55	1.02		
	3	1500*	0.00	1.05		
	4	1400#	0.04	1.08		
28 days	1	1400;2000;2000*	0.00	1.08	1600	2000
	2	2000	0.89	1.07		
	3	1400	1.03	1.06		
	4	770;1600*	0.00	1.05		

5.2 Lower Bound Estimates of Vibration Resistance

As can be seen from Tables 5.1 - 5.24, there does not appear to be any simple relation between the intensity of the shock vibration applied and the corresponding values of R_1 and R_2. Moreover, the value of ppv2 is sometimes higher than that of ppv1, albeit theoretically ppv2 should be lower than ppv1. Hence, the vibration resistances of the concrete mixes tested cannot be precisely determined.

An attempt of applying theoretical analysis to the ppv results will be made in the next chapter, but it will soon be seen that any rigorous analysis is extremely difficult. In the meantime, it should be more prudent to find out the lower bound of the vibration resistances of the concrete mixes tested and make use of the lower bound estimates to set safe ppv limits for preventing vibration damages to concrete.

From each pair of ppv1 and ppv2 values, a lower bound estimate of the vibration resistance of the concrete may be obtained as:

$$\text{ppv3} = \text{minimum}(\text{ppv1}, \text{ppv2}) \qquad \ldots(5.5)$$

In cases where none of the hammered prisms in the same set had failed (in these cases, ppv1 is given a value equal to “>ppv2”), ppv3 is taken to be equal to “ppv2”. On the other hand, in cases where all the hammered prisms had failed (in these cases, ppv2 is given a value equal to “<ppv1”), ppv3 is taken to have a value equal to “<ppv1” because in this situation, the value of ppv1 is not really a lower bound of the vibration resistance of the concrete tested (there was the possibility that the concrete tested could have failed at a somewhat lower vibration intensity than ppv1). Therefore, when the value of ppv3 is given a “<” sign, care should be taken when interpreting its implications that this is not really a lower bound value.

The ppv3 values of the concrete mixes tested are listed in Tables 5.25 - 5.27 for ages of less than 12 hrs. and in Tables 5.28 - 5.30 for ages of 12 hrs. to 28 days. Again, the ppv3 values demonstrate that the vibration resistance of concrete is very erratic. Taking the results presented in Table 5.25 as an example, it can be seen that for similar concretes (all the results presented are those of grade 20 concretes cured at 20ºC - 30ºC), the ppv3 values at the same age of 4 hrs. can range from 79 mm/s to 700 mm/s and that even for the same concrete, e.g. G20/25PFA/20C, the ppv3 value can change from a value of 1700 mm/s at 8 hrs. to 520 mm/s at 10 hrs.

No obvious effects of using PFA up to 25% and curing temperature within the range of 20ºC - 30ºC on the vibration resistance of concrete can be identified from the ppv3 results presented in the tables. Any probable effects of PFA and curing temperature within the ranges studied have been over-shadowed by the large intrinsic variation of the vibration resistance of concrete. In view of this situation, it is suggested that when setting vibration control limits to avoid vibration damages to concrete, a single set of vibration limits that applies to concretes with or without PFA and cured within the temperature range of 20ºC - 30ºC should be used; it does not seem to be worthwhile to have more refined vibration control limits that take these two factors into account. For the purpose of establishing a single set of vibration control limits for a specific grade of concrete, the lower bounds of the ppv3 values of the four batches of concrete in the same grade are worked out and tabulated in the last row of each table in Tables 5.25 - 5.30. These lower bounds should apply to all concretes of the specified grade regardless of whether they contain PFA and their curing temperature provided their PFA content and curing temperature fall within the ranges studied.

The lower bound estimates of the vibration resistances of G20, G30 and G40 concretes are plotted against time in Figs. 5.1 and 5.2 to illustrate how they vary with the age of the concrete. It is seen that the lower bound estimates of the vibration resistance of concrete generally start at about 50 mm/s while the concrete is still fresh, increase to about 300 mm/s at 12 hrs. when the concrete is already hardened but is still relatively weak, and then further increase to more than 700 mm/s when the concrete has fully developed its strength. The increase of these lower bound values with time is, however, not monotonic because they are superimposed with large intrinsic variation of the vibration resistance of concrete.

Table 5.25 - Lower bound of vibration resistances of G20 concretes - Series 1 tests

Batch no.	ppv3 at various ages (mm/s)					
	2 hrs	4 hrs	6 hrs	8 hrs	10 hrs	12 hrs
G20/0PFA/20C	330	380	290	420	480	410
G20/0PFA/30C	550	79	180	180	820	860
G20/25PFA/20C	67	700	130	1700	520	660
G20/25PFA/30C	56	150	310	1100	310	310
lower bound for G20 concretes	56	79	130	180	310	310

Table 5.26 - Lower bound of vibration resistances of G30 concretes - Series 1 tests

Batch no.	ppv3 at various ages (mm/s)					
	2 hrs	4 hrs	6 hrs	8 hrs	10 hrs	12 hrs
G30/0PFA/20C	100	500	670	490	410	820
G30/0PFA/30C	180	300	440	600	770	1200
G30/25PFA/20C	90	620	410	380	790	860
G30/25PFA/30C	69	460	660	860	760	830
lower bound for G30 concretes	69	300	410	380	410	820

Table 5.27 - Lower bound of vibration resistances of G40 concretes - Series 1 tests

Batch no.	ppv3 at various ages (mm/s)					
	2 hrs	4 hrs	6 hrs	8 hrs	10 hrs	12 hrs
G40/0PFA/20C	130	190	420	450	390	420
G40/0PFA/30C	97	110	190	250	190	390
G40/25PFA/20C	74	52	200	180	170	260
G40/25PFA/30C	46	250	200	230	530	700
lower bound for G40 concretes	46	52	190	180	170	260

Table 5.28 - Lower bound of vibration resistances of G20 concretes - Series 2 tests

Batch no.	ppv3 at various ages (mm/s)					
	12 hrs	18 hrs	24 hrs	3 days	7 days	28 days
G20/0PFA/20C	570	890	350	1200	1200	2300
G20/0PFA/30C	910	990	840	2500	2900	2900
G20/25PFA/20C	<340	1200	620	1400	1000	2300
G20/25PFA/30C	700	880	1700	<1200	2300	<2900
lower bound for G20 concretes	<340	880	350	<1200	1000	2300

Table 5.29 - Lower bound of vibration resistances of G30 concretes - Series 2 tests

Batch no.	ppv3 at various ages (mm/s)					
	12 hrs	18 hrs	24 hrs	3 days	7 days	28 days
G30/0PFA/20C	760	700	1100	740	1100	1200
G30/0PFA/30C	1100	990	1500	1100	720	<1400
G30/25PFA/20C	1000	850	1300	1400	810	1500
G30/25PFA/30C	640	780	1200	900	1300	980
lower bound for G30 concretes	640	700	1100	740	720	980

Table 5.30 - Lower bound of vibration resistances of G40 concretes - Series 2 tests

Batch no.	ppv3 at various ages (mm/s)					
	12 hrs	18 hrs	24 hrs	3 days	7 days	28 days
G40/0PFA/20C	<520	1000	720	500	910	?
G40/0PFA/30C	380	180	920	<1800	690	790
G40/25PFA/20C	250	310	360	1100	<1200	750
G40/25PFA/30C	480	1000	860	<1400	<1400	1600
lower bound for G40 concretes	250	180	360	500	690	750

5.3 Recommended Vibration Control Limits

Looking at the lower bound estimates plotted in Figs. 5.1 and 5.2, it is found very difficult to set different vibration control limits for different grades of concrete, although it is believed that the grade of the concrete should have a bearing on the vibration resistance. To overcome this difficulty, a lower bound for all the concrete mixes tested, i.e. concretes of grade 20 to 40, without PFA or with PFA up to 25%, and cured at a temperature within 20ºC - 30ºC, is worked out and used to establish a single set of vibration control limits for all these concretes. The lower bound value for all the concrete mixes tested is just the smallest value of ppv3 at a particular age among the twelve batches of concrete tested. These lower bound values of vibration resistance for all the concrete mixes tested are listed together with the lower bound estimates for the different grades of concrete in Tables 5.31 and 5.32. It is noteworthy that whilst the “<” sign appears in some of the lower bound estimates for an individual grade of concrete meaning that the values with the sign attached are not really lower bounds, the “<” sign no longer appears in the lower bound values for all the concrete mixes tested indicating that all the lower bound values arrived at are true lower bound values.

For easier visualization, the lower bound values of vibration resistance at various ages for all the concrete mixes tested are plotted against time in Figs. 5.3 and 5.4. From these two graphs, it is seen that the lower bound value of vibration resistance for all the concrete mixes tested changes more smoothly with the age of the concrete and is less erratic compared to the lower bound estimates for the individual grades of concrete because of the larger number of test samples from which the lower bound for all concrete mixes tested is derived.

Before a set of safe ppv limits can be established, there is still the problem of fixing an appropriate value for the factor of safety to be used. Although the lower bound values of vibration resistance listed in Tables 5.31 and 5.32 which are derived from a fairly large number of vibration tests should be reasonably reliable, it is still considered advisable to adopt a cautious approach of using a relatively large value for the factor of safety. Bearing in mind that the effects of shock vibration on concrete are quite erratic and not yet fully understood, a value of 5 is recommended.

Dividing the lower bound values of vibration resistance listed in Tables 5.31 and 5.32 by a factor of safety of 5 and adjusting the resulting values slightly so that they map into a smooth and monotonic function of the age of concrete, the following set of safe ppv limits is obtained:

Age of concrete	Safe ppv limit (mm/s)
≤ 4 hrs.	10
6 - 8 hrs.	20
10 - 12 hrs.	30
18 hrs.	40
24 hrs.	70
3 days	100
7 days	125
≥ 28 days	150

The above recommended vibration control limits are also given in the last rows of Tables 5.31 and 5.32 for easy comparison with the lower bound values of vibration resistance. Since these limits are derived from the lower bound values for all the concrete mixes tested, they should be applicable to all concretes whose grade, PFA content and curing temperature fall within the ranges studied.

These vibration control limits are compared to those adopted by others in Figs. 5.5 to 5.8. It is seen that the recommended vibration limits for green concrete (concrete at age less than 24 hrs.) are generally higher than those adopted by Gamble and Simpson (1985) and Bryson and Cooley (1985) but lower than those of Olofesson (1988) for concrete at age less than 4 hrs. For concrete more than 1 day old, the above limits are higher than all those adopted by Gamble and Simpson (1985), Bryson and Cooley (1985) and Olofesson (1988). However, the recommended limits are lower than those proposed by Hulshizer and Desai (1984) at virtually all ages.

Table 5.31 - Lower bound of vibration resistances of all concretes tested - Series 1 tests

Batch no.	vibration resistance at various ages (mm/s)					
	2 hrs	4 hrs	6 hrs	8 hrs	10 hrs	12 hrs
lower bound for G20 concretes	56	79	130	180	310	310
lower bound for G30 concretes	69	300	410	380	410	820
lower bound for G40 concretes	46	52	190	180	170	260
lower bound for all concretes tested	46	52	130	180	170	260
recommended vibration control limits	10	10	20	20	30	30

Table 5.32 - Lower bound of vibration resistances of all concretes tested - Series 2 tests

Batch no.	vibration resistance at various ages (mm/s)					
	12 hrs	18 hrs	24 hrs	3 days	7 days	28 days
lower bound for G20 concretes	<340	880	350	<1200	1000	2300
lower bound for G30 concretes	640	700	1100	740	720	980
lower bound for G40 concretes	250	180	360	500	690	750
lower bound for all concretes tested	250	180	350	500	690	750
recommended vibration control limits	30	40	70	100	125	150

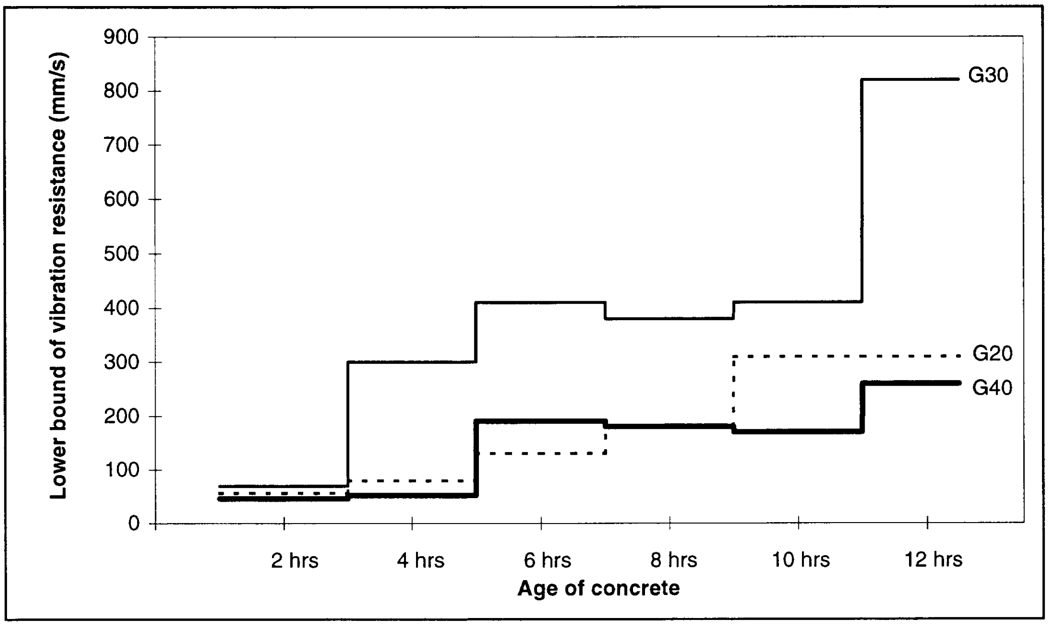


Figure 5.1 - Lower bounds of vibration resistances of G20, G30 and G40 concretes at ages between 2 hrs and 12 hrs

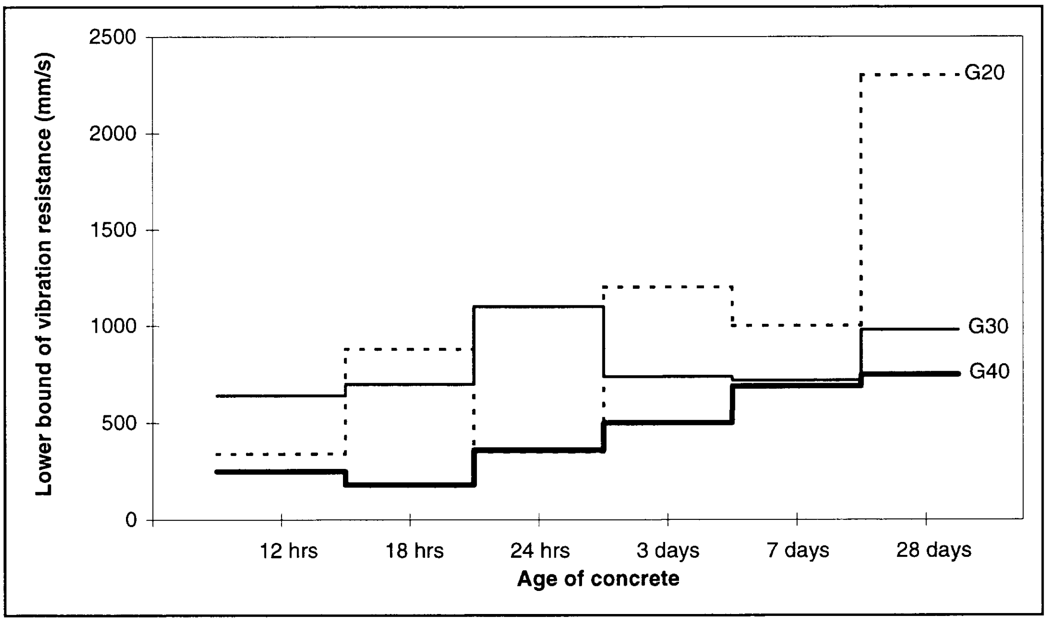


Figure 5.2 - Lower bounds of vibration resistances of G20, G30 and G40 concretes at ages between 12 hrs and 28 days

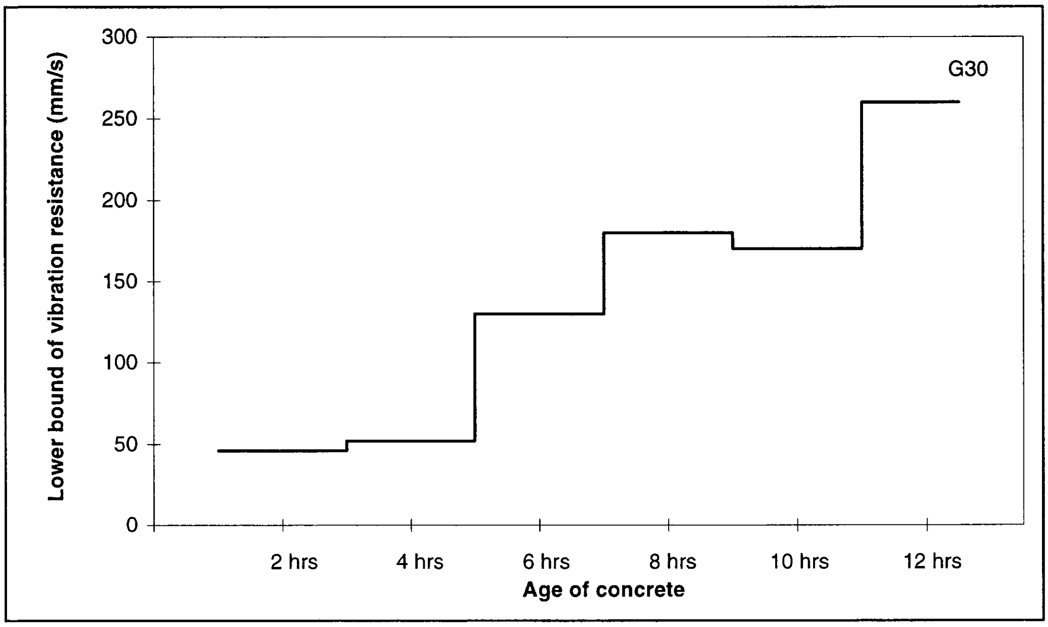


Figure 5.3 - Lower bound of vibration resistances for all concrete mixes tested at ages between 2 hrs and 12 hrs

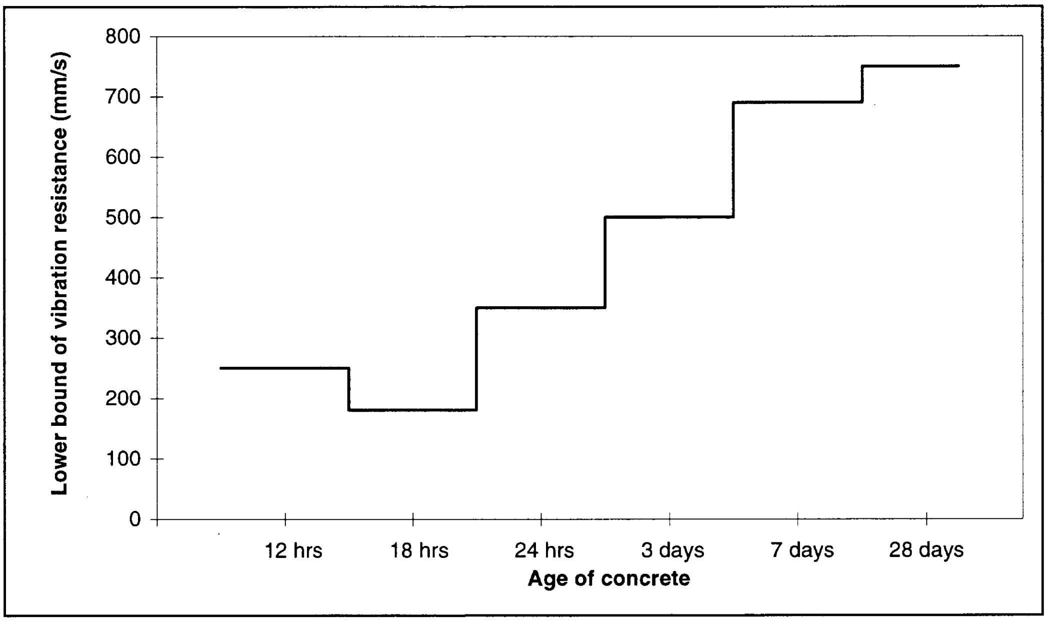


Figure 5.4 - Lower bound of vibration resistances for all concrete mixes tested at ages between 12 hrs and 28 days

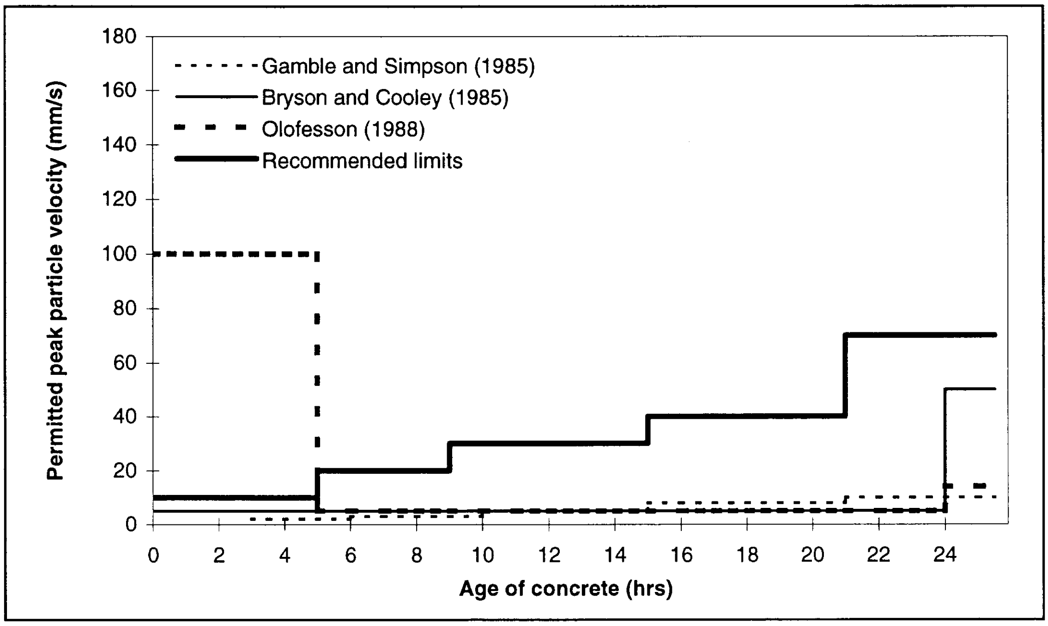


Figure 5.5 - Comparison of recommended vibration control limits to those of Gamble and Simpson, Bryson and Cooley, and Olofesson (age ≤ 24 hrs)

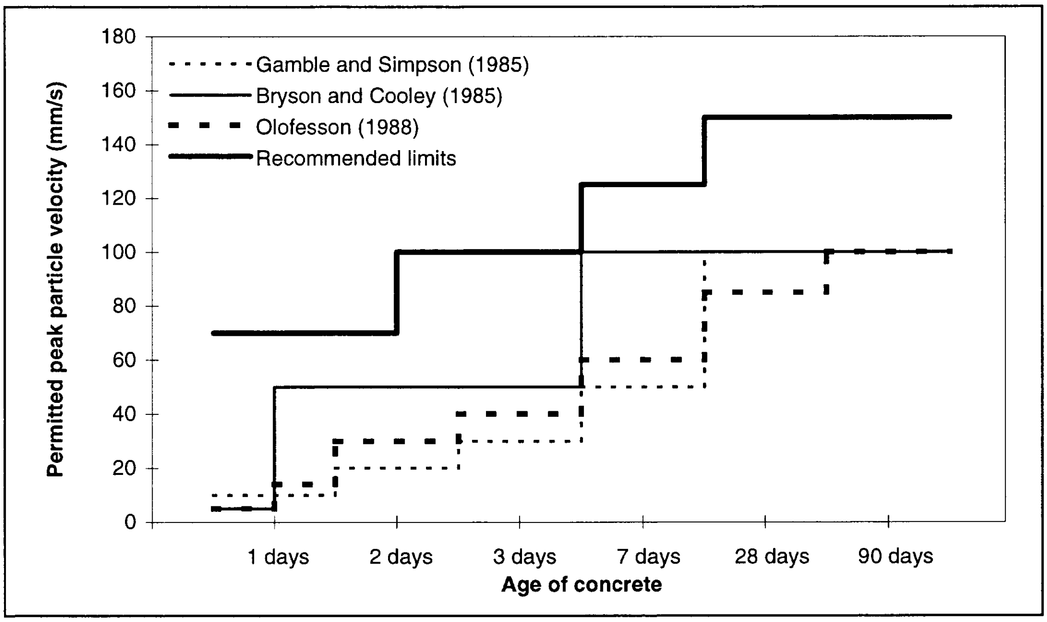


Figure 5.6 - Comparison of recommended vibration control limits to those of Gamble and Simpson, Bryson and Cooley, and Olofesson (age ≥ 24 hrs)

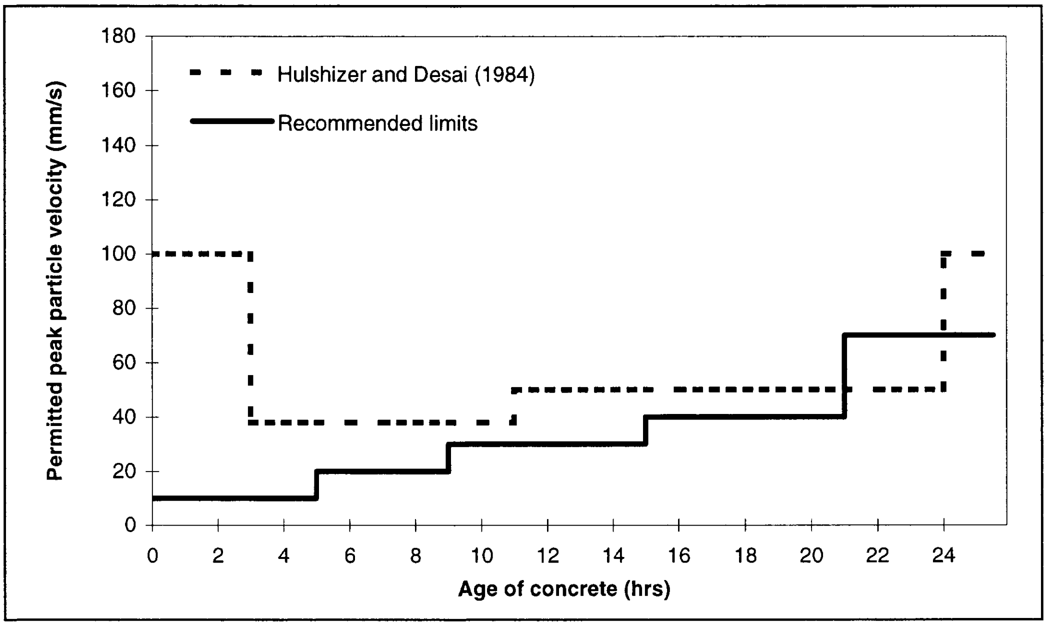


Figure 5.7 - Comparison of recommended vibration control limits to those of Hulshizer and Desai (age ≤ 24 hrs)

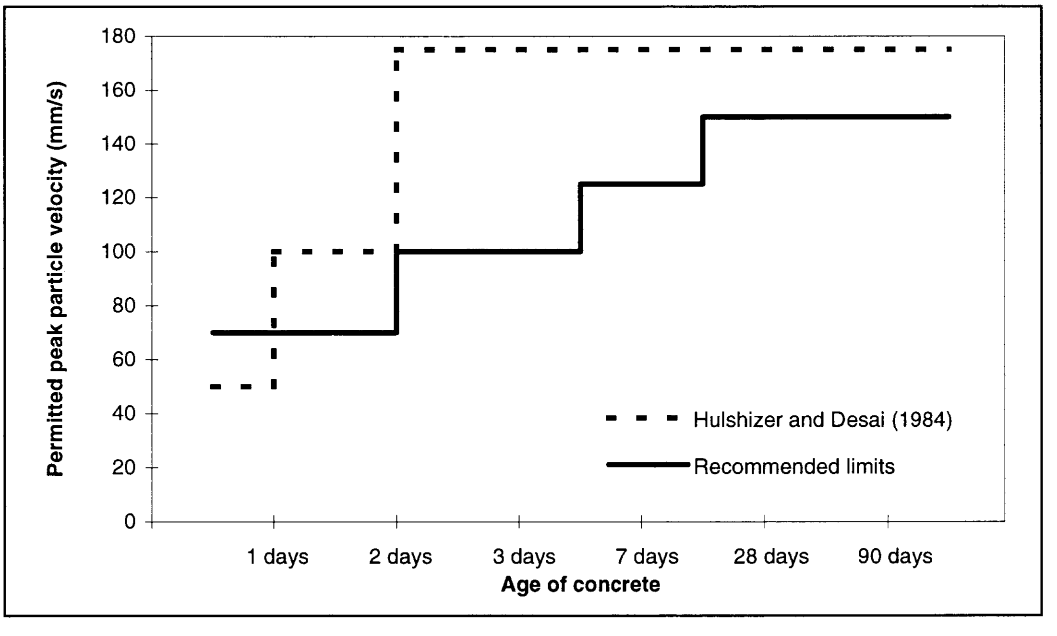


Figure 5.8 - Comparison of recommended vibration control limits to those of Hulshizer and Desai (age ≥ 24 hrs)

6. THEORETICAL ANALYSIS

6.1 Mean Value Estimates of Vibration Resistance

In the previous chapter, the lower bounds of the vibration resistances of the concrete mixes tested are used to establish vibration control limits to ensure that the vibration limits arrived at are on the safe side. However, for theoretical analysis, mean value estimates, which may not be on the safe side but should be more accurate, seem more appropriate. Mean value estimates of vibration resistance, abbreviated as ppv4, may be obtained as follows:

$$\text{ppv4} = \frac{1}{2}(\text{ppv1} + \text{ppv2}) \qquad \ldots(6.1)$$

In cases where none of the hammered prisms in the same set had failed (in these cases, ppv1 is given a value equal to “>ppv2”), ppv4 is taken to be equal to “>ppv2” meaning that the actual vibration resistance of the concrete tested should be higher than ppv2. On the other hand, in cases where all the hammered prisms had failed (in these cases, ppv2 is given a value equal to “<ppv1”), ppv3 is taken to have a value equal to “<ppv1” meaning that the actual vibration resistance of the concrete tested should be lower than ppv1. Values of ppv4 without any “<” or “>” sign should be more reliable as the ppv1 and ppv2 values from which they are derived are true ppv values at which failure of the concrete had or had not occurred. Values of ppv4 with either a “<” or “>” sign give only upper and lower limits respectively of the vibration resistance.

The ppv4 values so obtained are tabulated together with the material properties of the concrete mixes tested in Tables 6.1 - 6.3 for concretes at age $\leq$ 12 hrs. and in Tables 6.4 - 6.6 for concretes at age $\geq$ 12 hrs. Detailed analysis of the ppv4 results and correlation of the ppv4 values to the material properties of the concrete are presented in the next two sections, where an attempt of identifying the most important material parameter that determines the vibration resistance is also made.

Table 6.1 - Vibration resistances of G20 concretes at age ≤ 12 hrs.

Batch no.	Age at hammer test	Penetration resistance (MPa)	ppv4 (mm/s)
G20/0PFA/20C	2 hrs	0.01	>330
	4 hrs	1.43	>380
	6 hrs	-	295
	8 hrs.	-	>420
	10 hrs.	-	>480
	12 hrs.	-	>410
G20/0PFA/30C	2 hrs	0.03	>550
	4 hrs	2.97	120
	6 hrs	-	>180
	8 hrs.	-	>180
	10 hrs.	-	>820
	12 hrs.	-	>860
G20/25PFA/20C	2 hrs	0.01	>67
	4 hrs	0.22	>700
	6 hrs	1.74	200
	8 hrs.	-	1700
	10 hrs.	-	1310
	12 hrs.	-	2030
G20/25PFA/30C	2 hrs	0.01	>56
	4 hrs	0.19	>150
	6 hrs	2.67	>310
	8 hrs.	-	>1100
	10 hrs.	-	360
	12 hrs.	-	355

Table 6.2 - Vibration resistances of G30 concretes at age ≤ 12 hrs.

Batch no.	Age at hammer test	Penetration resistance (MPa)	ppv4 (mm/s)
G30/0PFA/20C	2 hrs	0.09	>100
	4 hrs	1.03	>500
	6 hrs	-	>670
	8 hrs.	-	495
	10 hrs.	-	475
	12 hrs.	-	>820
G30/0PFA/30C	2 hrs	0.11	>180
	4 hrs	3.54	>300
	6 hrs	-	>440
	8 hrs.	-	>600
	10 hrs.	-	800
	12 hrs.	-	>1200
G30/25PFA/20C	2 hrs	0.03	>90
	4 hrs	0.26	>620
	6 hrs	2.58	>410
	8 hrs.	-	>380
	10 hrs.	-	>790
	12 hrs.	-	>860
G30/25PFA/30C	2 hrs	0.04	>69
	4 hrs	1.18	>460
	6 hrs	-	665
	8 hrs.	-	980
	10 hrs.	-	>760
	12 hrs.	-	>830

Table 6.3 - Vibration resistances of G40 concretes at age ≤ 12 hrs.

Batch no.	Age at hammer test	Penetration resistance (MPa)	ppv4 (mm/s)
G40/0PFA/20C	2 hrs	0.03	>130
	4 hrs	0.43	230
	6 hrs	3.40	>420
	8 hrs.	-	>450
	10 hrs.	-	>390
	12 hrs.	-	>420
G40/0PFA/30C	2 hrs	0.15	>97
	4 hrs	1.53	>110
	6 hrs	-	>190
	8 hrs.	-	>250
	10 hrs.	-	225
	12 hrs.	-	>390
G40/25PFA/20C	2 hrs	0.01	197
	4 hrs	0.28	61
	6 hrs	2.20	>200
	8 hrs.	-	>180
	10 hrs.	-	>170
	12 hrs.	-	305
G40/25PFA/30C	2 hrs	0.01	67
	4 hrs	0.32	>250
	6 hrs	4.19	>200
	8 hrs.	-	>230
	10 hrs.	-	530
	12 hrs.	-	>700

Table 6.4 - Vibration resistances of G20 concretes at age ≥ 12 hrs.

Batch no.	Age at hammer test	f_{cu} (MPa)	f_t (MPa)	E_d (GPa)	c (m/s)	ppv4 (mm/s)
G20/0PFA/20C	12 hrs	3.9	0.43	-	-	605
	18 hrs	5.8	0.74	19.4	3140	905
	24 hrs	8.9	1.00	22.9	3410	525
	3 days	12.7	1.31	26.9	3700	1350
	7 days	17.7	1.54	30.6	3940	1400
	28 days	31.9	2.46	34.7	4200	2300
G20/0PFA/30C	12 hrs	4.8	0.48	15.2	2780	1055
	18 hrs	6.3	0.81	19.5	3150	1145
	24 hrs	11.0	1.21	22.5	3380	1020
	3 days	14.7	1.56	23.6	3460	2600
	7 days	21.5	1.99	28.1	3780	3000
	28 days	32.3	2.68	35.1	4220	3150
G20/25PFA/20C	12 hrs	1.3	0.13	8.4	2070	<340
	18 hrs	3.1	0.28	14.3	2700	1750
	24 hrs	4.9	0.49	16.4	2890	685
	3 days	8.9	1.01	21.6	3310	1850
	7 days	10.5	1.37	25.9	3630	1800
	28 days	22.2	1.76	31.8	4020	2700
G20/25PFA/30C	12 hrs	3.0	0.28	7.8	1990	>700
	18 hrs	3.4	0.32	12.9	2560	905
	24 hrs	5.0	0.61	16.8	2920	>1700
	3 days	9.9	1.09	21.6	3310	<1200
	7 days	12.2	1.08	26.8	3690	2900
	28 days	24.5	2.18	31.3	3990	<2900

Table 6.5 - Vibration resistances of G30 concretes at age ≥ 12 hrs.

Batch no.	Age at hammer test	f_{cu} (MPa)	f_t (MPa)	E_d (GPa)	c (m/s)	ppv4 (mm/s)
G30/0PFA/20C	12 hrs	4.1	0.40	13.9	2640	780
	18 hrs	7.5	0.76	18.9	3080	850
	24 hrs	10.2	0.94	20.4	3200	1350
	3 days	18.6	1.82	28.6	3790	870
	7 days	28.7	2.09	32.1	4010	1350
	28 days	39.3	2.87	36.5	4280	1300
G30/0PFA/30C	12 hrs	5.9	0.52	16.3	2860	1350
	18 hrs	9.0	0.81	20.2	3180	1095
	24 hrs	10.4	0.99	20.9	3240	1500
	3 days	21.9	1.79	27.7	3730	1250
	7 days	37.1	2.72	33.8	4120	1160
	28 days	41.9	3.07	35.9	4240	<1400
G30/25PFA/20C	12 hrs	2.6	0.27	12.2	2490	1050
	18 hrs	4.8	0.41	15.9	2840	925
	24 hrs	7.4	0.75	19.0	3110	1300
	3 days	10.4	1.09	24.7	3540	1550
	7 days	18.9	1.62	29.3	3860	1005
	28 days	33.9	2.54	34.9	4210	1700
G30/25PFA/30C	12 hrs	3.6	0.37	12.6	2530	670
	18 hrs	7.2	0.62	18.8	3090	880
	24 hrs	8.6	0.77	20.1	3200	1250
	3 days	14.8	1.30	27.2	3720	1150
	7 days	24.4	2.20	30.1	3910	1300
	28 days	43.1	3.04	35.7	4260	1190

Table 6.6 - Vibration resistances of G40 concretes at age ≥ 12 hrs.

Batch no.	Age at hammer test	f_{cu} (MPa)	f_t (MPa)	E_d (GPa)	c (m/s)	ppv4 (mm/s)
G40/0PFA/20C	12 hrs	2.0	0.29	10.3	2290	<520
	18 hrs	5.6	0.66	21.5	3310	1150
	24 hrs	7.1	0.73	23.1	3430	1010
	3 days	23.5	1.79	29.0	3840	555
	7 days	31.2	2.40	32.6	4070	925
	28 days	46.9	3.17	38.2	4410	?
G40/0PFA/30C	12 hrs	1.6	0.09	-	-	445
	18 hrs	13.0	1.06	22.3	3370	255
	24 hrs	13.4	1.12	23.0	3420	1060
	3 days	27.0	2.08	28.4	3800	<1800
	7 days	38.9	2.60	31.3	3990	845
	28 days	50.8	3.26	36.0	4280	810
G40/25PFA/20C	12 hrs	0.7	0.03	-	-	385
	18 hrs	2.0	0.33	8.8	2110	410
	24 hrs	3.9	0.64	16.1	2860	490
	3 days	18.0	1.83	27.1	3710	1200
	7 days	26.5	2.31	31.3	3990	<1200
	28 days	40.5	2.65	32.9	4090	1075
G40/25PFA/30C	12 hrs	2.2	0.20	8.1	2030	540
	18 hrs	4.3	0.37	18.1	3030	1000
	24 hrs	8.0	0.65	19.8	3170	925
	3 days	18.3	1.52	25.8	3620	<1400
	7 days	28.0	1.96	29.6	3880	<1400
	28 days	48.0	3.03	36.6	4310	1800

6.2 Vibration Resistance of Concrete at Age ≤ 12 Hours

At concrete ages of less than 12 hours, due to the difficulty of removing the moulds of the concrete for any stiffness or strength measurement, the only mechanical properties of the concrete measured were the penetration resistance and stiffening time of the mortar fraction of the concrete. These material properties have been presented in Section 4.2. Herein, the penetration resistances of the concretes at ages of 2 hrs., 4 hrs. and 6 hrs. are tabulated alongside the ppv4 results in Tables 6.1 - 6.3 for detailed study of any correlation between the vibration resistance and the penetration resistance of the concrete.

The ppv4 results presented in Tables 6.1 - 6.3 are plotted against the age of the concrete for the G20, G30 and G40 concretes in Figs. 6.1, 6.2 and 6.3 respectively. When the ppv4 results are plotted in these figures, the “<” and “>” signs of the ppv4 values are ignored (in fact, none of the ppv4 values has the “<” sign). From these figures, it is seen that the ppv4 results are very scattered but there is still a general trend that the vibration resistance increases with the age of the concrete. To better illustrate the spread of the test results, all the data points for the different batches of concrete tested are plotted together in Fig.6.4. The large scatter of the ppv4 results reflects the erratic nature of the vibration resistance of concrete. Quite often, at a given concrete age, the range of the ppv4 values is several hundred percent of the value of the lower bound of the range. In Fig.6.4, a best fit cubic polynomial equation for the data points is also presented. This best fit polynomial equation is somehow very close to a linear equation as indicated by the straightness of the polynomial curve. Hence, although the ppv4 results are very scattered due to the superposition on them of large intrinsic variation of the vibration resistance of concrete, their mean value increases steadily and more or less linearly with the age of the concrete up to an age of about 12 hours.

In order to investigate whether the vibration resistance of concrete can be correlated to the penetration resistance, the ppv4 values are plotted against the penetration resistance in Fig.6.5 up to a penetration resistance of about 4.0 MPa - the maximum measuring capacity of the stiffening time tester used. Again, the data points plotted are very scattered. A best fit line for the data points is also drawn in the figure to reveal the trend of variation of the vibration resistance with the penetration resistance. From this line, it can be seen that although the correlation between the vibration resistance and the penetration resistance is rather weak, there is a marginal increase in the vibration resistance as the penetration resistance increases from zero to about 4.0 MPa. Actually, at a penetration resistance of less than 4.0 MPa, the concrete is still plastic and thus behaves more like a colloidal material than a hardened solid. At this stage, the concrete has very little stiffness that could help to resist shock vibration. This is probably why the vibration resistance is almost independent of or increases only very slightly with the penetration resistance. What seems more important is the sensitivity of the mortar matrix of the plastic concrete to possible disruptions caused by the shock vibration. For this same reason, the stiffening time of the concrete does not appear to have significant influence on the variation of the vibration resistance with time.

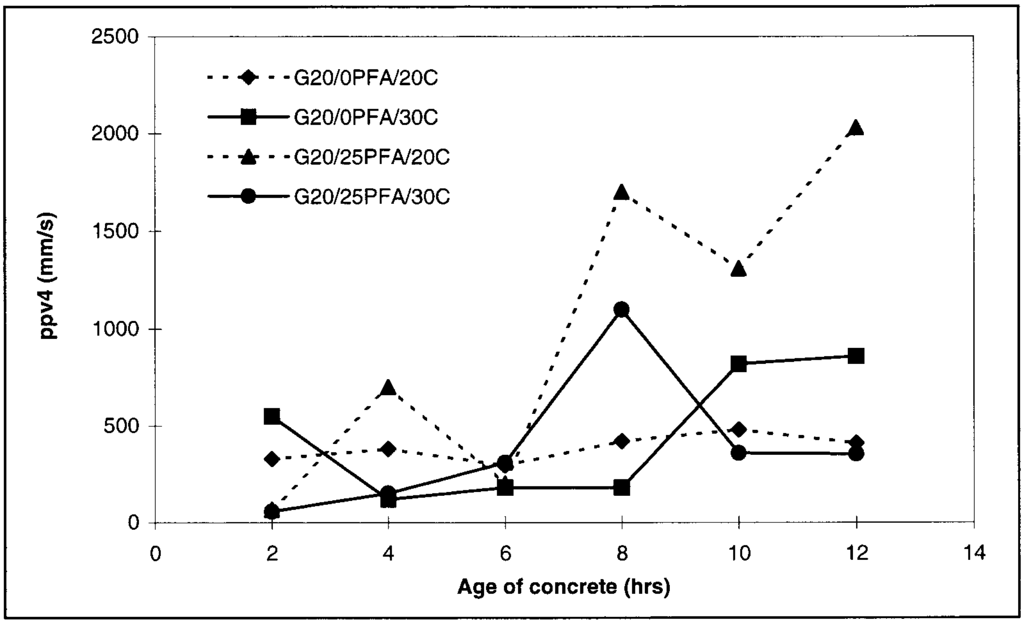


Figure 6.1 - ppv4 against age for G20 concretes (age ≤ 12 hrs)

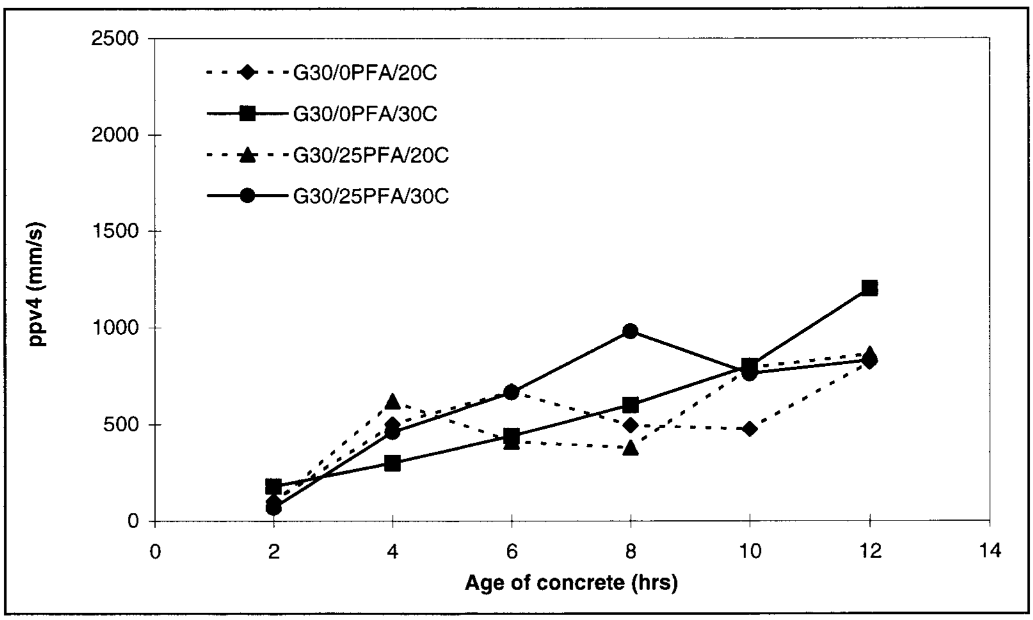


Figure 6.2 - ppv4 against age for G30 concretes (age ≤ 12 hrs)

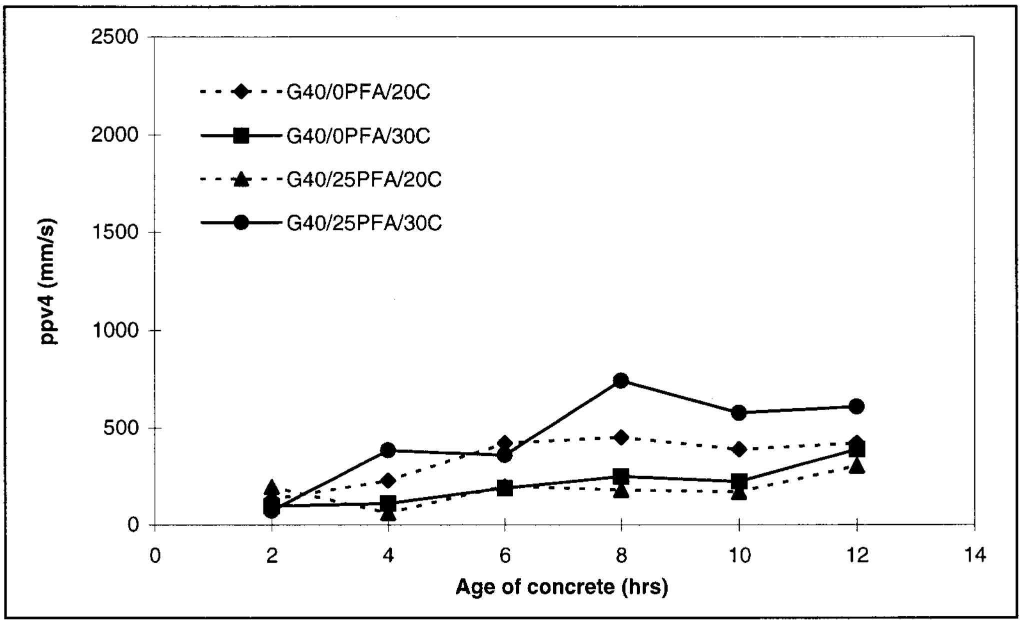


Figure 6.3 - ppv4 against age for G40 concretes (age ≤ 12 hrs)

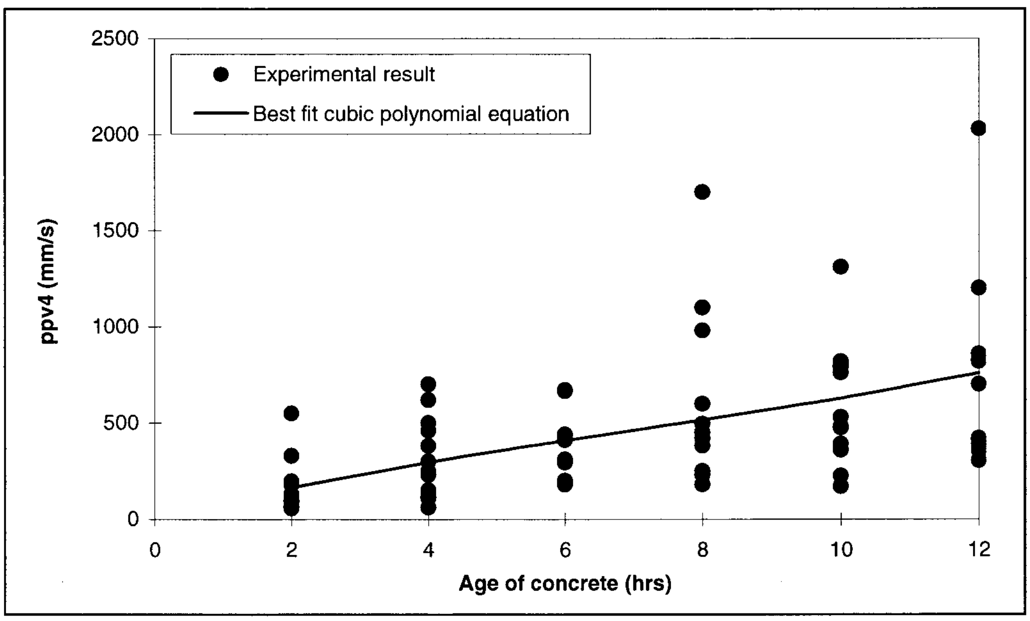


Figure 6.4 - ppv4 against age for all concrete mixes tested (age ≤ 12 hrs)

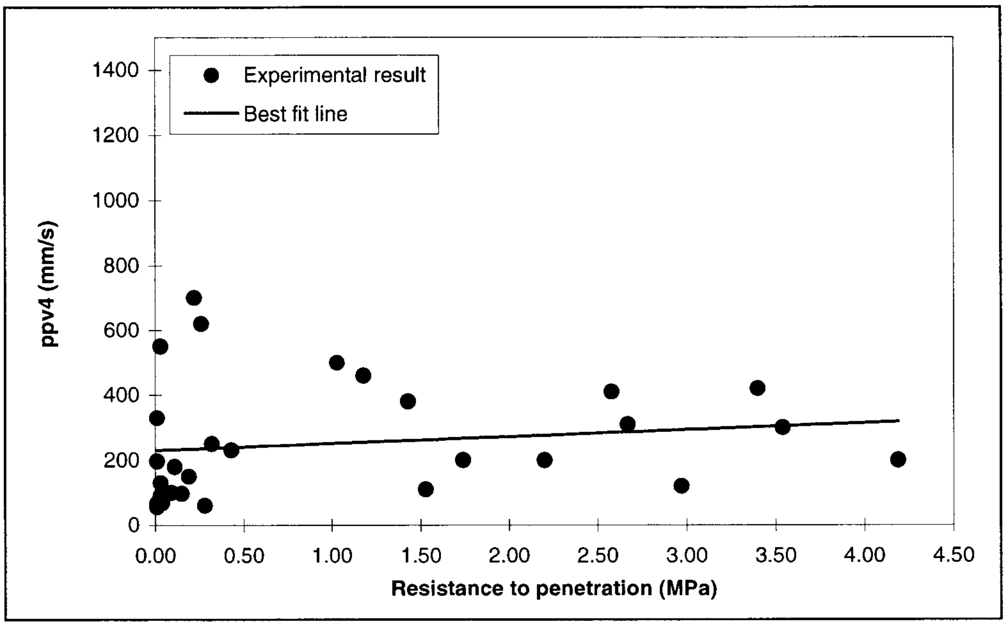


Figure 6.5 - ppv4 against penetration resistance for all concrete mixes tested

6.3 Vibration Resistance of Concrete at Age ≥ 12 Hours

At concrete ages of more than 12 hours, the compressive strength, tensile strength, dynamic and static moduli and longitudinal wave propagation velocity of the concretes had been measured and the results have been presented in Section 4.3. These material properties are listed alongside the ppv4 results in Tables 6.4 - 6.6 to see whether the vibration resistance of the concrete can be correlated to them.

The ppv4 results are plotted against the age of the concrete for the G20, G30 and G40 concretes in Figs. 6.6, 6.7 and 6.8 respectively. As before, when the ppv4 results are plotted, the "<" and ">" signs of the ppv4 values are ignored. From these figures, it is seen that even for hardened concrete (concrete older than 12 hours and hard enough for their moulds to be removed), the ppv4 results are very scattered. There is, nevertheless, an overall trend that the vibration resistance increases with the age of the concrete. To better illustrate the spread of the test results, all the data points for the different batches of concrete tested are plotted together in Fig.6.9. A best fit curve in the form of a power equation is also plotted in the figure to illustrate the trend of variation of the vibration resistance with age. It can be seen from this curve that beyond the age of 12 hours, the vibration resistance of concrete continues to increase with time up to at least the age of 28 days but the rate of increase gradually decreases with time.

The ppv4 results are plotted against the cube compressive strength of the concrete at the time of hammering in Fig.6.10. Although the shock vibrations are known to have little effect on the compressive strength of the concrete, it is believed that the compressive strength, which is normally taken as a measure of the quality or grade of the concrete, has a bearing on the ability of the concrete to resist high intensity shock vibrations. The data points plotted on the graph are fairly scattered and it is thus not easy to establish a clear cut correlation between the vibration resistance and the cube compressive strength. Several forms of equation for curve fitting of the data points, namely: first and second order polynomial equations, exponential equation and power equation, have been tried and it turns out after a series of regression analysis that a power equation gives the highest correlation index. The best fit power equation is found to be given by:

$$\text{ppv4} = 585 \cdot (f_{\text{cu}})^{0.26} \qquad \ldots(6.1)$$

in which the units for ppv4 and f_{cu} are mm/s and MPa respectively. It is plotted in Fig.6.10 to illustrate how it fits the data points. Due to the large scatter of the ppv4 results, the value of R^2 for this equation, though already the highest among those of the different forms of equation tried, is only 0.24.

However, the Eqn.6.1 above gives only a mean value estimate of the vibration resistance. As there might be large intrinsic variation in the vibration resistance, the actual vibration resistance of the concrete could be significantly higher or lower than the value predicted by this equation. What is more important is that there is a very high probability of about 50% of having the actual vibration resistance lower than the predicted mean value. Therefore, this equation is not considered safe to use in engineering practice. Like the use of characteristic strength in structural design, it is proposed to use a characteristic value estimate of the vibration resistance instead of the above mean value estimate to cater for the relatively large intrinsic variation of vibration resistance. As for the case of characteristic strength, the characteristic vibration resistance is defined as the vibration level at which no more than 5% of the concrete is expected to fail. Assuming that the characteristic vibration resistance, denoted by ppvc, is also related to the compressive strength of the concrete by a power equation of the form: ppvc = $k(f_{\text{cu}})^{0.26}$, it may be estimated by finding the value of k such that only 5% of the data points fall below this curve. In Fig.6.10, there are a total of 60 data points. 5% of the number of data points is 3. The value of k such that only three data points fall below the curve and the third and fourth lowest data points are at equal distance from the curve is found by trial and error to be 290. Hence, the characteristic vibration resistance may be estimated by the following equation:

$$\text{ppvc} = 290 \cdot (f_{\text{cu}})^{0.26} \qquad \ldots(6.2)$$

This equation, plotted as a dotted line in Fig.6.10, may also be used to establish the vibration control limits for avoiding damage to concrete during shock vibration. Adopting a factor of safety of 5, the safe vibration limits may be obtained by dividing the ppvc values so determined by 5, as listed in Table 6.7. Bearing in mind that most concretes cured at a normal temperature of 20ºC - 30ºC develop a cube strength of at least 3 MPa in 24 hours and their full strength potential in 28 days, the safe ppv limits listed in Table 6.7 of 77 - 160 mm/s for concretes with cube strength ranging from 3 to 50 MPa agree fairly well with the corresponding safe ppv limits given in Section 5.3 of 70 - 150 mm/s for concretes at age between 24 hours to 28 days. Hence, in most cases, similar vibration control limits would be obtained regardless of whether the ppv limits based on age as given in Section 5.3 or the ppv limits based on cube compressive strength as listed in Table 6.7 are used.

Table 6.7 - Vibration control limits based on mean cube compressive strength

f_{cu} (MPa)	ppvc (mm/s)	Safe ppv limit (mm/s)
3.0	385	77
5.0	440	88
10.0	530	106
20.0	630	126
30.0	700	140
40.0	755	151
50.0	800	160

As the most likely damage that would be caused by shock vibration is cracking, the tensile strength of the concrete should have a more direct influence on the vibration resistance than the compressive strength. In order to study how the tensile strength affected the vibration resistance, the ppv4 results are plotted against the split cylinder tensile strength of the concrete at the time of hammering in Fig.6.11. Again, the data points plotted on the graph are fairly scattered. As before, several different types of curve: first and second order polynomial equations, exponential equation and power equation, have been tried to fit the data points and a power equation is found to give the best correlation. The best fit power equation is given by:

$$\text{ppv4} = 1098 \cdot (f_t)^{0.29} \qquad \ldots(6.3)$$

in which the units for ppv4 and f_t are mm/s and MPa respectively. It is plotted in Fig.6.11 to illustrate how it fits the data points. The value of R^2 for this equation is 0.26, which is slightly better than the value of 0.24 for Eqn.6.1. Hence, it may be said that the vibration resistance correlates to the tensile strength slightly better than to the compressive strength.

Following the method used previously for estimating the characteristic vibration resistance from the cube compressive strength, the characteristic vibration resistance may also be estimated from the tensile strength by assuming that it is related to the tensile strength in the form ppvc = $k(f_{cu})^{0.29}$ and finding the value of k such that only 5% of the data points fall below the curve. The value of k so determined is found to be 527. Hence, the characteristic vibration resistance may be estimated from the tensile strength by the following equation:

$$\text{ppvc} = 527 \cdot (f_t)^{0.29} \qquad \ldots(6.4)$$

The above equation is plotted as a dotted line in Fig.6.11. Dividing the ppvc values by a factor of safety of 5, a set of vibration control limits can be obtained, as listed in Table 6.8.

These vibration control limits may be compared to those listed in Table 6.7 by converting the cube compressive strength values in Table 6.7 to tensile strength values using the simple rule that the tensile strength of concrete is approximately equal to 8 - 10 % of the cube compressive strength. After the conversion, it can be seen that the safe ppv limits given in Table 6.8 agree fairly well with those listed in Table 6.7.

Table 6.8 - Vibration control limits based on split cylinder tensile strength

f_t (MPa)	ppvc (mm/s)	Safe ppv limit (mm/s)
0.1	270	54
0.3	370	74
0.5	430	86
1.0	525	105
2.0	645	129
3.0	725	145
4.0	790	158

Before the theoretical study continues, it should be noted from the above analysis that the vibration resistance is neither proportional to the compressive strength nor proportional to the tensile strength. The believe of many engineers that the vibration resistance is proportional to the strength of concrete (however, it has never been clear whether the strength should be the compressive strength or the tensile strength) is unfounded.

Since the maximum dynamic tensile strain ε_d induced during shock vibration is dependent on the longitudinal wave propagation velocity c as given by the following equation:

$$\varepsilon_d = \frac{v}{c} \qquad \ldots(6.5)$$

where v is the peak particle velocity of the shock wave, it is expected that the longitudinal wave propagation velocity may also have significant influence on the vibration resistance of concrete. To study any possible dependence of the vibration resistance on the value of c, the ppv4 results are plotted against the corresponding values of c at the time of hammering in Fig.6.12. From this figure, it can be easily seen that in general, the vibration resistance is higher when the longitudinal wave propagation velocity is on the high side. As before, several different types of curve: first and second order polynomial equations, exponential equation and power equation, have been tried to fit the data points and a power equation is found to give the best correlation. The best fit power equation is given by:

$$\text{ppv4} = 0.047 \cdot (c)^{1.24} \qquad \ldots(6.6)$$

in which the units for ppv4 and c are mm/s and m/s respectively. It is plotted in Fig.6.12 as a solid line. Due to the large scatter of the ppv4 results, the value of R^2 for this equation is only 0.21. Assuming that the characteristic vibration resistance is related to the value of c by ppvc = $k(c)^{1.24}$ and finding the value of k such that only 5% of the data points fall below this curve, the equation that correlates the characteristic vibration resistance to the longitudinal wave propagation velocity is obtained as:

$$\text{ppvc} = 0.023 \cdot (c)^{1.24} \qquad \ldots(6.7)$$

It is plotted in Fig.6.12 as a dotted line. For most hardened concretes, the value of c ranges from about 2000 m/s at an age of 12 hours to about 4000 m/s upon development of full strength. Substituting these values of c into Eqn.6.7, the corresponding values of characteristic vibration resistance are evaluated as 285 mm/s and 673 mm/s respectively. Dividing these values by a factor of safety of 5, the safe ppv limits for hardened concrete are obtained as 57 mm/s for young hardened concrete at an age of 12 hours and 135 mm/s for fully hardened concrete at an age of about 28 days. These safe ppv limits are in good agreement with the corresponding values presented in Section 5.3.

Making use of Eqn.6.5, the dynamic tensile strain ε_d' at which the concrete failed when subjected to shock vibration may be estimated as ppv4/c. The dynamic tensile strain values so evaluated are plotted against the cube compressive strength in Fig.6.13 to study how the dynamic tensile strain capacity varies with the compressive strength. Curve fitting of the data points yields the following equation for the dynamic tensile strain capacity:

$$\varepsilon_d' = 321 \cdot (f_{cu})^{0.015} \qquad \ldots(6.8)$$

The value of R^2 for this equation is only 0.001 indicating that the dependence of the dynamic tensile strain capacity on the compressive strength of the concrete is rather weak. Following the method used before, the characteristic dynamic tensile strain capacity $\overline{\varepsilon_d}'$ is obtained as:

$$\overline{\varepsilon_d}' = 160 \cdot (f_{cu})^{0.015} \qquad \ldots(6.9)$$

The above two equations are plotted in Fig.6.13 as solid and dotted lines respectively. From these two curves, it can be seen that the dynamic tensile strain capacity increased only marginally with the compressive strength; in fact, it increased by only about 4% when the cube compressive strength increased from 3 MPa to 30 MPa. It is also noted that for hardened concrete, whose cube compressive strength is expected to be higher than 3 MPa, the characteristic value of dynamic tensile strain capacity is generally larger than 160 micro-strain. Hence, for simplicity, the characteristic dynamic tensile strain capacity of hardened concrete may be just taken as a constant value of 160 micro-strain regardless of the actual strength of the concrete. Dividing this value of characteristic dynamic tensile strain capacity by a factor of safety of 5, a safe dynamic tensile strain limit of 32 micro-strain is obtained.

The above study on dynamic tensile strain capacity is of particular importance to underground concrete structures which are restrained to deform together with the surrounding soil (restrained structures). Let the wave propagation velocity in the surrounding soil be denoted by c_s. The maximum dynamic strain ε_s produced by the propagation of a shock wave of peak particle velocity v through the soil is given by:

$$\varepsilon_s \quad = \quad \frac{v}{c_s} \qquad \text{…(6.10)}$$

If the surrounding soil has a lower stiffness than the concrete and hence a lower wave propagation velocity, i.e. $c_s < c$, then the strain produced in the soil will be larger than that produced in an independent concrete structure (a concrete structure free to deform independently of the surrounding soil). However, since the underground concrete structure is restrained to deform together with the surrounding soil, the strain produced in the concrete is forced to be equal to that produced in the soil. Therefore, when evaluating the maximum dynamic tensile strain ε_d produced in restrained concrete structures, the smaller of the values of c and c_s should be used, as in the following equation:

$$\varepsilon_d \quad = \quad \frac{v}{\min.(c, c_s)} \qquad \text{…(6.11)}$$

The vibration control should thus be exercised such that the dynamic tensile strain evaluated by the above equation is within the safe dynamic tensile strain limit. Working the other way round, the vibration control limits for restrained concrete structures may be set as:

$$\text{safe ppv limit} = \min.(c, c_s) \times \text{safe dynamic tensile strain limit} \qquad \text{…(6.12)}$$

When $c_s < c$, as for hardened concrete structures buried in soft soil, the safe ppv limit will be governed by the value of c_s. On the other hand, when $c_s > c$, as for green concrete cast against soil or hardened concrete structures embedded in hard rock, the safe ppv limit will be governed by the value of c, or in other words, the surrounding soil will have no effect on the vibration control limit.

In Chapter 2, it has been mentioned that the vibration resistance of concrete may be estimated from the wave propagation velocity c, dynamic tensile strength f_{td} and dynamic modulus E_d using the following equation:

$$v' = \frac{c}{E_d} f_{td} \qquad \text{...(6.12)}$$

The dynamic tensile strength had not been measured in this project. Nevertheless, as suggested by Raphael (1984), the dynamic tensile strength may be taken simply as 1.5 times the static tensile strength. In order to investigate whether the actual measured values of vibration resistance agree with the above theoretical prediction, the ppv4 results are plotted against the corresponding values of v' in Fig.6.14. A straight line whose equation is given by ppv4 $= v'$ is also drawn on the graph. It is seen that all the data points are above this line.

This indicates that the actual vibration resistance is generally higher than the above theoretical value. In fact, the measured vibration resistance is often two to three times higher than the theoretical vibration resistance. The reason why the measured vibration resistance is often so much higher than the above theoretical value is not known. Perhaps the theoretical value given in Eqn.6.12 gives just the threshold value of vibration intensity required to cause cracking or failure of the concrete; for unknown reasons, some of the concrete may be able to withstand a substantially higher vibration intensity before it cracks or fails.

Since the lowest data point lies only marginally above the line ppv4 $= v'$ in Fig.6.14, the theoretical value given by Eqn.6.12 may be taken as a lower bound to the vibration resistance. This is probably just an unintended coincidence. Nevertheless, a conservative estimate of the characteristic vibration resistance may be obtained using Eqn.6.12 as:

$$\text{ppvc} = \frac{c}{E_d} f_{td} \qquad \text{...(6.13)}$$

A safe ppv limit in terms of the values of c, E_d and f_{td} may then be derived by dividing the above ppvc expression by a factor of safety of 5.

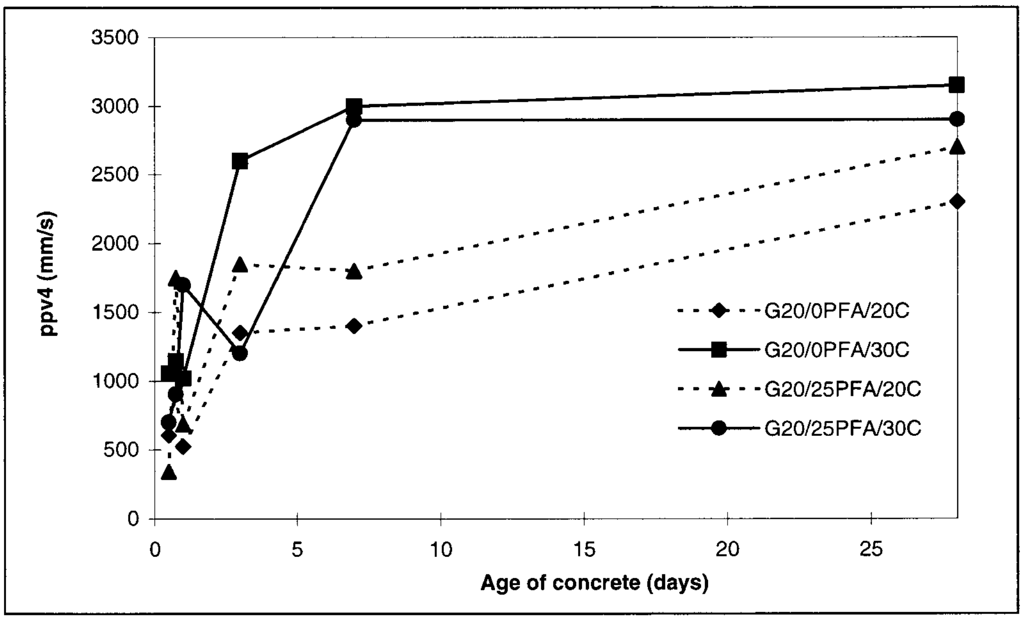


Figure 6.6 - ppv4 against age for G20 concretes (age ≥ 12 hrs)

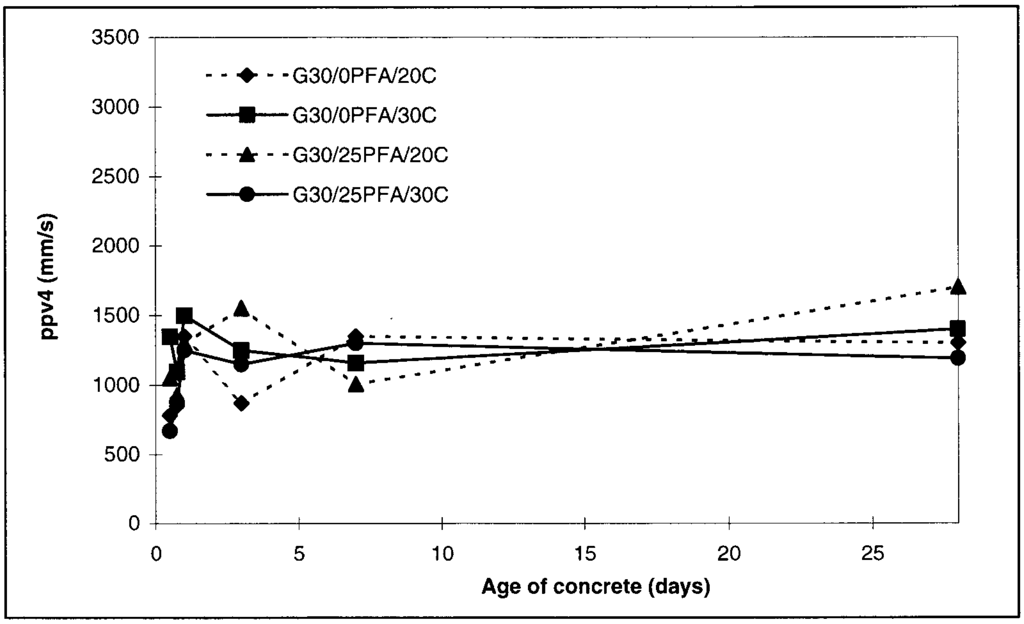


Figure 6.7 - ppv4 against age for G30 concretes (age ≥ 12 hrs)

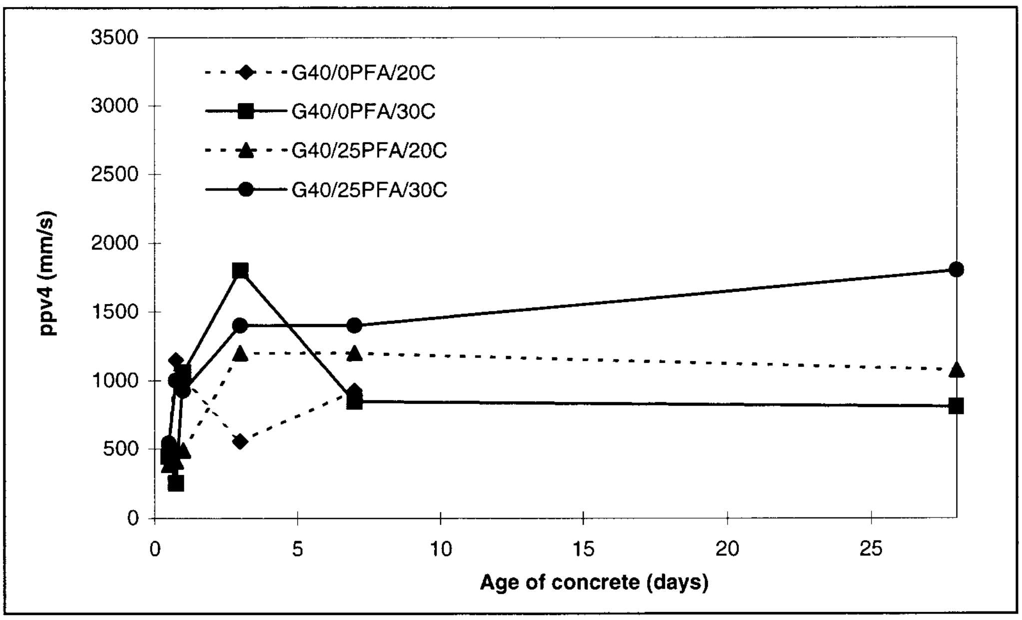


Figure 6.8 - ppv4 against age for G40 concretes (age ≥ 12 hrs)

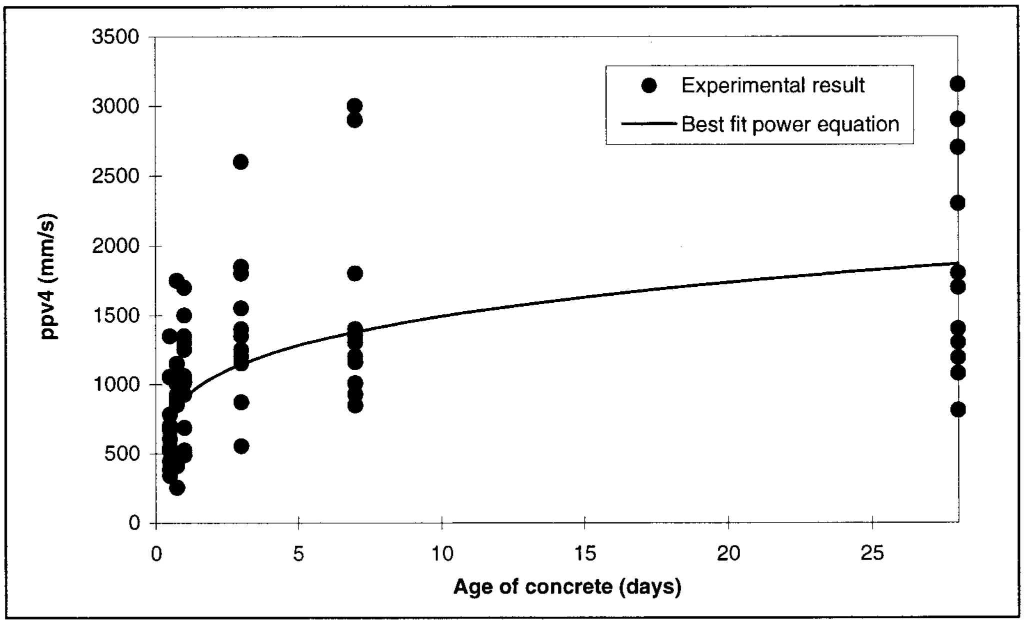


Figure 6.9 - ppv4 against age for all concrete mixes tested (age ≥ 12 hrs)

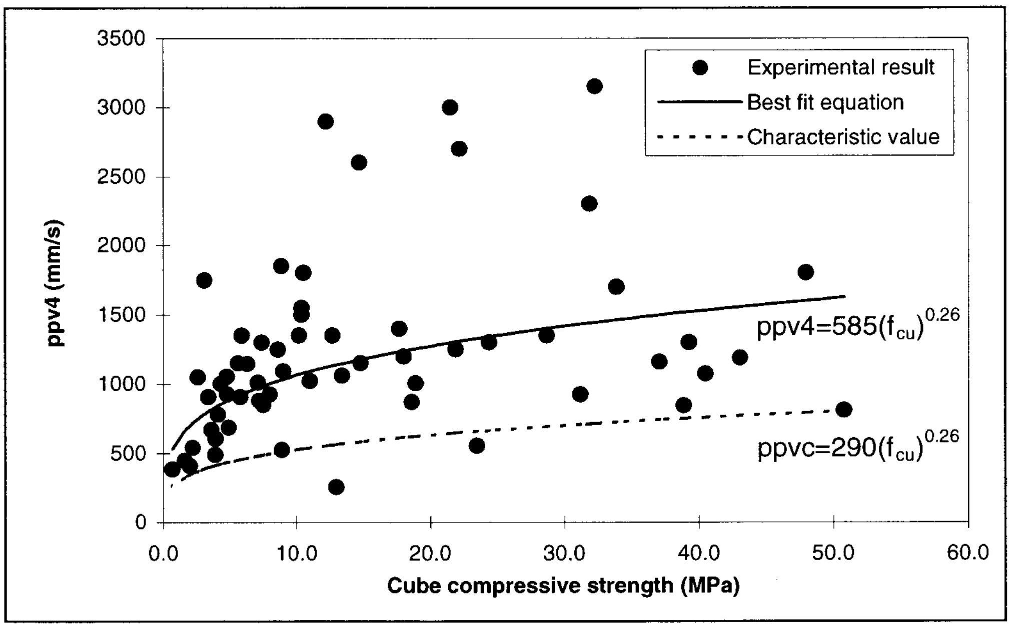


Figure 6.10 - ppv4 against cube compressive strength

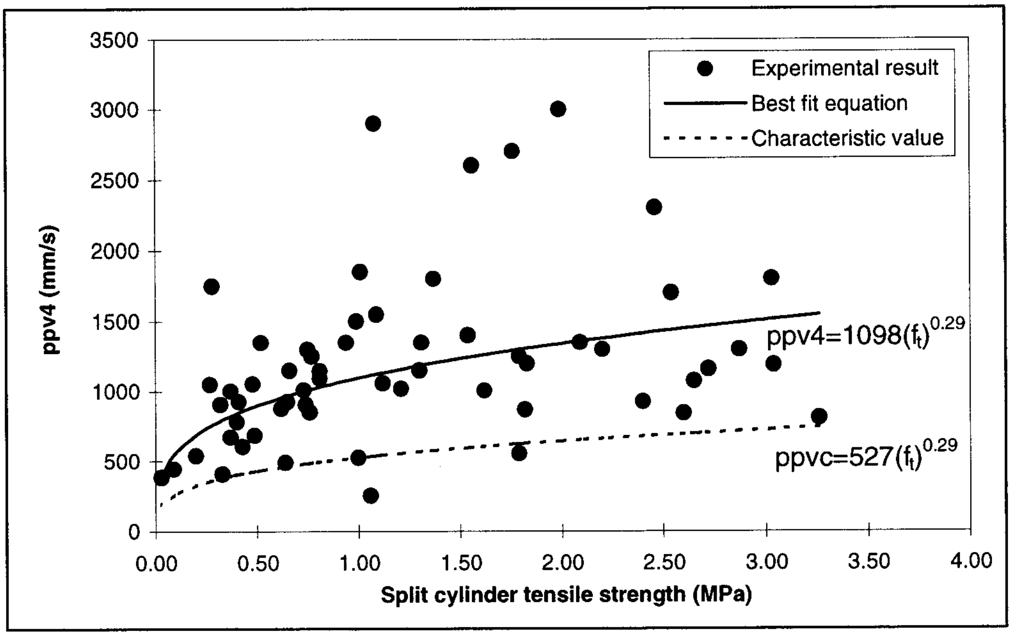


Figure 6.11 - ppv4 against split cylinder tensile strength

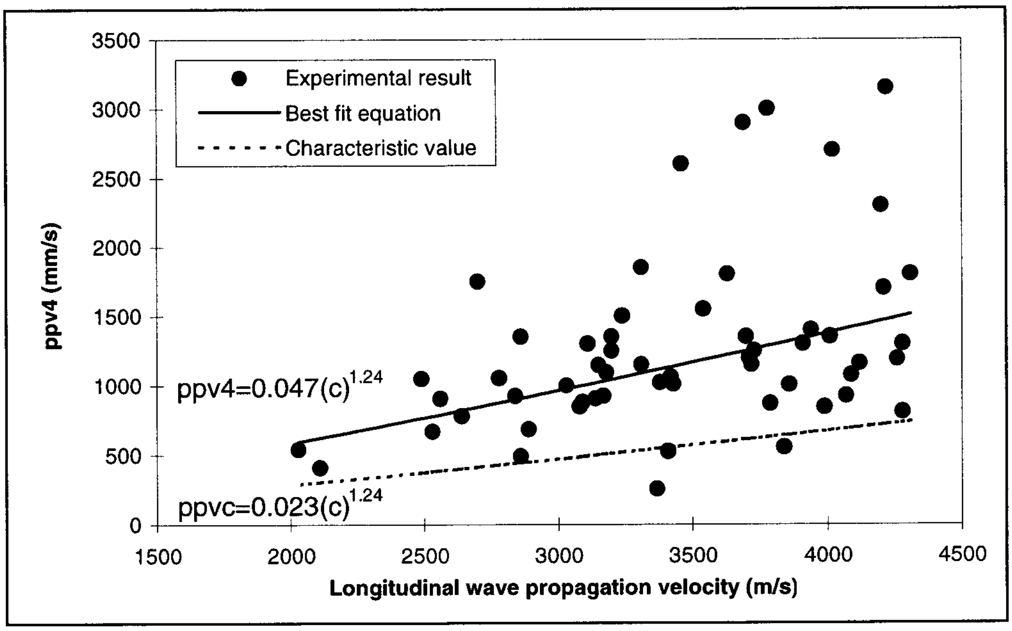


Figure 6.12 - ppv4 against longitudinal wave propagation velocity

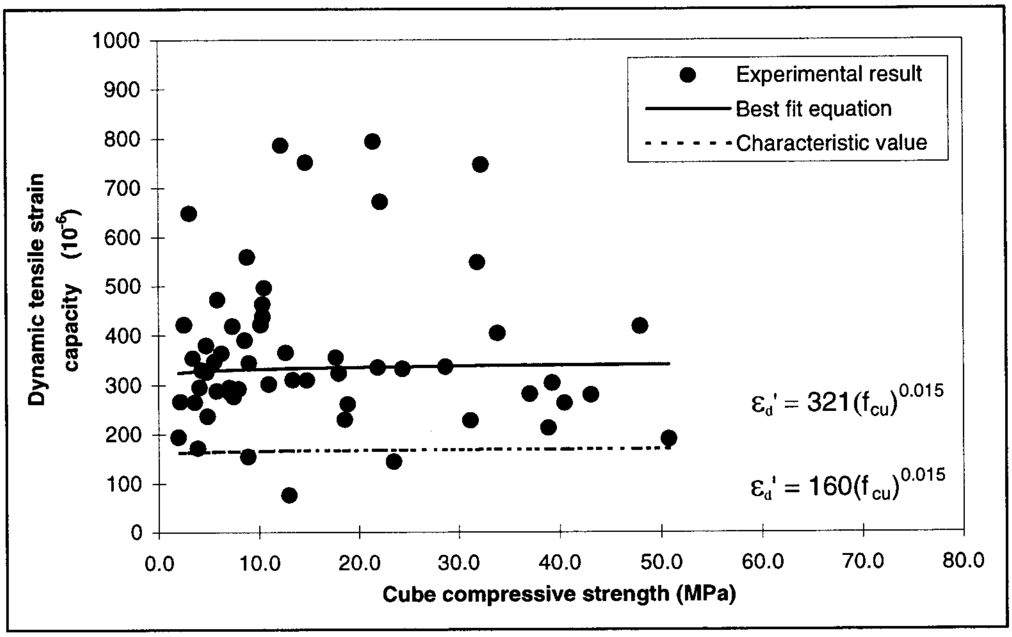


Figure 6.13 - Dynamic tensile strain capacity against cube compressive strength

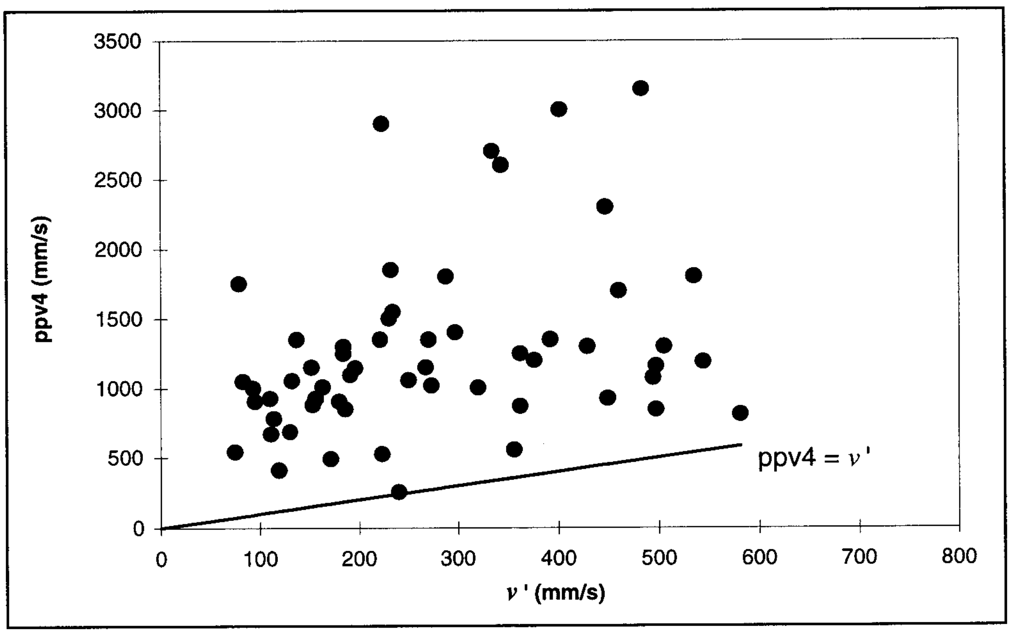


Figure 6.14 - ppv4 against v'

7. DISCUSSIONS

7.1 Failure Criterion for Vibration Resistance

The vibration resistance of concrete is the vibration intensity at which the concrete fails. However, the failure criterion has never been properly defined. In fact, the literature review carried out in the Literature Review Report No.1 of the present studied revealed that different researchers used quite different failure criteria.

Hulshizer and Desai (1984) considered the tensile strength of concrete unimportant for the reason that reinforced concrete is basically designed as a “cracked section”. Therefore, in their study of the effects of blasting vibration on concrete, no effort was made to test the tensile strength of the concrete; they tested only the compressive, shear and bond strengths of the vibrated concrete and compared the results to those of un-vibrated concrete to evaluate the vibration resistance of the concrete. Restricting only to compressive, shear and bond strengths, they found no significant change in these strength values despite vibrations of intensity up to 250 mm/s had been applied. At the end, they concluded there was no sign to indicate that the vibrated concrete would not perform as if they were un-vibrated and recommended relatively high vibration limits for blasting control.

On the other hand, as cited by Dowding (1985), Esteves (1978) defined failure as “fracture of concrete” when he performed his hammering tests. It is not clear what fracture means. Judging by common sense, fracture probably means complete cracking of a whole section of the concrete such that it breaks into at least two pieces or the appearance of at least one visible crack on the concrete surface. As the concrete prisms tested were still contained in moulds when they were subjected to hammering, it was unlikely that they would break into pieces. Hence, the most likely failure criterion adopted was the appearance of at least one visible crack on the concrete surface. Since the tensile strength of the hammered prisms had not been measured, it was not known whether the vibration applied had any effect on the tensile strength. Nevertheless, Esteves found that although there is a period of greater susceptibility to vibration cracking between 10 and 20 hours after casting, the threshold vibration for cracks to appear during this period is as high as 150 mm/s.

In the present study, a more definite failure criterion as “complete cracking of a whole section of the concrete such that it breaks into at least two pieces or having more than 30% reduction in tensile or compressive strength” is adopted. Since shock vibrations have little effect on the compressive strength, whether there has been significant reduction in tensile strength actually governs. Moreover, as breakage of the concrete into pieces imply reduction of its tensile strength to zero, the failure criterion may be simplified as “having more than 30% reduction in tensile strength”. The reason for adopting this failure criterion is that in certain cases, even without any signs of observable cracks formed on the surface, the tensile strength of the vibrated concrete might have already been significantly affected due to disturbance to its mortar matrix or formation of internal cracks.

Referring back to Tables 5.1 - 5.24, it is seen that among the 288 concrete prisms which had been subjected to hammering at age $\leq$ 12 hours (Series 1 tests), 27 had failed (cracked or tensile strength reduced by more than 30%) but only one of them (Prism No.4 of G40/25PFA/20C tested at 12 hours) had cracked. At ages of 10 hours or less, none of the hammered prisms had any observable cracks formed on them, although quite a number of them had their tensile strengths reduced by more than 30%. Hence, at early ages of concrete (up to 10 hours), the effects of shock vibration are not easily observable by naked eyes. When making the judgment of whether the shock vibration applied has caused adverse effects on the concrete, it is not reliable to just consider the occurrence of visible cracks on the surface because there is the possibility that the concrete has been adversely affected without any signs of cracking.

It is, therefore, important when comparing the vibration resistance results of different researchers to take into account the actual failure criteria used by them. The failure criterion adopted by Hulshizer and Desai is the most lenient. As a results, they produced very high vibration resistance results. The failure criterion adopted by Esteves is also on the lenient side as any possible effect on the tensile strength was not considered. These are the reasons why the vibration resistances obtained in the present study are relatively low compared to the results of Hulshizer and Desai at virtually all ages of concrete and also to the results of Esteves at ages of less than 12 hours.

7.2 Vibration Resistance During First Few Hours

The largest discrepancy between the vibration control limits being adopted by different engineers lies in the vibration control limit to be applied during the first few hours before the concrete sets.

Some researchers (Hulshizer and Desai 1984) suggested that blasting vibrations applied to concrete before it sets are just like additional vibration for compaction and therefore can only be beneficial rather than detrimental. Hence, they adopted a very high vibration control limit of 100 mm/s during the first 3 hours. As cited by Olofesson (1988), Ontario Hydro had used the same vibration control limit of 100 mm/s during the first 10 hours for concrete cured at 5°C and during the first 5 hours for concrete cured at 21°C.

Other researchers (Akins and Dixon 1979; Gamble and Simpson 1985), however, imposed severe restrictions during the entire period from 0 to 24 hours after casting. Akins and Dixon argued that although re-vibration of concrete at 2 - 4 hours after initial placement in such a way that the concrete is returned to a plastic condition may be beneficial, it is unlikely that the transient vibration due to blasting can return the concrete to a plastic condition and afford the advantages of re-vibration. Shock vibrations sufficient to disturb the concrete matrix but less than sufficient to return the concrete to plastic state can be harmful. Added with the considerations that little is known about the effects of vibration on green concrete and autogenous healing cannot be relied on, they imposed a vibration limit of 5.1 mm/s during the entire period of green concrete, i.e. the first 24 hours. On the other hand, Gamble and Simpson simply assumed that the vibration resistance of concrete is directly proportional to the compressive strength and as the strength of concrete is very low before it hardens, imposed vibration control limits which are even more restrictive than those recommended by Akins and Dixon during the first 12 hours.

With the large amount of test results produced in the present study, a settlement to this historical conjecture or dilemma can now be made. The test results indicate that the vibration resistance of concrete is lowest during the first few hours but would gradually increase with age until the concrete is fully hardened. Hulshizer and Desai's view that the concrete should be able to withstand very large vibration before it sets and it is only after the concrete has set that it becomes particularly susceptible to vibration damage is unfounded. The period that the concrete is particularly susceptible to vibration damage actually starts before the concrete sets. In any case, it is considered unwise to set a high vibration control limit for concrete before it sets. The changeover of permitted vibration intensity from a very high level while the concrete is still wet to a very low level after the concrete has set is too abrupt. Since the setting time (to be more exact, the stiffening time) of the concrete depends on the mix composition and is quite sensitive to the curing temperature, it is not easy to precisely determine the changeover time and as a result, there is the danger that the concrete might have entered the low vibration resistance stage earlier than expected.

Nevertheless, the measured vibration resistance of green concrete is generally substantially higher than that assumed by Akins and Dixon. Hence, Akins and Dixon were a bit overly conservative. The blasting operation would become very expensive if their vibration control limits are adopted. The vibration control limits proposed in the present study, which are substantially higher than those adopted by Akins and Dixon, will allow more power blasting near green concrete and hence faster and more economical construction.

Finally, the test results clearly show that the vibration resistance is neither directly proportional to the compressive strength nor to the tensile strength (the material parameters that really determine the vibration resistance will be discussed in the next section). Gamble and Simpson’s assumption that the vibration resistance is proportional to strength is incorrect. The very low vibration control limits proposed by them would virtually ban all kinds of blasting operation near green concrete.

7.3 Material Parameters Determining Vibration Resistance

For concrete at age less than 12 hours, the only material properties measured were the penetration resistance of the mortar fraction and the stiffening time. However, it is found that the vibration resistance is almost independent of the penetration resistance. Any possible dependence of the vibration resistance on the penetration resistance has been covered up by the large intrinsic variation of the vibration resistance. The stiffening time is also found to have little influence on the change of vibration resistance with time. Hence, no material property that would directly or indirectly influence the vibration resistance of concrete at age less than 12 hours has been identified. The only parameter which the vibration resistance is found to be closely related to is the age of the concrete. Regression analysis of the test results reveals that on average, the vibration resistance increases steadily and more or less linearly with the age of the concrete up to 12 hours.

For concrete with age more than 12 hours, the compressive strength, tensile strength, dynamic and static moduli, and longitudinal wave propagation velocity had been measured. Although theoretically the compressive strength should not have a direct influence on the vibration resistance, as most material properties including those that may affect the vibration resistance are either directly or indirectly related to the compressive strength, there should be an indirect relation between the vibration resistance and the compressive strength. Regression analysis of the test data indicates that the vibration resistance is approximately proportional to $(f_{cu})^{0.26}$. Since the most likely damage to concrete by shock vibration is cracking or reduction in tensile strength, the tensile strength is expected to have a direct influence on the vibration resistance. Correlation of the vibration resistance to the tensile strength reveals that the vibration resistance may be taken as proportional to $(f_t)^{0.29}$. The vibration resistance is, therefore, neither directly proportional to the compressive strength nor to the tensile strength, and it is thus believed that both the compressive strength and the tensile strength are not the material properties that would directly determine the vibration resistance.

On the other hand, regression analysis of the corresponding results of the vibration resistance and the longitudinal wave propagation velocity yields the relation that the vibration resistance is proportional to $(c)^{1.24}$. Although the vibration resistance is related to the wave propagation velocity by a power equation, the curve plotted in Fig.6.12 for this relation is almost a straight line. Hence, there is a certain linear relation between the vibration resistance and the wave propagation velocity. Actually, the ratio of the vibration resistance in terms of ppv to the wave propagation velocity is the dynamic tensile strain capacity of the concrete. Regression analysis of the dynamic tensile strain capacity results indicates that the variation of the dynamic tensile strain capacity with the compressive strength of the concrete is very small and the dynamic tensile strain capacity may therefore be taken to have a constant value. The ratio of the vibration resistance to the wave propagation velocity is thus more or less constant, or in other words, the vibration resistance is approximately proportional to the wave propagation velocity. In view of the above regression analysis results, it is postulated that the longitudinal wave propagation velocity and the dynamic tensile strain capacity are the major material parameters determining the vibration resistance of a concrete. Among these two parameters, the dynamic tensile strain capacity of the concrete is of particular importance because the concrete fails when the dynamic tensile strain capacity is exceeded by the dynamic tensile strain but the dynamic tensile strain produced by a shock wave needs not be dependent on the longitudinal wave propagation velocity of the concrete. In an unrestrained concrete structure, the dynamic tensile strain induced is equal to the ppv of the vibration divided by the wave propagation velocity of the concrete, but in a restrained concrete structure which is forced to deform together with the surrounding soil, the strain induced in the concrete will be affected by that induced in the soil which is equal to the ppv of the vibration divided by the wave propagation velocity of the soil. Further discussions on this topic are given in the next section.

7.4 Restrained and Unrestrained Structures

Restrained and unrestrained structures respond quite differently to blasting vibrations and should therefore be treated separately.

Restrained structures (mainly underground structures) are restrained to deform together with the surrounding soil and thus the strain produced in the structure is affected by soil-structure interaction. Let the wave propagation velocity in the concrete be denoted by c and that in the soil by c_s. If the concrete structure and the surrounding soil are free to deform independently, then the maximum strains ε_d and ε_s produced in the concrete and in the soil are given respectively by:

$$\varepsilon_d = \frac{v}{c} \qquad \ldots(7.1)$$

$$\varepsilon_s = \frac{v}{c_s} \qquad \ldots(7.2)$$

However, due to interaction between the structure and the soil, the strains produced in them are forced to be equal and the actual strain produced in the composite system will be somewhere between the two values given by the above two equations. It is not easy to evaluate the effects of the soil-structure interaction. If the structure is relatively small such that the surrounding soil has a much larger volume than the structure, then the structure will be forced to have its strain equal to that given by Eqn.7.2. On the other hand, if the structure is big, then the effects of the soil-structure interaction will be relatively small and the structure will have its strain close to that given by Eqn.7.1. Any real situation lies between these two extremes and a conservative estimate for the strain produced in the concrete structure may be taken as the larger of the two values given by Eqns. 7.1 and 7.2, as depicted below:

$$\begin{aligned} \varepsilon_d &= \max.\left(\frac{v}{c}, \frac{v}{c_s}\right) \\ &= \frac{v}{\min.(c, c_s)} \end{aligned} \qquad \ldots(7.3)$$

When $c_s < c$, the strain produced in the concrete structure will be determined by the value of c_s, or in other words, the ground strain induced by the blasting vibration may cause failure of the concrete structure. When $c_s > c$, the strain produced in the concrete structure will be determined by the value of c, or in other words, the soil-structure interaction will not have any detrimental effect on the structure.

Unrestrained structures (mainly aboveground structures) are unrestrained by any nearby soil and are therefore free to deform and vibrate. For an unrestrained structure, the strain induced by the propagation of a shock wave through it is governed solely by Eqn.7.1 and the ground strain induced by the shock wave will not affect the structure. However, since the structure is free to vibrate, the shock induced structural vibration needs to be considered as the structure will shake as if it is subjected to an earthquake. The shaking response of the structure to the blasting vibration depends on many factors including the natural frequency and damping of the structure, and the intensity, duration and frequency spectrum of the ground motion. Since the ground motions due to blasting are similar in nature to those due to earthquakes, the dynamic response of a structure subjected to ground shaking due to blasting may be analyzed as in the seismic case.

The vibration resistance of concrete obtained in the present study and the vibration control limits derived therefrom are applicable only to unrestrained structures which are bulky or massive such that they are unlikely to have any dynamic response to the blasting vibration apart from the propagation of shock wave through them. When the vibration control limits are applied to restrained structures, the effects of the ground strain induced in the surrounding soil need also to be considered. In order to avoid possible damages due to large ground strain, the ground strain should be limited to within the safe dynamic tensile strain capacity of the concrete. Hence, for restrained structures, the vibration control should be exercised such that both the following conditions are complied with:

(a) the ppv of the blasting vibration must not exceed the safe ppv limit of the concrete;

(b) the ground strain, evaluated as the ratio of the ppv of the blasting vibration to the wave propagation velocity of the surrounding soil, must not exceed the safe dynamic tensile strain capacity of the concrete.

For hardened concrete, i.e. concrete at age more than 12 hours, the condition (a) would be more critical when $c < c_s$ while the condition (b) would be more critical when $c > c_s$. For concrete at age less than 12 hours, the wave propagation velocity had not been measured but judging from its low stiffness, it may be assumed that $c < c_s$ and thus the condition (a) would govern. In fact, the dynamic tensile strain capacity of concrete at such early age had never been measured and the condition (b) cannot be directly applied. Nevertheless, for completeness, it is suggested to keep the condition (b) for concrete at age less than 12 hours with the safe dynamic tensile strain capacity of the concrete taken as that of hardened concrete which has a constant value of 32 micro-strain. It is considered a good practice to always impose condition (b) so that the ground strain would not cause damage to any nearby unknown buried concrete structure. Condition (a) is a lot more restrictive than condition (b) when applied to concrete at age less than 12 hours and thus the enforcement of condition (b) would not change the vibration control limits for concrete at such age.

When the vibration control limits are applied to unrestrained structures which may have significant dynamic response to the ground motion caused by blasting, the stresses induced by the dynamic response need also to be considered. In such case, detailed dynamic analysis is required and the vibration control should be exercised such that both the following conditions are complied with:

(a) the ppv of the blasting vibration must not exceed the safe ppv limit of the concrete;
(b) the dynamic response of the structure to the ground motion due to blasting must not be excessive as to cause damage to the structure.

In addition to the above, it should be advisable to always monitor the actual vibration response of the structure and carry out regular inspections to see if any damage has been caused. If the structure is already occupied, than the possibility of causing nuisance to the tenants and/or disturbance to any sensitive equipment housed inside should also be considered. Avoidance of nuisance and disturbance to people and sensitive equipment may impose a very severe restriction to the blasting operation.

8. CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

8.1 Conclusions

An experimental programme of subjecting typical concrete used in Hong Kong to high intensity shock vibrations to evaluate their vibration resistances at different ages and investigate the short/long term effects of shock vibration on concrete is completed. Using the test results so gathered, a theoretical study to correlate the vibration resistance of a concrete to its mechanical properties and develop a method of setting up appropriate vibration control criteria based on the known properties of concrete has been carried out. The conclusions of the experimental investigation and the theoretical study are presented in the followings.

Method of testing:

(1) A method of testing fresh/hardened concrete for their resistance against shock vibration by subjecting prisms cast of the concrete to hammer blows has been developed. The hammering operation is made automatic and synchronized with the data logging process by the use of a computer controlled hammer release device. A piezoelectric high-g accelerometer fixed at mid-length of the prism is used to pick up the acceleration response of the concrete. Analogue output from the conditioning amplifier of the accelerometer is digitized by a computer controlled A/D converter and then stored in a hard disk for subsequent digital signal processing. The velocity response of the concrete is obtained by numerical integration of the acceleration data and then a trend removal process is applied to extract the pure wave component of the velocity response from which the peak particle velocity of the shock vibration applied is determined. The short term effects of the shock vibration applied are studied by observing the occurrence of cracks on the surface of the concrete prism after hammering while the long term effects are evaluated by continuing to cure the concrete prism until the 28-day and then testing the prism for its tensile and compressive strengths. The tensile strength of the concrete is measured by a direct tension test method instead of the easier three-point beam flexural test method in order to eliminate the unpredictable effects of the random positioning of the transverse cracks formed by hammering on the test results. The broken pieces of the concrete prism after the tension test are then used to determine the compressive strength of the concrete. This hammering test method has been shown to be a practical and efficient method of evaluating the short/long term effects of high-intensity shock vibration on concrete. It does not require the use of explosives and can therefore be safely carried out in a laboratory. Unlike previous test methods developed by others which do not bother to measure the tensile strength of concrete, the present method gives the effects of the shock vibration on the tensile strength as well as the compressive strength. In fact, it is believed this is the first time the effects of shock vibration on the tensile strength of concrete are studied in detail.

Material properties:

(2) As part of the experimental programme, a large amount of test results on the material properties of typical concrete used in Hong Kong has been produced. The concretes studied are of grades 20 - 40, without PFA or with 25% PFA, and cured at 20^{o}C or

30°C while the material properties tested are stiffening time, cube compressive strength, split cylinder tensile strength, longitudinal wave propagation velocity, dynamic modulus and static modulus. The stiffening time results indicate that the stiffening time increases with the concrete grade and PFA content but would become significantly shorter at higher curing temperature. Depending on the mix composition and curing temperature, the stiffening time to reach a penetration resistance of 0.5 MPa (corresponding to the extreme limit for placing concrete) ranges from 2.7 to 4.8 hours while the stiffening time to reach a penetration resistance of 3.5 MPa (corresponding to the time limit for avoidance of cold joints) ranges from 4.0 to 6.8 hours. On the other hand, statistical analysis of the test results gives the following inter-relation between the cube compressive strength f_{cu}, split cylinder tensile strength f_t, longitudinal wave propagation velocity c, dynamic modulus E_d, static modulus E_d and static tensile strain capacity ε_c':

$$f_t = 0.24 \cdot (f_{cu})^{2/3}$$
$$E_c = 0.68 \cdot E_d$$
$$E_c = 4.17 \cdot (f_{cu})^{1/2}$$
$$E_d = 6.13 \cdot (f_{cu})^{1/2}$$
$$c = 2041 \cdot (f_{cu})^{0.2}$$
$$\varepsilon_c' = 24.1 \cdot (f_{cu})^{0.44}$$

in which the unit(s) for compressive and tensile strengths are MPa, for dynamic and static moduli are GPa, for wave propagation velocity is m/s, and for tensile strain capacity is micro-strain.

Effects of shock vibration on concrete:

(3) The effects of shock vibration on concrete have been studied in details by the hammering test method developed herein. It is seen from the test results that even at intensities high enough to break the concrete into pieces, shock vibrations have in general little effect on the compressive strength of the concrete. Depending on the intensity of the shock vibration applied, the concrete may have its tensile strength significantly reduced without any visible cracks formed on the surface, become partially cracked without breaking, or become completely cracked such that it is broken into pieces. However, the results on cracking and tensile strength of the hammered concrete prisms reveal that the effects of shock vibration on concrete are rather erratic. While one concrete sample may be broken into pieces when subjected to shock vibration of a certain intensity, another sample cast from the same batch of concrete and cured side by side may be able to withstand shock vibration of a much higher intensity without cracking or any deterioration in strength.

(4) It is also found from the test results of the cracked concrete prisms that even with the cracks kept closed and further water curing applied until the age of 28-day, there would not be any significant self-healing that could be relied on. Thus, it may be concluded that once the concrete is cracked, the lost in tensile strength is permanent and there is little hope that any significant tensile strength could be recovered by means of self-healing despite continuous water curing. It is, therefore, important to

avoid any damage to the concrete due to blasting and not to rely on a false hope of self-healing.

Vibration resistance of concrete:

(5) Since it is not possible to continuously adjust the shock vibration intensity until the concrete fails, the vibration resistance of concrete cannot be directly measured. Nevertheless, for this particular study in which shock vibrations of different intensity were applied in such a way that some of the test prisms were cracked while the others remained un-cracked, the vibration resistances of the concretes tested may be indirectly determined from the lowest intensity of shock vibration applied that had caused failure of the concrete (ppv1) and the highest intensity of shock vibration applied that had not caused any failure of the concrete (ppv2). The failure criterion adopted for determining the vibration resistance is "complete cracking or having more than 30% reduction in tensile or compressive strength". However, due to the erratic nature of the effects of shock vibration on concrete, ppv1 is not always greater than ppv2. From each pair of ppv1 and ppv2 values, a lower bound estimate of the vibration resistance (ppv3) may be obtained as min.(ppv1,ppv2). The ppv3 results of the 12 batches of concrete tested are listed in Tables 5.25 - 5.27.

(6) No obvious effects of using PFA up to 25% and curing temperature within the range of 20°C - 30°C on the vibration resistance of concrete can be identified from the ppv3 results. Any probable effects of PFA and curing temperature within the ranges studied have been over-shadowed by the large intrinsic variation of the vibration resistance of concrete. In view of this situation, it is suggested that when setting vibration control limits to avoid vibration damages to concrete, a single set of vibration limits that applies to concrete with or without PFA and cured within the temperature range of 20°C - 30°C should be used; it does not seem to be worthwhile to have more refined vibration control limits that take these two factors into account.

(7) Comparison of the lower bound estimates of the vibration resistances of the different grades of concrete in Figs. 5.1 and 5.2 reveals that no effect of concrete grade can be meaningfully differentiated from the test results, although it is believed that the concrete grade should have a bearing on the vibration resistance. To overcome this difficulty, a lower bound for all the concrete mixes tested, i.e. concretes of grade 20 - 40, with 0 - 25 % PFA and cured at 20°C - 30°C, is worked out and used to establish a single set of vibration control limits for them. The lower bound values of vibration resistance are:

Age of concrete	Lower bound of vibration resistance (mm/s)
2 hrs.	46
4 hrs.	52
6 hrs.	130
8 hrs.	180
10 hrs.	170
12 hrs.	250
18 hrs.	180
24 hrs.	350
3 days	500
7 days	690
28 days	750

Correlation of vibration resistance to other mechanical properties:

(8) For theoretical study, mean value estimates of the vibration resistance (ppv4), which are taken as equal to (ppv1+ppv2)/2 and should be more accurate than the lower bound estimates, are used. For concrete of age less than 12 hours, it is found that the correlation between ppv4 and the penetration resistance of the mortar fraction of the concrete is rather weak and that the vibration resistance increases only marginally with the penetration resistance. For concrete of age more than 12 hours, correlation of ppv4 to the cube compressive strength f_{cu} and split cylinder tensile strength f_t yields the following equations:

$$\text{ppv4} = 585 \cdot (f_{cu})^{0.26}$$
$$\text{ppv4} = 1098 \cdot (f_t)^{0.29}$$

in which the unit(s) for ppv4 is mm/s, and for compressive and tensile strengths are MPa. From these two equations, it can be seen that the vibration resistance is neither proportional to the compressive strength nor to the tensile strength. The believe of many engineers that the vibration resistance is proportional to the strength of the concrete is unfounded. Correlation of ppv4 to the longitudinal wave propagation velocity c yields:

$$\text{ppv4} = 0.047 \cdot (c)^{1.24}$$

in which the unit for wave propagation velocity is m/s. Correlation of the dynamic tensile strain capacity ε_d', which is actually just the ratio of the vibration resistance in terms of ppv to the longitudinal wave propagation velocity, gives the following equation:

$$\varepsilon_d' = 321 \cdot (f_{cu})^{0.015}$$

where the unit for tensile strain capacity is micro-strain. It is seen from this equation that the dynamic tensile strain capacity changes only slightly with the compressive strength of the concrete and may therefore be taken to have a constant value. Hence, the ratio of the vibration resistance to the wave propagation velocity is more or less constant. In other words, the vibration resistance is approximately proportional to the wave propagation velocity. It is thus postulated that the longitudinal wave propagation velocity and the dynamic tensile strain capacity are the major material parameters determining the vibration resistance of a concrete.

(9) However, the above mean value estimates are not suitable for direct use to establish vibration control limits. As there may be large intrinsic variation in vibration resistance, the actual vibration resistance of the concrete could be significantly lower than the values predicted by the above equations. What is more important is that there is a very high probability of about 50% of having the actual vibration resistance lower than the mean value. Like the use of characteristic strength in structural design, it is proposed to use a characteristic value of the vibration resistance to cater for the large intrinsic variation of vibration resistance. The characteristic vibration resistance (ppvc) is defined as the vibration level at which no more than 5% of the concrete is expected to fail. Assuming that ppvc varies with the other mechanical properties of concrete in the same way as for ppv4 and applying curve fitting to the test results, the following equations are obtained:

$$\text{ppvc} = 290 \cdot (f_{cu})^{0.26}$$
$$\text{ppvc} = 527 \cdot (f_t)^{0.29}$$
$$\text{ppvc} = 0.023 \cdot (c)^{1.24}$$

The corresponding characteristic value of dynamic tensile strain capacity so derived is approximately equal to 160 micro-strain.

(10) The measured vibration resistance is generally higher than the theoretical prediction given by the following equation:

$$v' = \frac{c}{E_d} f_{td}$$

where v' is the theoretical vibration resistance in terms of ppv and f_{td} is the dynamic tensile strength which may be taken as 1.5 times the static tensile strength. In fact, the measured vibration resistance is often two to three times higher than the above theoretical value. Nevertheless, since some of the test results are just marginally higher than v', the theoretical value given by this equation may be taken as a lower bound to the vibration resistance.

<u>Recommended vibration control limits</u>:

(11) For the purpose of establishing vibration control limits, it is suggested to adopt a relatively large factor of safety of 5 to allow for the large intrinsic variation of the vibration resistance. A set of vibration control limits that applies to concretes of grade 20 - 40, with 0 - 25 % PFA and cured at 20°C - 30°C may be established from the lower bound values of vibration resistance given in Conclusion (7) by dividing the lower bound values by a factor of safety of 5 and adjusting the resulting values slightly so that they map into a smooth and monotonic function of the age of concrete, as presented below:

Age of concrete	Safe ppv limit (mm/s)
≤ 4 hrs.	10
6 - 8 hrs.	20
10 - 12 hrs.	30
18 hrs.	40
24 hrs.	70
3 days	100
7 days	125
≥ 28 days	150

(12) Alternatively, the vibration control limits may also be set up based on the known properties of concrete using the equations for characteristic vibration resistance given in Conclusion (9). Using the equation correlating the characteristic vibration resistance to the cube compressive strength and dividing the ppvc values so obtained by a factor of safety of 5, the following vibration control limits are established:

Cube compressive strength (MPa)	Safe ppv limit (mm/s)
3.0	77
5.0	88
10.0	106
20.0	126
30.0	140
40.0	151
50.0	160

Using the equation correlating the characteristic vibration resistance to the split cylinder tensile strength and dividing the ppvc values so obtained by a factor of safety of 5, the following vibration control limits are established:

Split cylinder tensile strength (MPa)	Safe ppv limit (mm/s)
0.1	54
0.3	74
0.5	86
1.0	105
2.0	129
3.0	145
4.0	158

(13) Since the vibration resistance is roughly proportional to the longitudinal wave propagation velocity and the characteristic value of the dynamic tensile strain capacity (the dynamic tensile strain capacity is just the ratio of vibration resistance to longitudinal wave propagation velocity) is approximately equal to a constant value of 160 micro-strain, the vibration control limits may also be established by first dividing the characteristic dynamic tensile strain capacity by a factor of safety of 5 to obtain the safe dynamic tensile strain capacity as 32 micro-strain and then multiplying by the longitudinal wave propagation velocity to produce the vibration control limits as follows:

safe ppv limit
= 32 micro-strain $\times$ longitudinal wave propagation velocity

(14) The above vibration control limits are applicable only to unrestrained structures which are unlikely to have significant dynamic response to the blasting vibration apart from the propagation of shock wave through them. When the vibration control limits are applied to restrained structures, the effects of the ground strain induced in the surrounding soil need also to be considered. In order to avoid damages due to large ground strain, the ground strain should be limited to within the safe dynamic tensile strain capacity of the concrete. Hence, for restrained concrete structures, the vibration control should be exercised such that both the following conditions are complied with:

(a) the ppv of the blasting vibration must not exceed the safe ppv limit of the concrete;
(b) the ground strain, evaluated as the ratio of the ppv of the blasting vibration to the wave propagation velocity of the surrounding soil, must not exceed 32 micro-strain.

(15) When the vibration control limits are applied to unrestrained concrete structures which may have significant dynamic response to the ground motion caused by blasting, the stresses induced by the dynamic response need also to be considered. In such case, the vibration control should be exercised such that both the following conditions are complied with:

(a) the ppv of the blasting vibration must not exceed the safe ppv limit of the concrete;
(b) the dynamic response of the structure to the ground motion due to blasting must not be excessive as to cause damage to the structure.

8.2 Recommendations for Further Study

The experimental investigation carried out in this project has been confined to laboratory testing. So far, no field testing has been performed to measure the resistance of concrete to real blasting vibrations. It is, therefore, recommended to cast concrete on typical construction sites where blasting is going to take place at different times before the blasting and at different distances from the blast to investigate the short/long term effects of blasting vibrations on concrete at various ages. However, it should be noted that especially for green concrete, the effects of the blasting vibrations may not be easily observable by naked eyes and thus the vibrated concrete, after being continuously cured until a certain age, should be taken out for testing of their tensile and compressive strengths. Since soil-structure interaction can affect the response of the concrete, sites with different soil conditions should be selected for the study. As the moulds may have some effects on the interaction between the concrete and the soil, some of the concrete should be cast without moulds, i.e. directly against the soil, while some other concrete cast inside moulds so that any possible difference due to the use of moulds may be investigated.

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