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REVIEW OF THE NEW ZEALAND STANDARD FOR CONCRETE STRUCTURES (NZS 3101) FOR HIGH STRENGTH AND LIGHTWEIGHT CONCRETE EXPOSED TO FIRE

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Abstract

This report describes the thermal properties, strength, and elastic modulus of lightweight, normal, and high strength concrete at elevated temperatures.

Section 6 (Design for Fire Resistance) of the New Zealand standard for concrete structure (NZS3101, 1995) provides recommended values and minimum requirements for concrete at elevated temperatures. These values will be reviewed with respect to overseas standards and experimental data to find their applicability to lightweight and high strength concretes.

A series of tests were performed on 1m x 1m lightweight and high strength concrete specimens to determine their insulation fire resistance. The specimens were produced in three thicknesses; 60, 130, and 175mm. This follows the method of earlier tests by Wade et al. (1991) and Wade (1992) on New Zealand aggregate concretes.

It was determined that the strength reduction curve given by NZS3101 over-predicts the strength of high strength concrete at elevated temperatures, though the values for elastic modulus and insulation fire resistance can be applied to high strength concrete.

The insulation fire resistance, strength, and elastic modulus values given by NZS3101 were found to also apply to lightweight concrete.

The report recommends a change to the elastic modulus curve given in NZS3101. The purpose of this is to give consistency between the strength and elastic modulus curves, which do not currently reach zero at the same temperature.

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1. Introduction

General

Standards New Zealand provides guidance for the design of concrete elements at elevated temperatures within NZS3101. At present NZS3101 describes minimum requirements and recommended values for several concrete properties that are dependent on the concrete type. These are;

- thickness for insulation fire resistance
- strength reduction at elevated temperatures
- elastic modulus reduction at elevated temperatures

Currently the values given in NZS3101 (1995) do not directly allow for high strength concrete, and only provides limited guidance for lightweight concrete

This report reviews the contents of NZS3101 with respect to its consistency to other national standards, existing experimental data, and the results of testing done for this report to determine its applicability to lightweight and high strength concretes at elevated temperatures. This will compliment a project by Allington (1998) that is reviewing the structural requirements of NZS3101 for lightweight concrete.

The standards of Australia, Britain, U.S.A, Canada, and Europe will be used for comparison with NZS3101.

There has already been much experimental research into the performance of lightweight, ordinary, and high strength concrete at elevated temperatures for overseas concretes, with only limited experimental research for New Zealand concretes. Wade et al. (1991) conducted initial tests on the insulation fire resistance of common New Zealand concretes. These were followed by additional tests (Wade, 1992) that provided the experimental results to base the insulation fire resistance requirements for concrete members in Section 6 on New Zealand specific data. This is the only part of the Section 6 that is based on New Zealand specific data.

Many researchers have reported on the properties of the different types of concrete at elevated temperatures. General reviews of the finding of these researchers are described by Neville (1997), Schneider (1985), and Bazant and Kaplan (1996). Purkiss (1996), Buchanan (1999), Fleischmann (1995), and Harmathy (1993) also provide general reviews relating to the design of concrete members at elevated temperatures. In relation to high strength concrete recent papers have been produced by Tomasson (1998) and Phan (1996).

Scope of Report

The report consists of 5 chapters.

Chapter 2 reviews Section 6 of the New Zealand standard NZS3101 and provides a comparison with the national standards of Australia, Britain, Europe, U.S.A. and Canada. The chapter then goes on to review experimental data relating to the thermal properties, strength, and elastic modulus of lightweight, ordinary, and high strength concrete at elevated temperatures. These are also compared with the values in Section 6 of NZS3101. The chapter finishes with a review of spalling and the increased susceptibility of high strength concrete to explosive spalling.

Chapter 3 describes the experiments that were performed on lightweight, high strength and ribbed floor slabs to determine their insulation fire resistance.

Chapter 4 reviews the experimental results in relation to NZS3101 and the experimental data discussed in chapter 2.

Chapter 5 summarises the key findings of the report and provides recommendations for further actions.

2. Literature Review

2.1. Introduction

This literature review discusses the recommended values and curves of NZS3101 for the design of concrete elements at elevated temperatures, and provides a comparison with existing overseas standards and experimental results. The general properties of concrete at elevated temperatures and the susceptibility of concrete to destructive spalling are also covered.

2.2. Review of Standards for Design of Fire Resistance

The standards of many countries recommend values when designing the fire resistance of concrete members. This section will review the recommended values from the New Zealand standard NZS3101 (1995) for;

- minimum thickness to provide insulation fire resistance,
- strength at elevated temperatures,
- elastic modulus at elevated temperatures,

These will then be compared with the recommended values from the standards of;

Australia (AS3600, 1994)

Europe (ENV 1992-1-2, 1995)

Britain (BS8110, 1985)

North America (UBC, 1997)

Canada (NBCC, 1995)

New Zealand Standard 3101 Part 1: 1995 Chapter 6

Insulation

The minimum effective thickness of a slab or wall required by NZS3101 (1995) to provide for insulation fire resistance ratings are shown in Table 2.1. These values apply to load bearing or non-load bearing walls, and plain or ribbed slabs, though the minimum width of the ribs is specified for each fire resistance rating. (Table 6.3, NZS3101 1995)

The thicknesses required by the New Zealand standard varies depending on the aggregate used in the concrete, acknowledging their differing thermal properties. The values given by NZS 3101 (1995) are based on extensive tests with New Zealand concretes done by Wade (1992) at BRANZ.

Table 2.1: NZS3101 minimum effective slab and wall thicknesses required for insulation fire resistance ratings (Table 6.1 NZS3101, 1995)

Fire resistance rating (minutes)	Effective thickness (mm) for different aggregate type		
	Type A aggregate	Type B aggregate	Type C aggregate
30	50	45	40
60	75	70	55
90	95	90	70
120	110	105	80
150	140	135	105
180	165	160	120
Aggregate types A – quartz, greywacke, basalt & all others not listed B – dicite, phonolite, andesite, rhyolite, limestone C – pumice & selected lightweight aggregates			

The New Zealand standard does place a limit on the aspect ratio (height of wall/thickness of wall) of a wall depending on the load ratio, and degree of lateral and rotational restraint.

If $N^* \leq 0.03f'_cA_g$ then h_{we}/t_w must be not greater than 50 or,

$N^* > 0.03f'_cA_g$ then h_{we}/t_w must be not be greater than 20

Where h_{we} shall be taken as,

i) 1.0 h_{wu} if neither support is rotationally restrained;

ii) 0.85 h_{wu} if one support is rotationally restrained; or

iii) 0.70 h_{wu} if both supports are rotationally restrained.

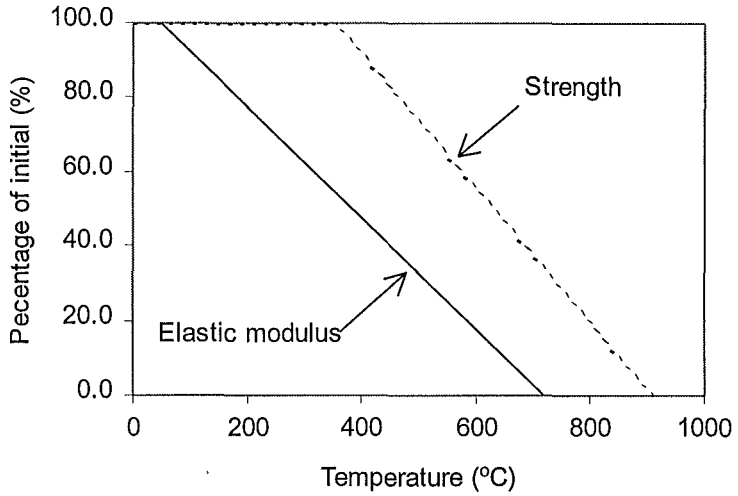


Figure 2.1: Variation of strength and elastic modulus of concrete at elevated temperature (NZS3101 1995)

As the graph shows the elastic modulus and strength curves do not reach zero at the same temperature. This gives erroneous values for design calculations at temperatures at the upper end of the curves, as a material cannot have strength without also having stiffness.

Comparison of Standards

The standards of Australia, Britain, Canada, U.S.A, New Zealand, and Europe vary in their specification of minimum member thickness for insulation fire protection depending on the scale of external loading, aggregate type, aspect ratio and amount of reinforcing steel contained in the member (steel ratio, p).

Insulation

Figures 2.2 to 2.4 show the comparison of minimum thickness requirements for load bearing walls from the different standards for siliceous, calcareous and lightweight concretes. The graphs show that overall the New Zealand standard requires the thinnest slab thickness to provide insulation fire protection.

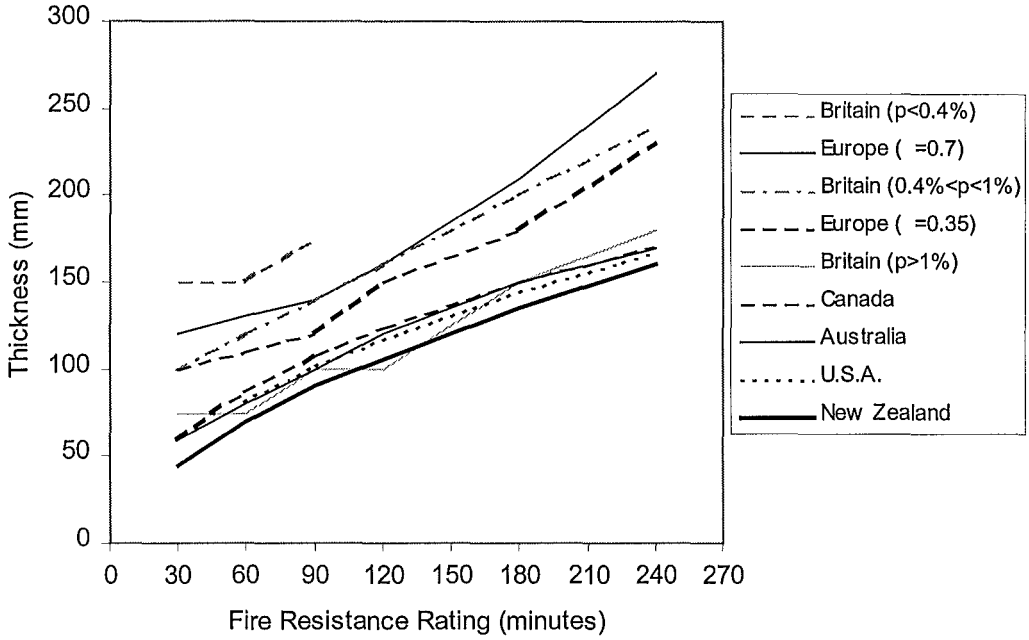


Figure 2.2: Comparison of minimum thickness requirements for siliceous load bearing walls.

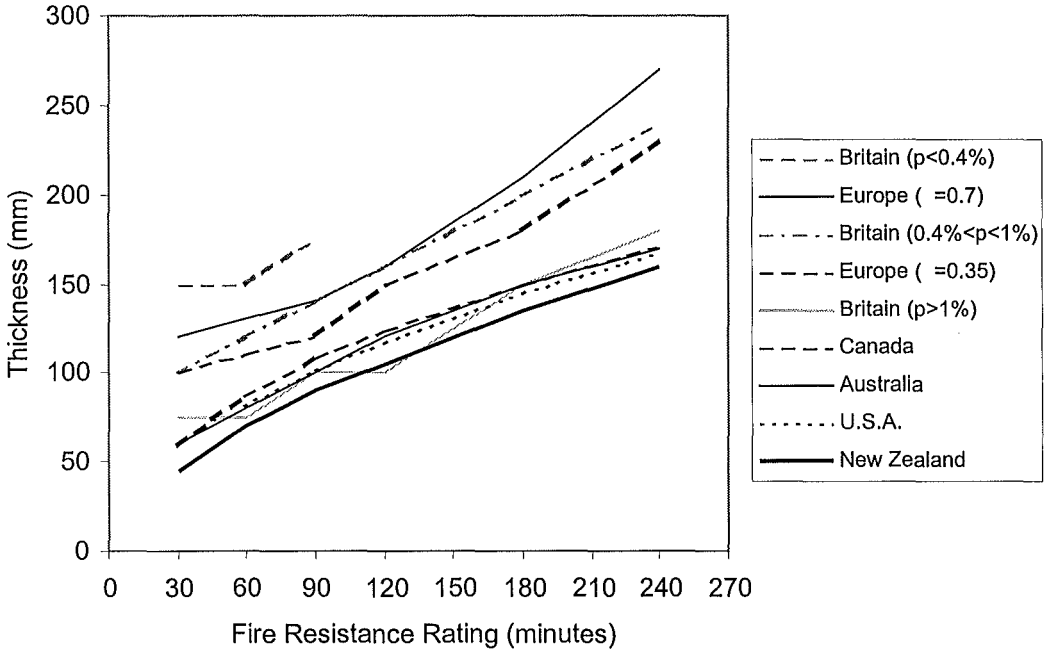


Figure 2.3: Comparison of minimum thickness requirements for calcareous load bearing walls.

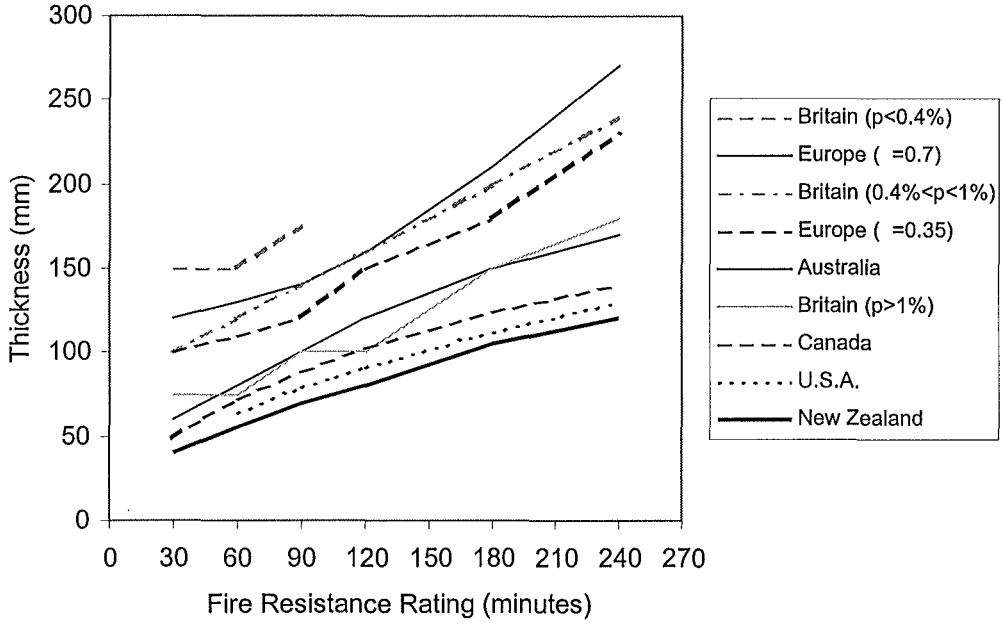


Figure 2.4: Comparison of minimum thickness requirements for lightweight load bearing walls.

Similar graphs for non-load bearing, plain floor slabs, and ribbed floor slabs are shown in Appendix A. They also show NZS3101 to consistently require the thinnest slab thicknesses to provide isolation fire resistance. This was also found to be more pronounced for lightweight concretes.

For the comparison of the minimum thickness requirements for a ribbed slab it must be noted that the Canadian and American standards define the equivalent thickness differently to the other standards. The two definitions of effective thickness are shown in Figure 4.6.

Strength

Figure 2.5 shows the comparison of the strength of concrete at elevated temperatures recommended by the different standards. The Canadian and American standards do not provide values for the strength of concrete at elevated temperatures.

This figure shows that the values for concrete strength at elevated temperatures from Britain (lightweight), Australia, and New Zealand coincide with each other. The

values for the strength of concrete at elevated temperature from the Eurocode begin reducing at a lower temperature, making it the more conservative of the standards, while the values for Britain (dense) is markedly higher than the other standards.

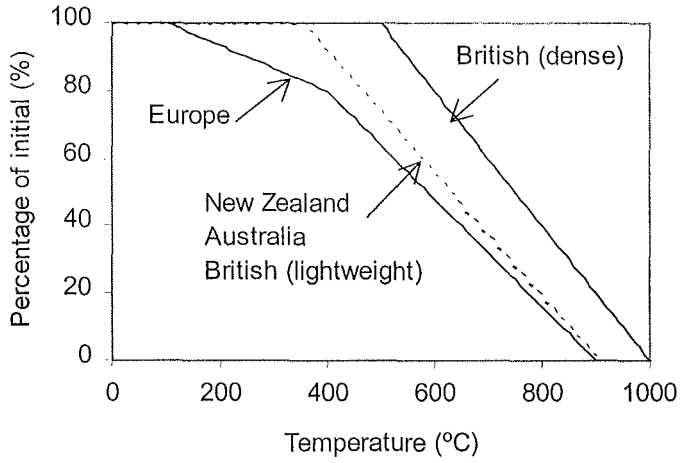


Figure 2.5 Concrete strength at elevated temperatures

Elastic Modulus

Only the Australian and New Zealand standards provide values for the elastic modulus of concrete at elevated temperatures. Both standards recommend the same values for the elastic modulus of concrete at elevated temperatures. (see review of NZS3101 Figure 2.1)

2.3. *Thermal Properties*

The thermal properties of the various types of concrete are important for evaluation of the performance of non-uniform concrete sections, such as hollow core or ribbed flooring, exposed to fire without the need for full scale tests. Specific heat, density and conductivity influence the rate of temperature change within the concrete section over a period of time. Thermal diffusivity is a relationship that shows the effect these properties have on the development of temperature gradients. The thermal diffusivity represents the rate at which temperature changes can occur within a mass (Neville 1997). The relationship is

$$\delta = \frac{k}{\rho c}$$

where δ= thermal diffusivity (m^2 / s)

k = conductivity (J / s m K)

ρ = density (kg / m^3)

c = specific heat (J / kg K)

The larger the value of thermal diffusivity of a mass the faster the temperature changes will occur. The value of thermal diffusivity is dependent on the aggregate type, moisture content, degree of hydration of the cement paste, and exposure to drying (Neville 1997). These factors effect the conductivity, density, and specific heat of the concrete.

Thermal Conductivity

Thermal conductivity is the most variable of the properties in the thermal diffusivity equation. Thermal conductivity is the measure of the ability of a material to transfer heat, and is described as the ratio of heat flux to temperature gradient. The typical values of thermal conductivity for some concrete types are listed below in Table 2.2.

Table 2.2: Typical values of thermal conductivity for different concrete types (selected from Scanlon and McDonald, 1994)

Aggregate type	Wet Density of Concrete Kg/m^3	Conductivity W/mK
Quartzite	2440	3.5
Granite	2420	2.6
Basalt	2520	1.2
Limestone	2450	3.2

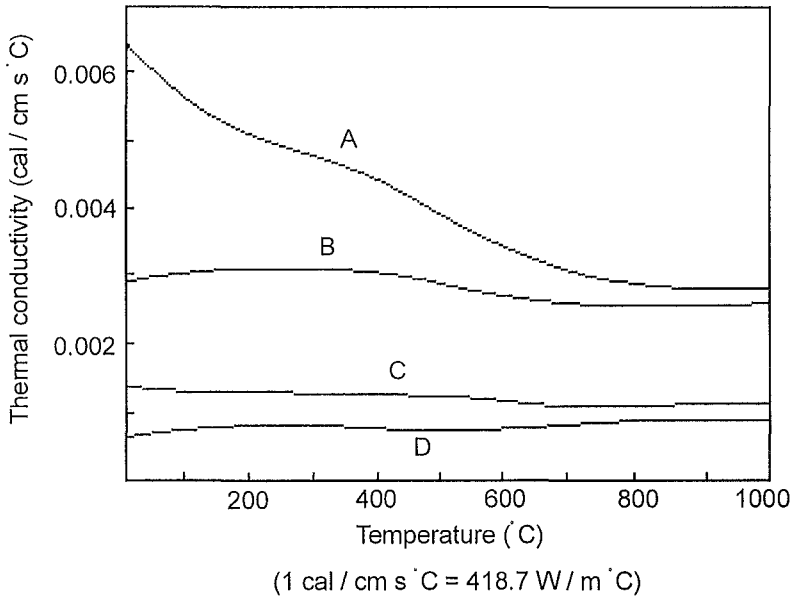
The large variations of the values for the thermal conductivity of the different concretes shown in Table 2.2 highlights the considerable influence the aggregate type has on the thermal conductivity.

The conductivity of concrete is also highly affected by its moisture content, as water has a higher conductivity than air. Though the effects of a variation in the moisture content are not as large as those caused by the aggregate type for normal weight concrete, in lightweight concrete the affects can be quite pronounced. A 10% increase

of the moisture content results in the increase of conductivity of lightweight concrete by about one half (Neville, 1997). Loudon and Stacey (1966) have recommended ranges of values for the conductivity for several lightweight concrete types and densities at different moisture contents.

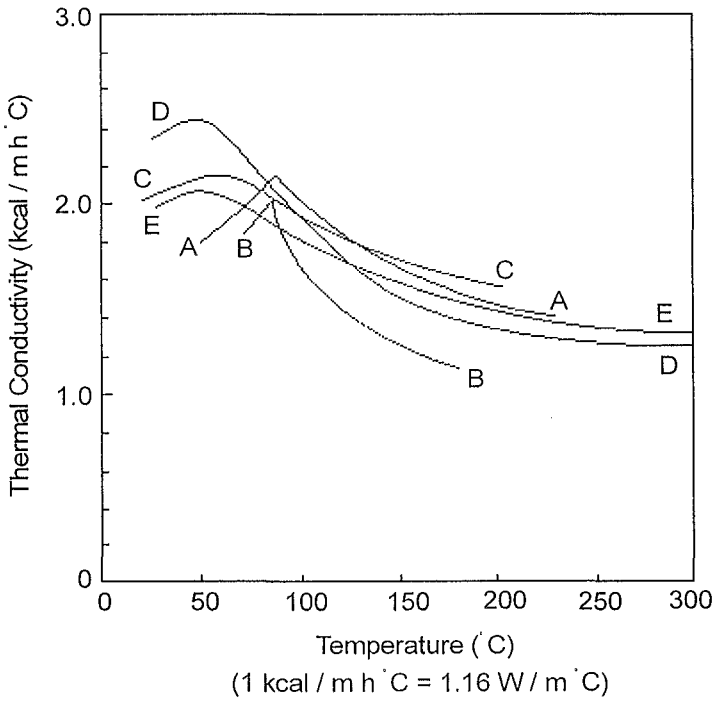
The amount of entrained air within a mix does have a notable effect on the conductivity of concrete due to the low conductivity of air. As the amount of entrained air increases the conductivity is reduced, particularly for air contents over 10 percent (ACI 216, 1981). This is more prevalent with lightweight concretes that use a variation of the entrained air in a mixture to alter the density.

At elevated temperatures the thermal conductivity of concrete specimens that are oven dried declines (Harmathy, 1970), as shown in Figure 2.6. This is not the case initially for non-oven dried samples which show a rise in their conductivity, shown in Figure 2.7. This increase is considered to be attributed to the increase of the thermal conductivity of water (0.60W/m^{o}C at 25°C to 0.68W/m^{o}C at 130°C) being larger than the decrease in that of the concrete. Beyond 100°C the reduced moisture content causes the decrease in the conductivity of the concrete to govern. More discussion on the thermal conductivity of high strength and ordinary concrete at elevated temperatures are given by Tomasson (1998), Schneider (1985), and Bazant and Kaplan (1996).



A: quartz aggregate
B: anorthosite aggregate
C: expanded shale aggregate A
D: expanded shale aggregate B

Figure 2.6: Thermal conductivity of oven dried concrete (Harmathy and Allen, 1973).



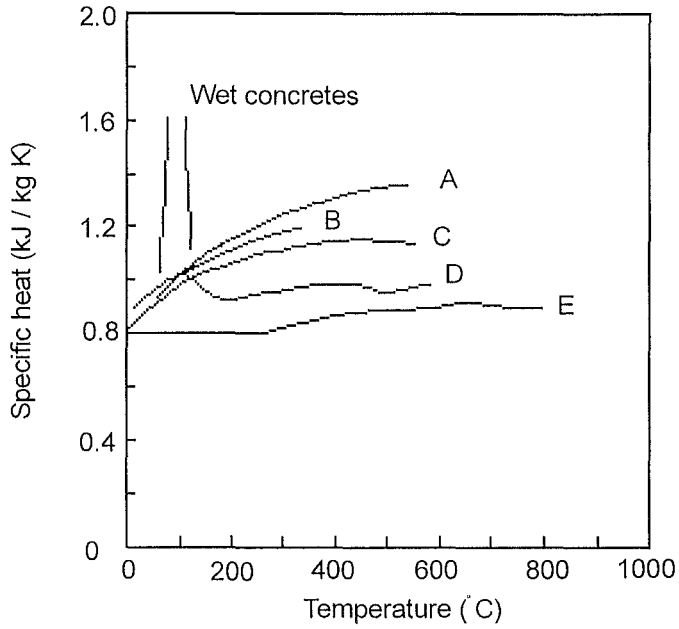
A: limestone aggregate (Crispino 1972)
B: byrates aggregate (Crispino 1972)
C: gravel agregate (Abe et al. 1972)
D: quartzite aggregate (Marechel 1972)
E: quartzite aggregate (Marechal 1972)

Figure 2.7: Thermal conductivity of non-oven dried concrete

Specific Heat

Specific heat is the measure of the heat capacity of concrete. The type of aggregate has only a small affect on the specific heat of concrete (Harmathy and Allen 1973), but it is greatly affected by the moisture content. This is due to the large difference between the values of specific heat of the concrete and water, 840 to 1170 J/kgK and 4187J/kgK respectively. This shows that a small change in the moisture content of the concrete causes a comparatively large change in the specific heat.

Figure 2.8 shows the variation of the specific heat at elevated temperatures. At temperatures between 90°C and 150°C the specific heat of the wet concrete increases dramatically. The peak can be two to three times the magnitude of the initial value (Blundell et al, 1976, Ohgishi, 1972). This is due to the large amount of energy being absorbed by the phase change of the liquid water in the concrete to steam. After the peak the values of specific heat continue to increase, though this is not as significant in the granite aggregate concrete. The specific heat of the lightweight aggregate concrete does not show the same gradual increase. The variations of the specific heat with temperature are attributed to chemical and physical reactions that take place within the concrete. These reactions are generally endothermic, removing energy from the environment. In limestone concrete Hildenbrand et al (1978) found that the specific heat increased from 1.65 kJ / kg K to 3.6 kJ / kg K at 1100°C depending on whether decarbonation was taken into account.



A: calcareous concrete (Harmathy 1970)
B: Gravel concrete (Colette et al 1978)
C: carbnate concrete (Colette 1978)
D: lightweight (Colette 1978)
E: granite concrete (Odeen 1972)

Figure 2.8: Specific heat of different concrete types.

The volumetric specific heat shown in Figure 2.9 gives a different perspective of the changes that occur at elevated temperatures. The figure shows a distinctive peak between 450 to 560°C. This peak is associated with the dehydration of the calcium hydroxide in the cement paste. Harmathy (1970) estimated the change of specific heat of idealised Portland cement paste, with a water cement ratio of 0.5, through theoretical considerations and experimental data. He considered the two main reactions taking place to be the dehydration of the tobermorite gel and calcium hydroxide. Using the results from differential thermal analysis with data on the enthalpy of tobermorite gel, calcium hydroxide, and their dehydrated products, the apparent specific heat for an idealised Portland cement paste at elevated temperatures was determined. This is shown in Figure 2.10.

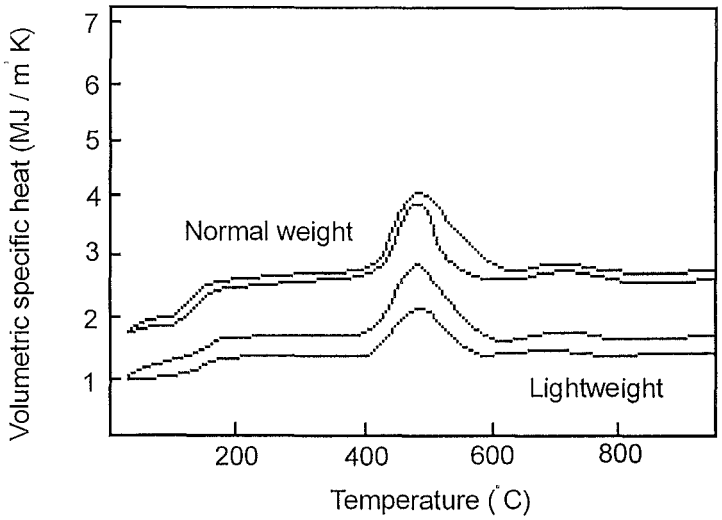


Figure 2.9: Volumetric specific heat of normal weight and lightweight concrete. (ACI 216, 1981)

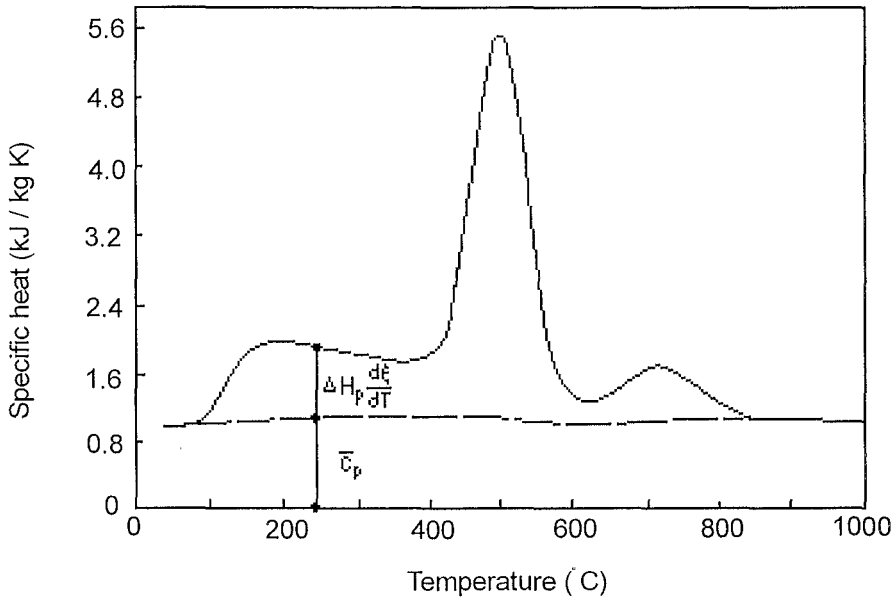


Figure 2.10: Apparent specific heat of idealised Portland cement paste with a water cement ratio of 0.5 at elevated temperatures. (Harmathy, 1970)

2.4. Concrete Strength at Elevated Temperatures

The strength of concrete at elevated temperature is usually expressed as a percentage of its strength at normal temperatures. The values that have been determined for the strength of concrete at elevated temperatures by different researches have indicated that they can have wide variation (Bazant and Kaplin, 1996). This is due the effect of dehydration and transformation reactions, changes in porosity and pore pressures, differential deformation of the cement paste and aggregates, and moisture content. These things are not only caused by the characteristics of the concrete being tested, by the way of mix proportions and the nature of the material being tested, but also the conditions under which the tests were conducted. Malhotra (1956) and Mohamedbhai (1986) reported that the duration of exposure at a particular temperature and the rates of heating and cooling cause significant changes in the residual strength of heated concrete. Pihlajaavaara (1972) reported that 18 factors must be taken into account when determining the effects of elevated temperature on concrete properties.

Test Methods

To understand the results from different experiments we must first look at the test method used and the effects it can have on the results produced. Specimens can be subjected to different stress and heating conditions before testing, which can effect the values obtained. The different ways a concrete sample can be tested are shown in Figure 2.11.

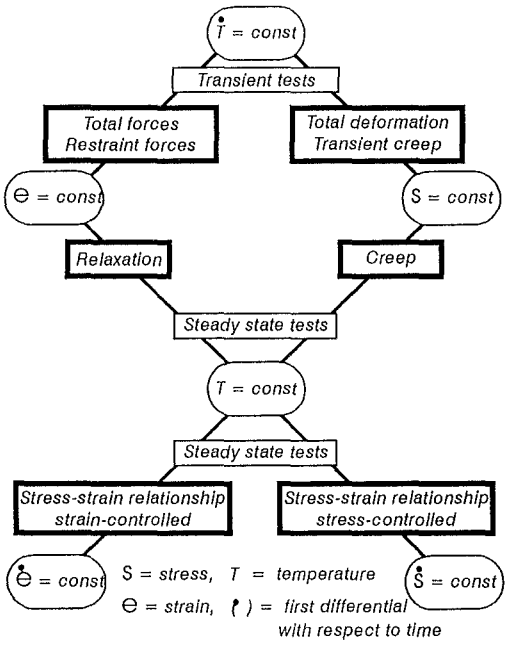


Figure 2.11: Testing regimes for determining the mechanical properties of material at elevated temperatures (Buchanan, 1999).

The specimen can be tested in a 'steady state' condition, with the specimen at a uniform temperature, or 'transient state' condition, where a non-uniform temperature gradient exists through the specimen.

In the steady state method the specimen can be heated to a specific temperature with either no load, or a proportion of the ultimate load applied. Once the specimen is deemed to be at a uniform temperature the stress or strain is increased at a controlled rate until failure occurs. This method produces good results for the strength of concrete at specific temperatures.

In the transient state method the specimen is exposed to a steady or varying temperature environment depending on the type of test being carried out. Similar loading condition to the steady state method can be used. The transient state method can also be used to test specimen where a constant applied load until ultimate failure occurs. This variation defines a failure temperature instead of a failure load. The transient state method is not reliable at predicting the strength of concrete at elevated temperatures due to the non-uniform internal temperature gradient, and the

dependency of the thermal gradient on the environment the specimen is exposed to. In saying this, the transient state method can be considered to be more analogous to the environment that a specimen would be exposed to during a real fire (Bazant and Kaplan, 1996).

The specimen can also be tested at normal temperature for residual strength by allowing it to cool after being heated or loaded, as previously mentioned. This is important for analysis of concrete members that have been exposed to fire. The values obtained from these tests can be affected by the ability of the moisture to exit from the specimen. Tests done on sealed specimen showed different residual strengths compared to unsealed specimen (Lankard et al., 1971). The original strength of the concrete has little effect on the percentage of residual compressive strength of concrete after cooling (Bazant and Kaplan, 1996)

Compressive Strength at Elevated Temperatures

Ordinary and Lightweight Concrete

At elevated temperatures the compressive strength of concrete reduces. The effect the type of aggregate in the concrete has on the compressive strength is shown in Figure 2.12 for specimen unloaded during heating. This figure shows that for temperatures below 450°C the aggregate has little effect on the loss of compressive strength. Above this temperature the siliceous aggregate concrete exhibits a much greater loss of strength than the calcareous or expanded shale lightweight concretes. Similar trends were found for the strength of specimen tested stressed ($0.4\sigma_c$) and the unstressed residual strength. This drop in the compressive strength of siliceous concrete is considered to be caused by the abrupt volume change that occurs due to the inversion of α quartz to β quartz that occurs at 570°C (Bazant and Kaplan, 1996).

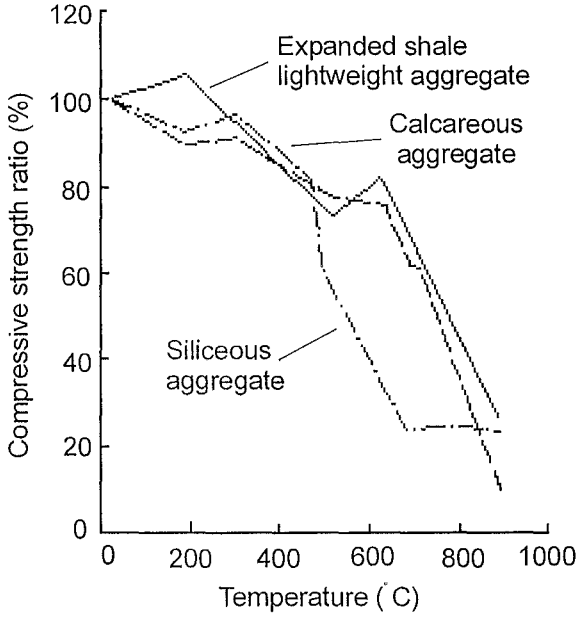


Figure 2.12: Variation of compressive strength for siliceous, calcareous and expanded shale lightweight aggregate concretes. (Bazant and Kaplan, 1996; Figure 5.3)

High Strength Concrete

High strength concrete has been found to have a much different variation of compressive strength than ordinary or lightweight concrete at elevated temperatures. Figure 2.13 shows the results of tests performed by Jumppanen (1989) and Holst (1994). As Figure 2.13 shows high strength concrete temporarily loses a large amount of strength at low temperatures, which does not occur for ordinary or lightweight concretes. The initial temporary loss of strength reaches a minimum of approximately 60% at around 200°C. This is probably due to high steam pressures that build up in the temperature interval between 150-200°C as vaporising water is trapped within the concrete (Tomasson, 1998).

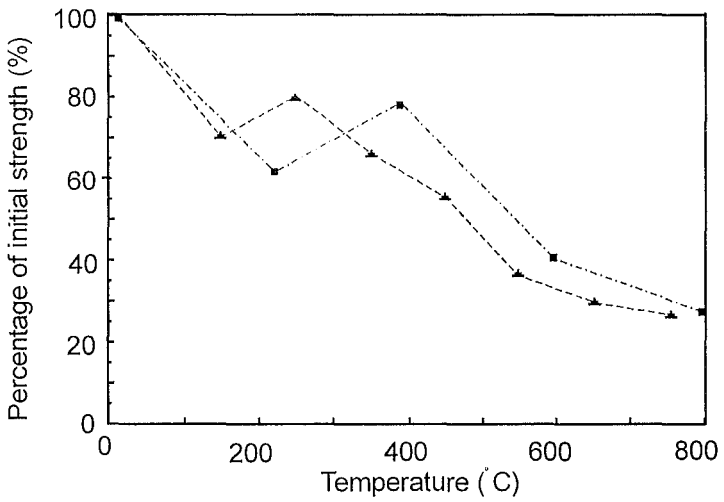


Figure 2.13: Variation of compressive strength of high strength concrete at elevated temperatures. (Tomasson, 1998)

Comparison with NZS3101

Figure 2.14 shows the comparison of the strength reduction curve given by NZS3101 with existing experimental data for siliceous, calcareous and lightweight concrete at elevated temperatures. The curve given by NZS3101 gives a good correlation with the experimental data.

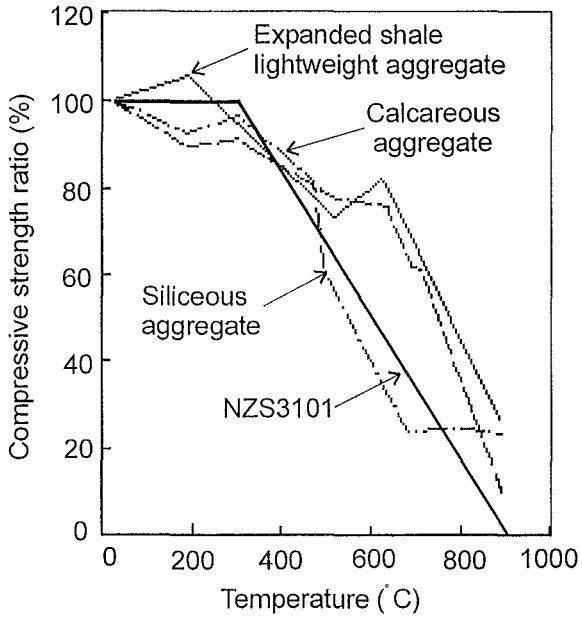


Figure 2.14: Comparison of existing experimental data for strength of siliceous, calcareous and lightweight concrete at elevated temperature with the recommended values from NZS3101.

High Strength Concrete

Figure 2.15 shows the comparison of experimental data for high strength concrete at elevated temperatures with the recommended values from NZS3101. It shows that the values recommended by NZS3101 greatly overestimate the strength of high strength concrete at elevated temperatures.

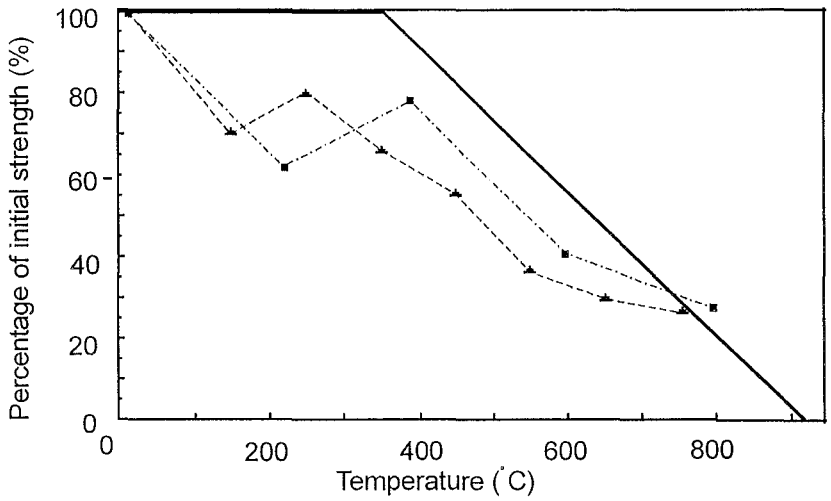


Figure 2.15: Comparison of existing experimental data for strength of high strength concrete at elevated temperatures with the recommended values from NZS3101.

2.5. *Elastic Modulus at Elevated Temperatures*

The elastic modulus of concrete reduces with increasing temperature. This is caused by the breaking of bonds in the microstructure of the cement paste as it is heated. The temperature increase also increases the rapid short-time creep causing an additional apparent decrease in the elastic modulus. Figure 2.16 show the decrease of the elastic modulus of siliceous, calcareous, and lightweight aggregate concrete, and high strength concrete at elevated temperatures.

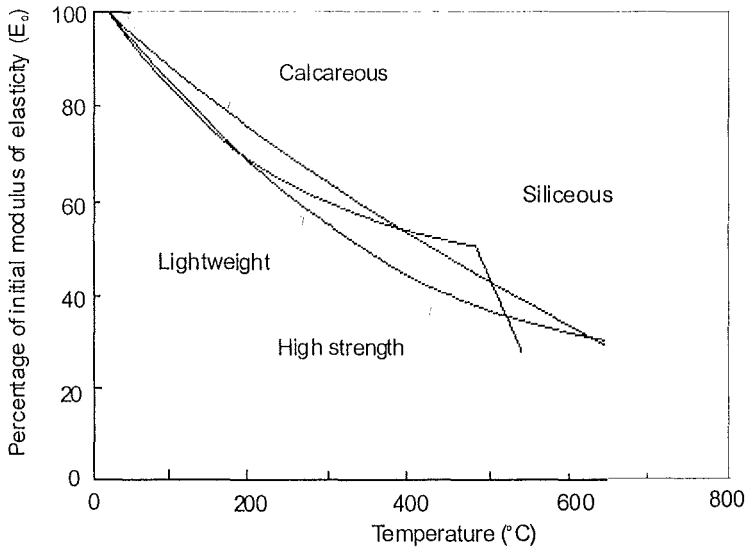


Figure 2.16: Modulus of elasticity for high strength (Tomasson, 1998) and siliceous, calcareous, and lightweight aggregate (Cruz, 1966) concrete at elevated temperature.

From Figure 2.16 it can be seen that the aggregate type does have an effect on the decrease of the elastic modulus. Aggregates that are more compatible and chemically stable, such as limestone (calcareous), have a slower decrease than other aggregates (Cruz, 1966). The siliceous aggregate concrete has a smooth decrease of its elastic modulus to about 500°C, above which the elastic modulus begins to drop dramatically. This corresponds to a similar drop in the compressive strength of siliceous concrete, as show in Figure 2.16, caused by the inversion of α quartz in the aggregate to β quartz.

Aoyagi et al. (1972) found that the relative decrease of the elastic modulus is much larger for high strength concrete than in low strength concretes. This trend is shown by results from Jumppanen (1989) in Figure 2.16.

The moisture in the concrete has been found to cause a variation in the reduction of the elastic modulus. Lankard et al. (1971) reported sealed specimen exhibited lower values of elastic modulus than dried specimen. However, Bertero and Polivka (1972) did not observe this to have a strong effect. Gross (1973) and Sullivan and Labani (1974) found that the moisture movement that occurs as with the temperature increase of the specimen reduces the apparent elastic modulus value. This has been attributed to the movement of moisture causing some bond ruptures in the cement. This shows that the rate of heating has an affect on the values of elastic modulus at elevated temperature, as it influences the rate of moisture movement in a specimen. Harada (1959) found that the higher the rate of heating the greater the decrease of the elastic modulus. A higher rate of heating causes higher thermal stresses, higher pore pressures, and higher stresses from thermal shrinkage. These further damage the microstructure of the concrete, which result in the reduction of the elastic modulus.

The elastic modulus of specimen which have been heated and allowed to cool before testing have shown a similar reduction. This indicates that the damage to the microstructure of the concrete that occurs at elevated temperature is permanent (Harada, 1957)

Comparison with NZS3101

Figure 2.17 shows the comparison of the recommended values of the modulus of elasticity of concrete at elevated temperatures from NZS3101 with existing experimental data. For temperatures below 450°C the values recommended by NZS3101 have a good correlation with the experimental data. Above this temperature the values become conservative for lightweight and calcareous concrete. This is due to the values from NZS3101 allowing for the dramatic decrease in the elastic modulus of siliceous concrete at 500°C. The values from NZS3101 then continue to decrease until 0% initial elastic modulus is reached at 720°C.

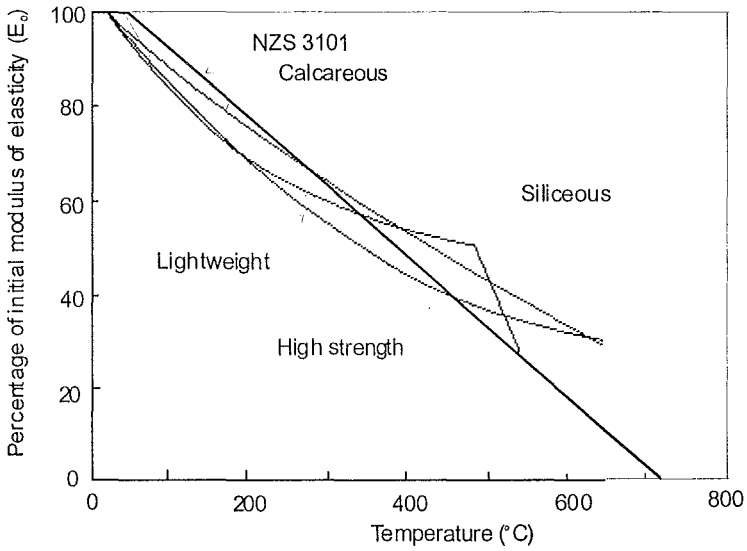


Figure 2.17: Comparison of existing experimental data (Cruz, 1966; Jumppanen, 1989) for reduction of modulus of elasticity at elevated temperature with recommended values from NZS3101.

As can be seen from Figure 2.17 there is a lack of experimental data for the elastic modulus of concrete at temperatures above 600°C. This is the region in which the values for the elastic modulus given in NZS3101 become questionable. Figure 2.14 shows the compressive strength reducing to zero at 910°C, which is well above the 20°C at which the elastic modulus has reached zero. This gives the impression of infinite deformation of concrete at temperatures above 720°C, which is not what occurs in reality.

To give some consistency between the values of strength and elastic modulus given in NZS3101 the slope of the lower portion could be adjusted to reach zero at 910°C. Figure 2.18 shows a possible variation of the values from NZS3101 that could be used. It should be noted that this variation has no experimental basis.

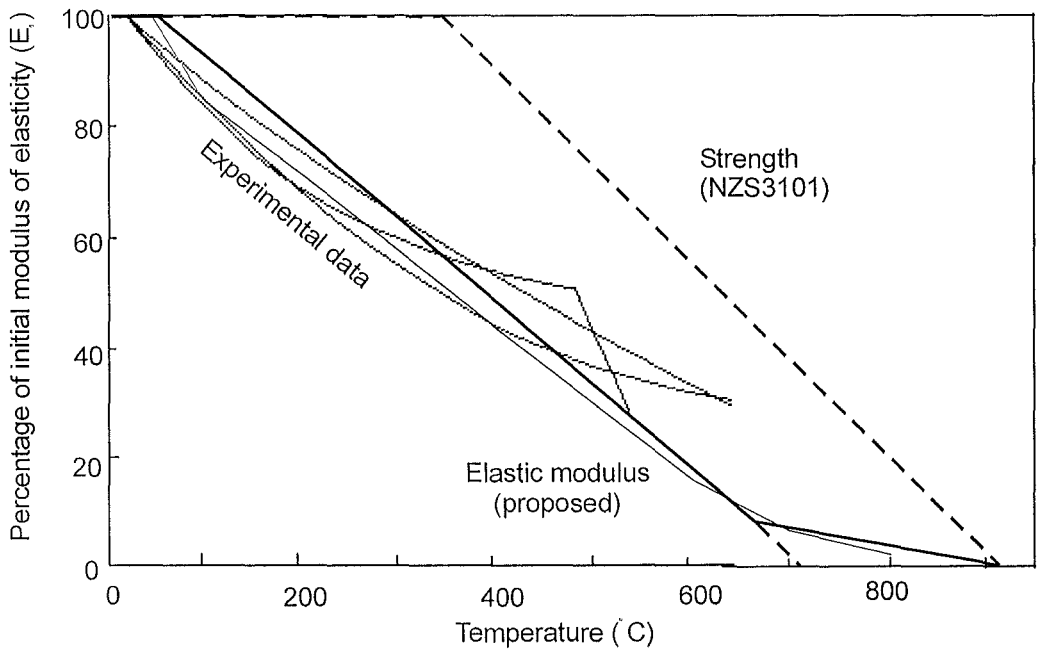


Figure 2.18: Possible variation to the values of elastic modulus at elevated temperature from in NZS3101 to give consistency with compressive strength at elevated temperature.

2.6. *Spalling*

Spalling can have insignificant or detrimental effects on the fire resistance of a concrete member depending on the type of spalling that occurs. Though most spalling that occurs from exposure to fire causes only superficial damage to a concrete member, the worst types of spalling cause the ejection of a large area of concrete from the exposed surface. This has the effect of reducing the protective cover or thickness of the concrete below that assumed in design calculations, which can lead to the premature insulation or structural failure of the member. Currently NZS3101 does not give advice for the consideration of spalling when designing a member.

Types of Spalling

The different types of spalling that have been observed during fire endurance testing have been grouped into categories based on the location, nature, and severity of the spalling;

- Explosive spalling is the ejection of large pieces of concrete from the surface of a member due to high pore pressures caused by the production of steam within the concrete.
- Surface spalling includes pitting, blistering and local removal of surface material
- Aggregate splitting is the splitting or failure of the aggregate near the surface and is often accompanied by surface splitting
- Sloughing off occurs when the surface layer or corner of a concrete member is gradually eroded due to extended exposure to fire.

Factors that Influence Spalling

There are several factors that have been noted to influence the occurrence and scale of spalling. (Lie (1972), Malhotra (1984), Schneider (1985))

Moisture content of the concrete
Compressive stress caused by restraint of thermal expansion or external load
Aggregate type
Rapid temperature rise at exposed surface
Concrete density and permeability

From these five factors it is generally accepted that the first three are the most influential to spalling, though there is a much higher occurrence of explosive spalling in high strength concrete containing silica fume. This is due to the extremely low permeability of concrete containing silica fume.

High Strength Concrete Containing Silica Fume

High strength concretes containing silica fume have been found to be more susceptible to explosive spalling in comparison to ordinary or lightweight concretes. This is caused by the extreme reduction in the permeability of the concrete through the addition of the silica fume. It has been found that the addition of as little as 5% silica fume can reduce the coefficient of permeability of the concrete by 3 orders of magnitude (Khayat and Aitcin 1992). This has the effect of reducing the level of moisture content that may cause explosive spalling. Explosive spalling has been noted

in tests of concrete containing silica fume with water/cement ratios of only 0.26 (Jumppanen (1989)).

The extremely low permeability of concrete containing silica fume also decreases the rate at which the evaporated moisture can pass through the concrete to relieve the built up pore pressure. This can be seen in Figure 2.19 that shows the temperature, moisture, and pore pressure profiles through a slab after being exposed to a fire for some time.

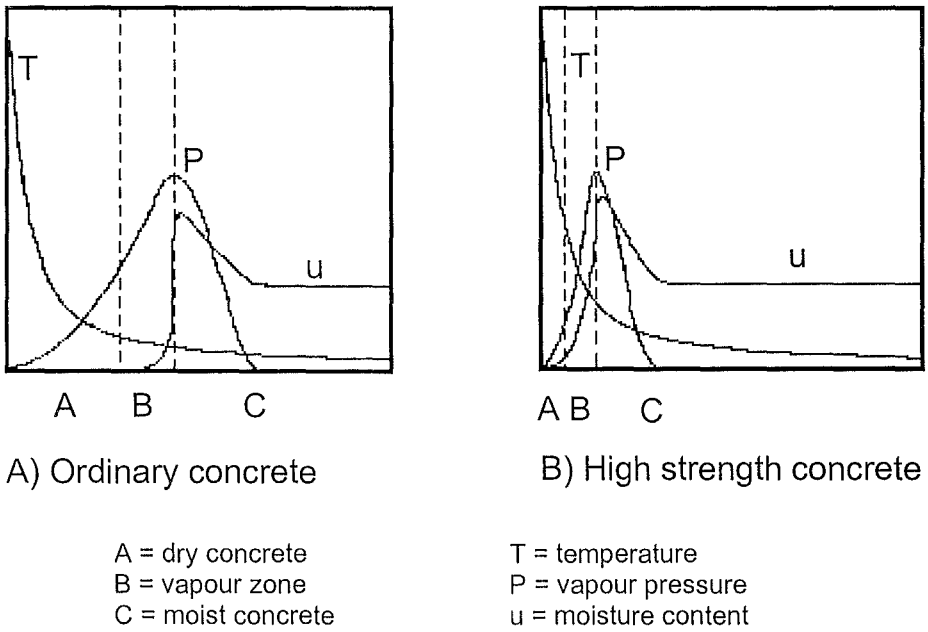


Figure 2.19: Temperature, pressure, and moisture profiles through a slab exposed to a fire (Anderburg, 1998).

This reduces the rate of temperature rise required to produce pore pressures sufficient to cause explosive spalling. Hertz (1992) observed explosive spalling in concrete heated over about 300°C, with only a relatively slow rate of temperature rise of 60°C per hour, which is much slower than if it were being exposed to a real fire.

Controlling Occurrence Spalling

To reduce the susceptibility of concrete to explosive spalling polypropylene fibres can be added to the concrete. This has been shown in experimental studies by Matsumoto

et al. (1994) on high strength reinforced columns, which showed a dramatic reduction in the degree of spalling in columns containing polypropylene fibres.

The polypropylene fibres reduce the occurrence of spalling by providing a path for the evaporated moisture to travel through the concrete by melting out at elevated temperatures. This relieves some of the built up pore pressures that contribute to spalling.

3. Experimental Analysis of Insulation Fire Resistance

3.1. Overview

Wade et al (1991) conducted the initial tests on the fire performance of non-load bearing concrete walls containing New Zealand aggregates. These tests were conducted on 1m x 1m slabs with a uniform thickness of 130mm. Wade (1992) completed additional testing on similar slabs with uniform thicknesses of 60mm and 175mm. The results from these tests have been used as the basis of the insulation fire resistance requirements in NZS3101.

This series of tests will be conducted on expanded clay lightweight aggregate concrete and high strength concrete specimens similar to those tested by Wade. An additional test will be performed on a pair of ribbed floor slab specimen. From the results of the tests the applicability of the current values given in NZS3101 to lightweight, high strength and ribbed slabs will be determined.

3.2. Method

The specifications of the concrete slabs were similar to those tested during the earlier tests by Wade et al (1991) and Wade (1992). Details and dimensions are shown in Figure 3.1. An additional pair of 130mm normal weight concrete slabs was prepared incorporating a steel tray deck flooring system as shown in Figure 3.2.

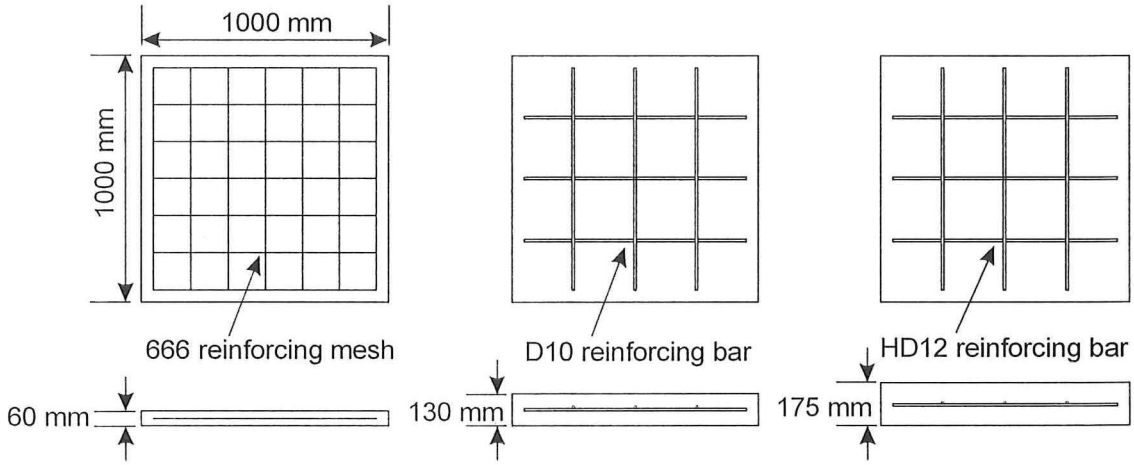


Figure 3.1: Plain slabs specimen nominal dimensions and reinforcement

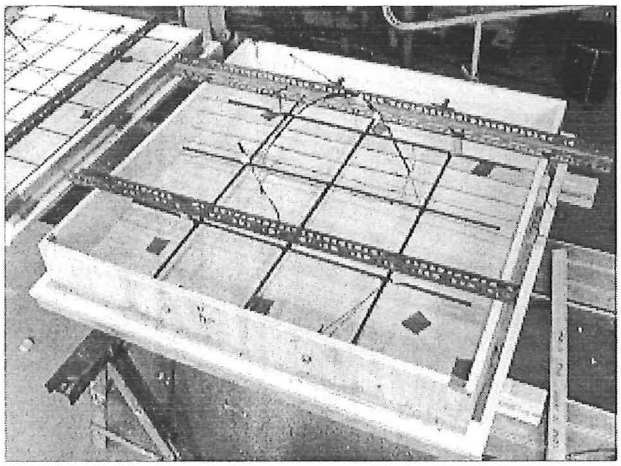


Figure 3.2: Mould for 130mm plain slab.

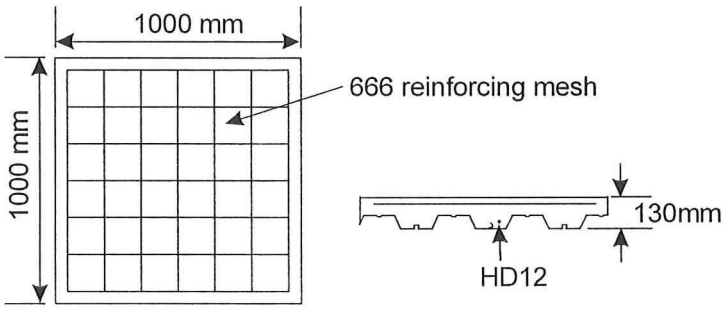


Figure 3.3: 1m x 1m steel tray flooring slabs

A slab of each nominal thickness was produced for both a lightweight and high strength concrete. An additional slab was produced for each nominal thickness and concrete type with polypropylene fibres added. This was done to attain comparative data on the effect the polypropylene fibres have on the thermal characteristics of the different concrete types.

Materials and Placement

Concrete

The high strength and lightweight concretes were supplied by Firth, while the normal weight concrete was produced on site at the University of Canterbury in the Civil Engineering Structures Lab. Descriptions of the concrete mixes used are supplied in Table 3.1.

Table 3.1: Description of concrete mix specifications based on $1m^3$.

a) 80 MPa High Strength Concrete

Mass (kg)	Volume (m^3)	Description
1188 kg	0.445	13mm Greywacke aggregate
414 kg	0.156	Cement sand (Yaldhurst sand)
104 kg	0.039	Dune sand (Kaiapoi sand)
535 kg	0.170	General Purpose Cement
163 L	0.163	Water
47 kg		Microposz (Silica fume)
5.8 L	0.006	Super plasticiser
0.9 L	0.000	Water reducer

b) 30MPa Lightweight Concrete

Mass (kg)	Volume (m^3)	Description
794 kg	0.690	Lightweight aggregate
735 kg	0.277	Sand
390 kg	0.124	Cement
1 L		Water reducer
150ml		Air entraining admix

c) 30MPa Normal Weight Concrete

Mass (kg)	Volume (m^3)	Description
564	0.211	19mm Greywacke aggregate
554	0.208	13mm Greywacke aggregate
674	0.254	Cement sand (Yaldhurst sand)
169	0.063	Dune sand (Kaiapoi sand)
260	0.083	Cement
650ml		Water reducer
30ml		Air entraining admix

Steel

The reinforcing steel and its location in the slabs with uniform thickness were similar to those used by Wade et al (1991) and Wade (1992). The reinforcing steel was placed at mid depth in each of the slabs with uniform thickness. The steel placed in each slab varied depending on the thickness of the slab as shown by Figure 3.1. Table 3.2 lists the reinforcing steel that was used in each slab.

Table 3.2: Reinforcing steel used in each slab of uniform thickness

Slab Thickness (mm)	Reinforcing Steel
60	665 mesh
130	D10
175	HD12

Dimond Industries supplied the Hi-Bond steel flooring tray. The steel requirements were specified in the Hi-Bond Design Manual based on a continuous 2.6m span in a commercial occupancy requiring a 49 minute fire resistance rating (Example 4, Hi-Bond Design Manual 1997). The steel tray floor slabs had 666 mesh placed at a minimum cover depth of 25mm. The additional fire reinforcement consisted of a HD12 reinforcing bar in the bottom of a rib at a minimum cover of 25mm to the bottom and 40mm to the side of the rib. This is shown if Figure 3.3.

Drying and Relative Humidity Measurement

The slabs were cast horizontally and wet cured vertically for 7 days to obtain good hydration of the cement and simulate realistic construction practices. The slabs were then stacked and stored horizontally, as shown if Figure 3.4, with a minimum 50mm gap between them to allow for ventilation, at the side of the lab in a shaded area.

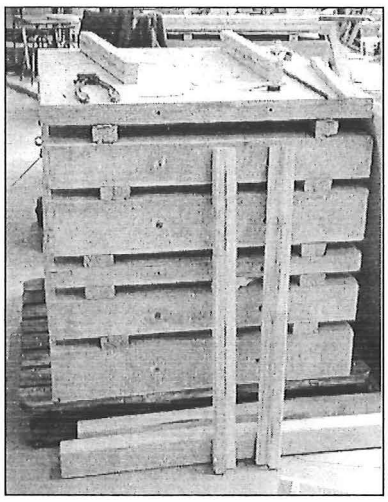


Figure 3.4: Specimen stacked in lab

Each slab had a single relative humidity sensor placed at a mid depth level in a 10mm diameter well. The wells were drilled into the slabs after the slab had cured. The relative humidity sensor and thermistor were sealed into the well with a silicon sealant, as had been used for the previous tests by Wade et al. 1991. The relative humidity sensor was a HH-3605-A. A thermistor was also placed into the well to provide an accurate temperature for the evaluation of the conditions at mid depth of the slab. They were supplied with a stable DC voltage from a battery cell consisting of 4 1.5V AA batteries. The input and output voltages were measured with a calibrated HP3654 multi-meter.

Fire Test Method

The concretes were tested in the 1.0m x 2.2m diesel fired pilot furnace at the BRANZ Fire Laboratory, Wellington, shown in Figure 3.5. The testing was carried out to ISO 834 (1975), between December 1998 and January 1999.

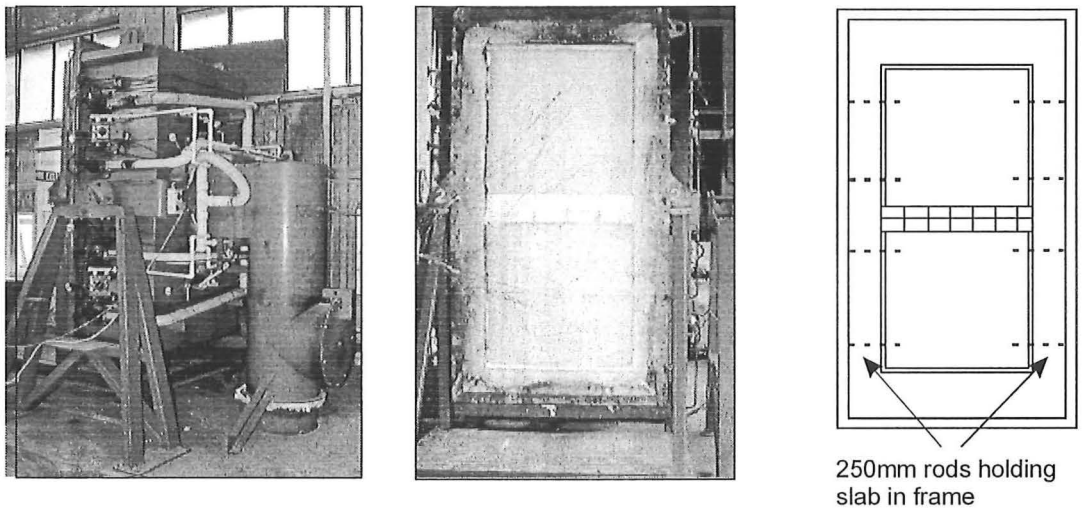


Figure 3.5: Pilot furnace at BRANZ (Wellington), and specimen mounted in furnace frame.

The slabs were tested in a vertical orientation and fastened to a frame with four 250mm long 16mm diameter rods, with no external loads applied, as shown in Figure 3.5. The rod holes were drilled while the slabs lay in the frame and the rods hammered into place. This provided some restraint against thermal expansion, though was not expected to have any effect on the fire performance of the slab.

Temperature Measurement

The unexposed surface temperatures were determined using five 'key' thermocouples, type K chromel/alumel disc thermocouples, placed in accordance with ISO 834, as shown in Figure 3.6. The internal temperatures were measured with thermocouples at various depths through the slab. These thermocouples were attached to 1.5mm high tensile wire strung parallel to the firing surface, located within 2mm of the specified depth. The temperatures of the reinforcing bars were also measured by five thermocouples placed on the reinforcing, as shown in Figure 3.6.

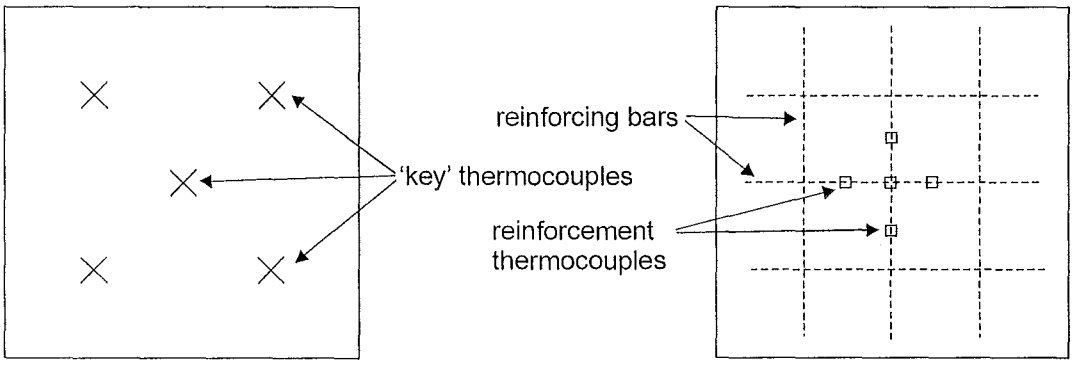


Figure 3.6: Location of thermocouples on unexposed surface and reinforcing bars.

Data Logging

The thermocouples were connected to a Hewlett Packard 3497 data acquisition unit (DAU) controlled by a Hewlett Packard 9000 computer.

3.3. Experimental Results

Insulation Fire Resistance

The insulation fire resistance of a specimen is defined as the time for either;

- an average temperature rise of 140°C at the five 'key' thermocouples placed on the unexposed face of the specimen, or
- a single point on the unexposed face of the specimen reaches 180°C.

The times for each slab to reach insulation failure are listed in Table 3.3.

Table 3.3: Mid depth relative humidity and times to insulation failure from test.

Specimen Type	Nominal Thickness (mm)	Fibres	Relative Humidity (%)	Mean Time for $\Delta T>140^{o}C$ (minutes)
Lightweight	60	n	76	75
	60	y	77	75
	130	n	86	309
	130	y	88	320
	175	n	89	no result[1]
	175	y	88	no result[1]
High Strength	60	n	73	54
	60	y	73	50
	130	n	86	187
	130	y	85	181
	175	n	87	335
	175	y	86	340
HiBond (ordinary cement)	130	n	90	59[2]
	130	y	92	89

[1] test terminated after 348 minutes with unexposed surface at 90°C.

[2] due to the high moisture content in the specimen destructive spalling occurred causing early insulation failure.

Variation of Temperature through Slab

The temperature of the furnace, the unexposed surface, and various depths within the slabs were measured during the tests. Figures 3.7 to 3.18 show the variation of the temperature of the slab as the test progressed. The furnace temperatures and ISO834 curves have also been included on these figures.

The depths at which the internal temperatures were measured are shown on the figures. The internal temperatures at the various depths within the slab are given as either the temperature from a single thermocouple or the average of two thermocouples where data was available.

All the figures show a steady increase in the temperature through the slab over the duration of the test. The evaporation of free water within the slab caused the

temperatures to plateau for some time at around 100°C. After the free water had been removed from an area the temperature continued to increase again. Through comparison of the internal temperatures it can be seen that the polypropylene fibres have an insignificant effect on the heating of the slab.

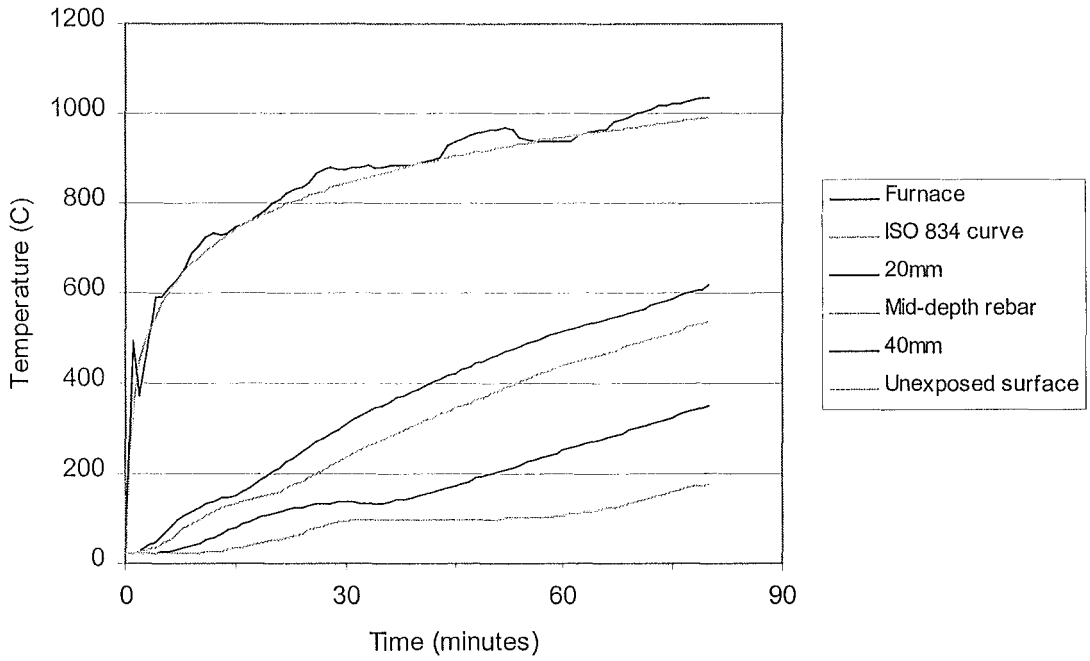


Figure 3.7: 60mm expanded clay lightweight concrete slab.

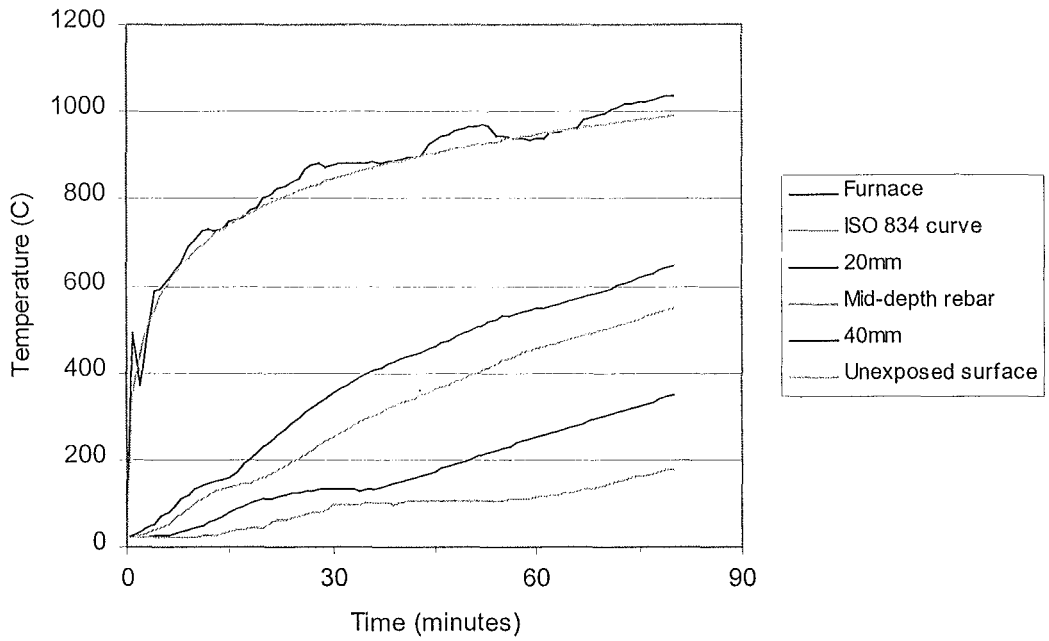


Figure 3.8: 60mm expanded clay lightweight concrete slab with polypropylene fibres.

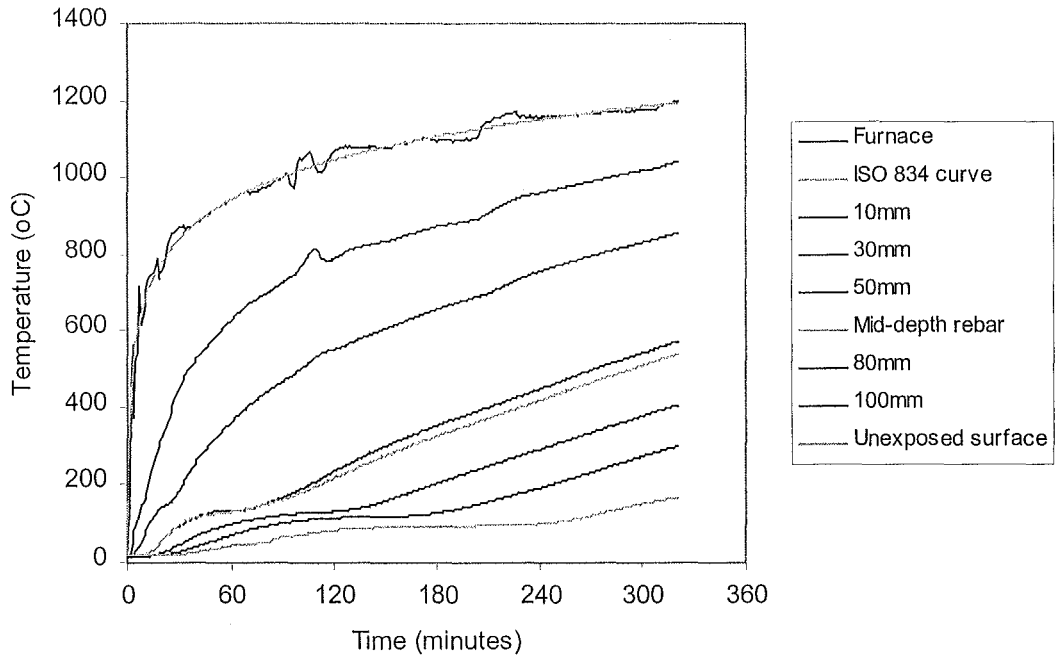


Figure 3.9: 130mm expanded clay lightweight concrete slab.

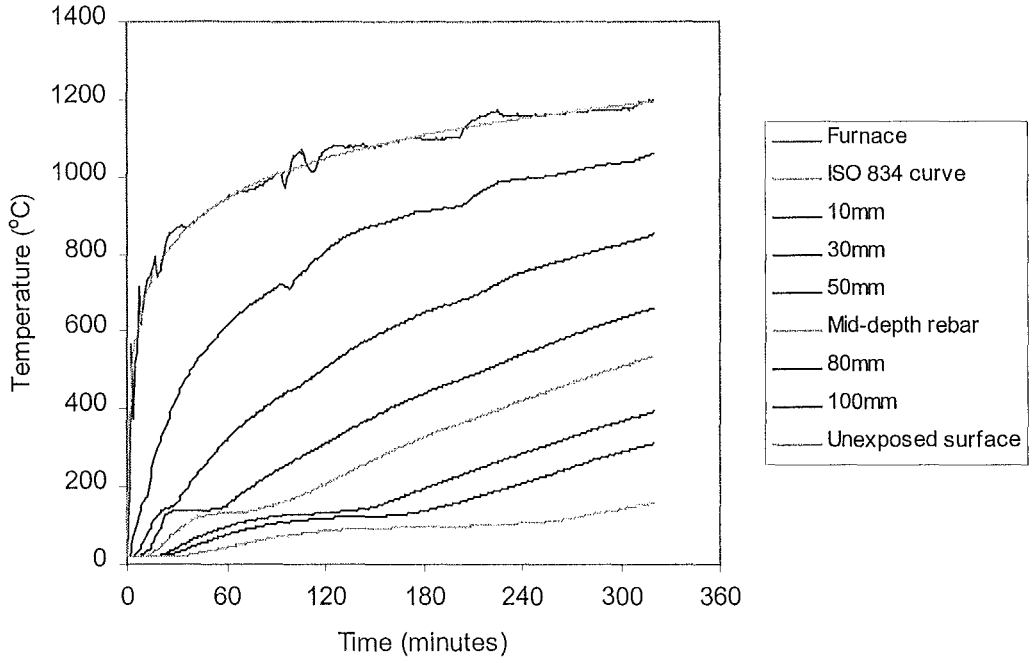


Figure 3.10: 130mm expanded clay lightweight concrete slab with polypropylene fibres.

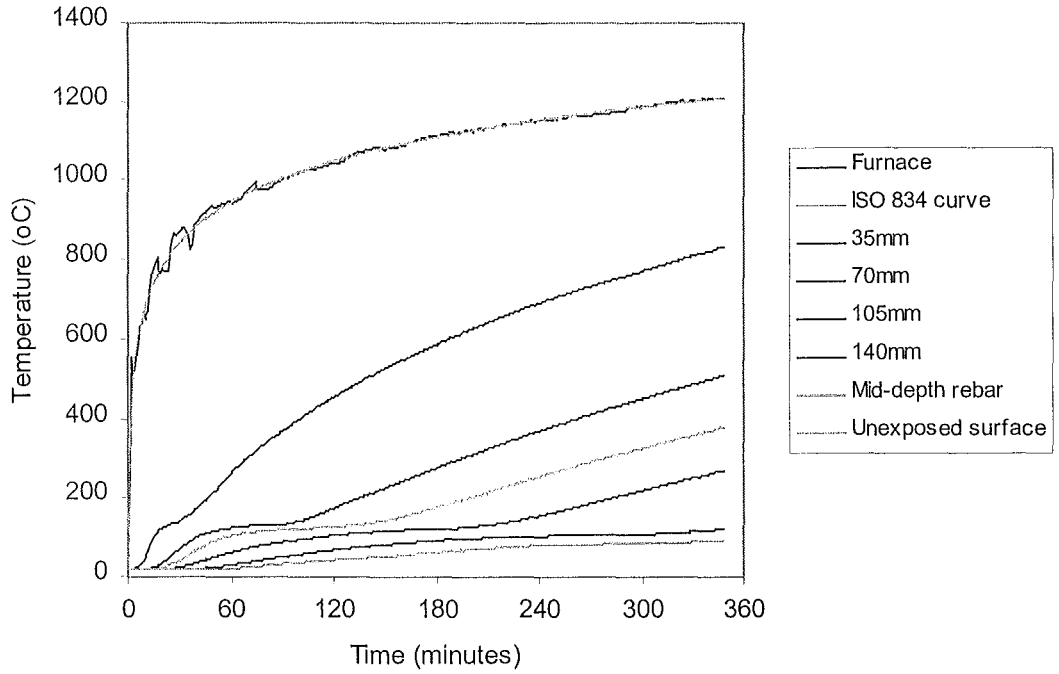


Figure 3.11: 175mm expanded clay lightweight concrete slab.

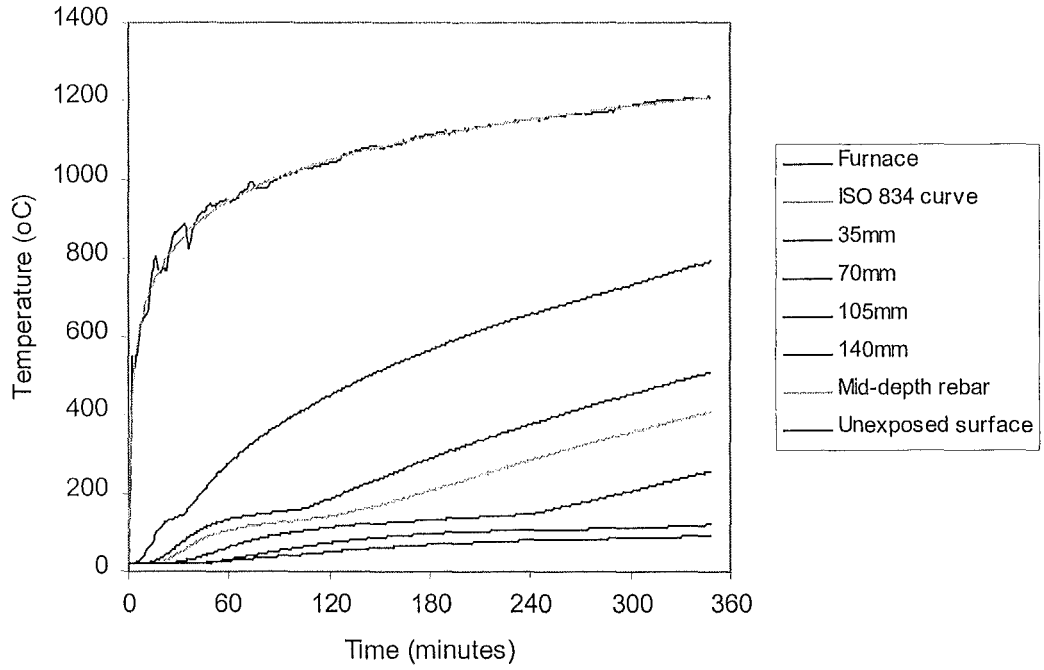


Figure 3.12: 175mm expanded clay lightweight concrete slab with polypropylene fibres.

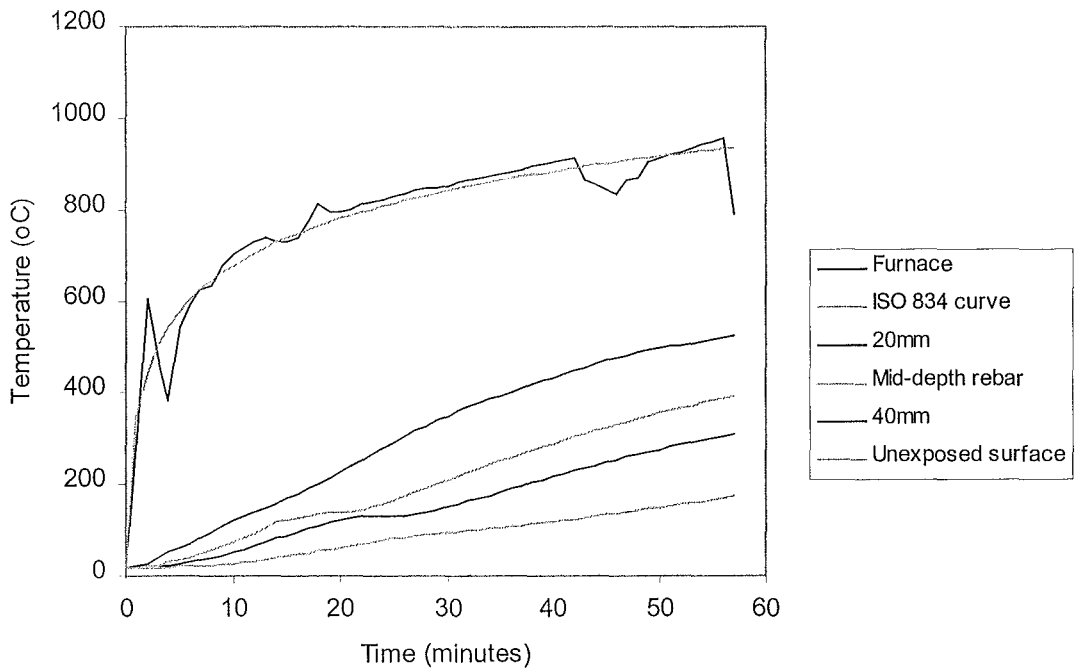


Figure 3.13: 60mm high strength concrete slab.

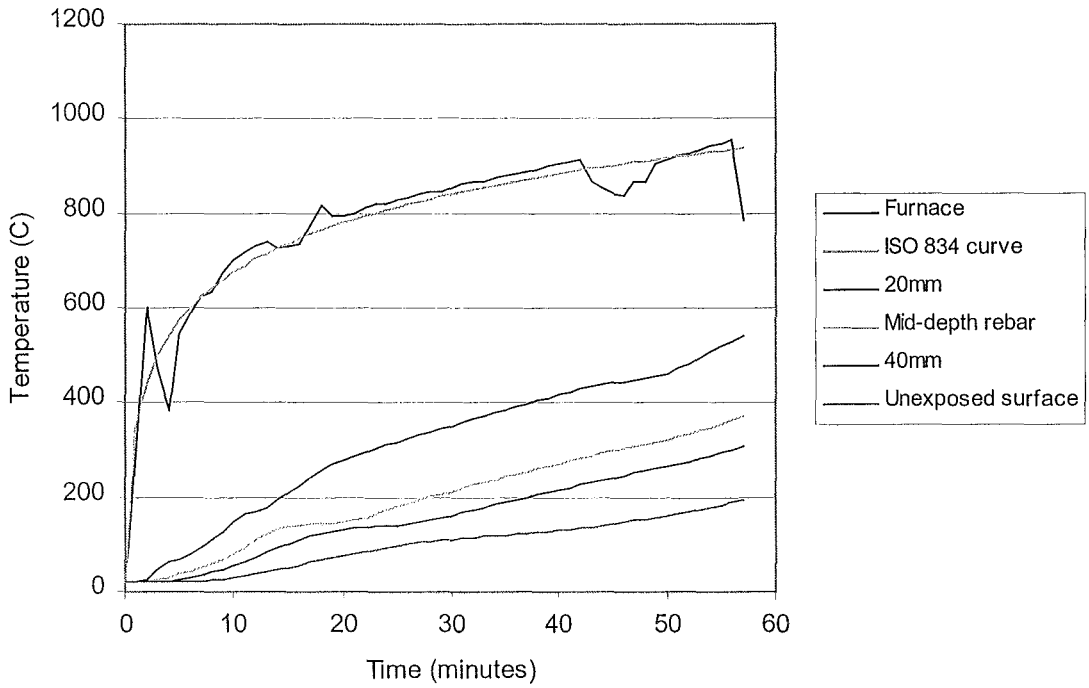


Figure 3.14: 60mm high strength concrete slab with polypropylene fibres.

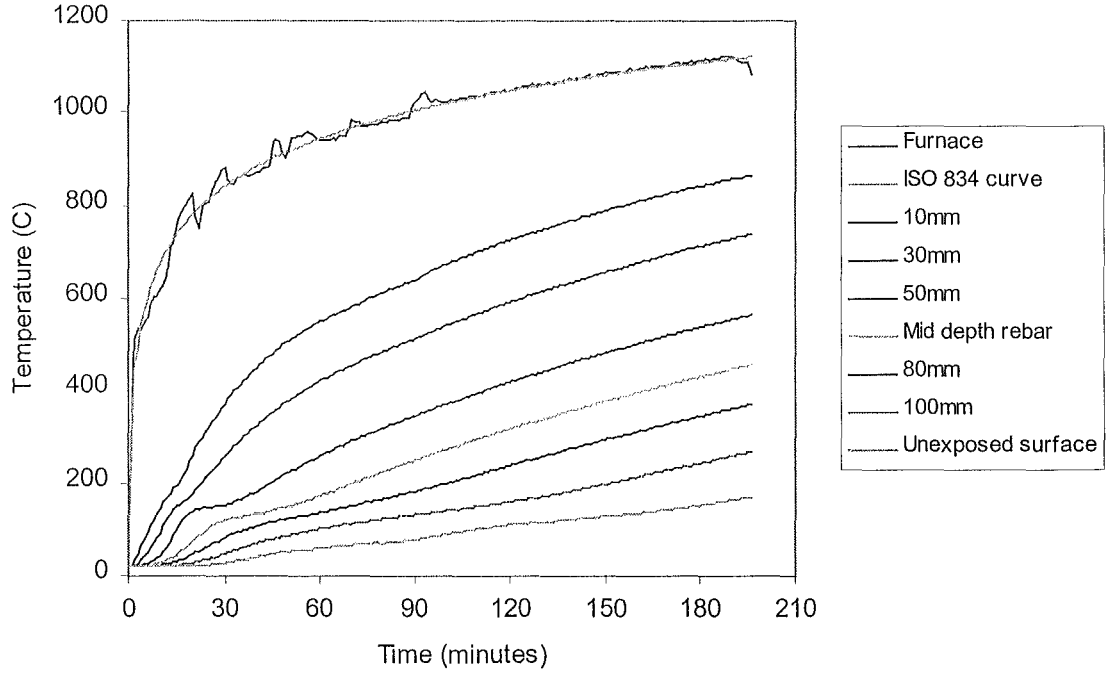


Figure 3.15: 130mm high strength concrete slab.

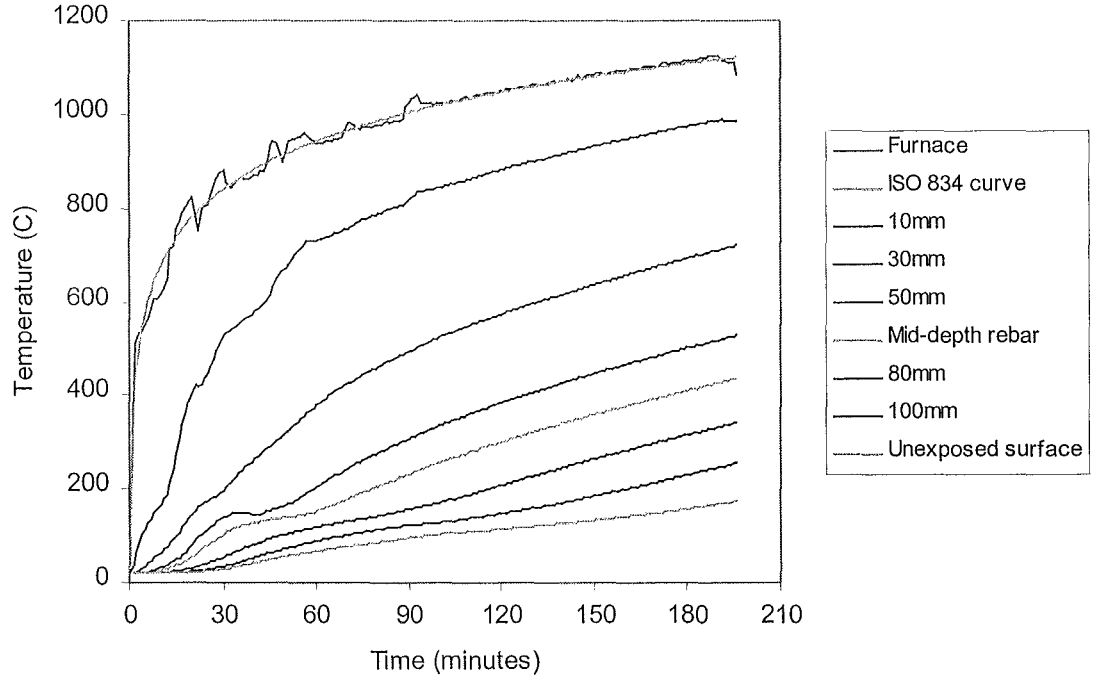


Figure 3.16: 130mm high strength concrete slab with polypropylene fibres.

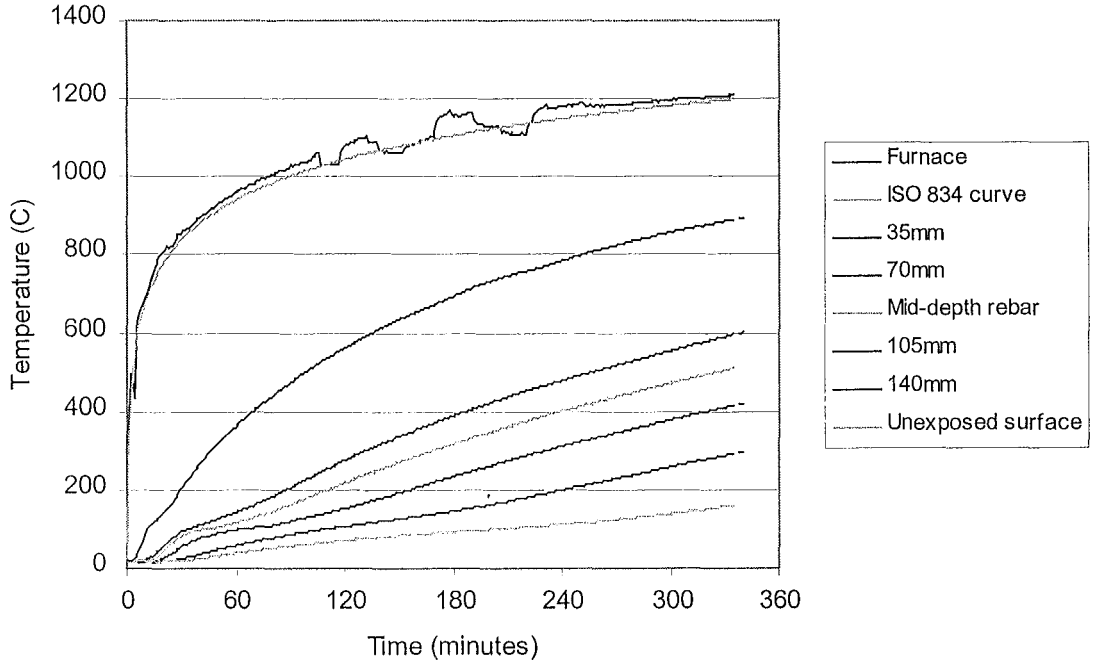


Figure 3.17: 175mm high strength concrete slab.

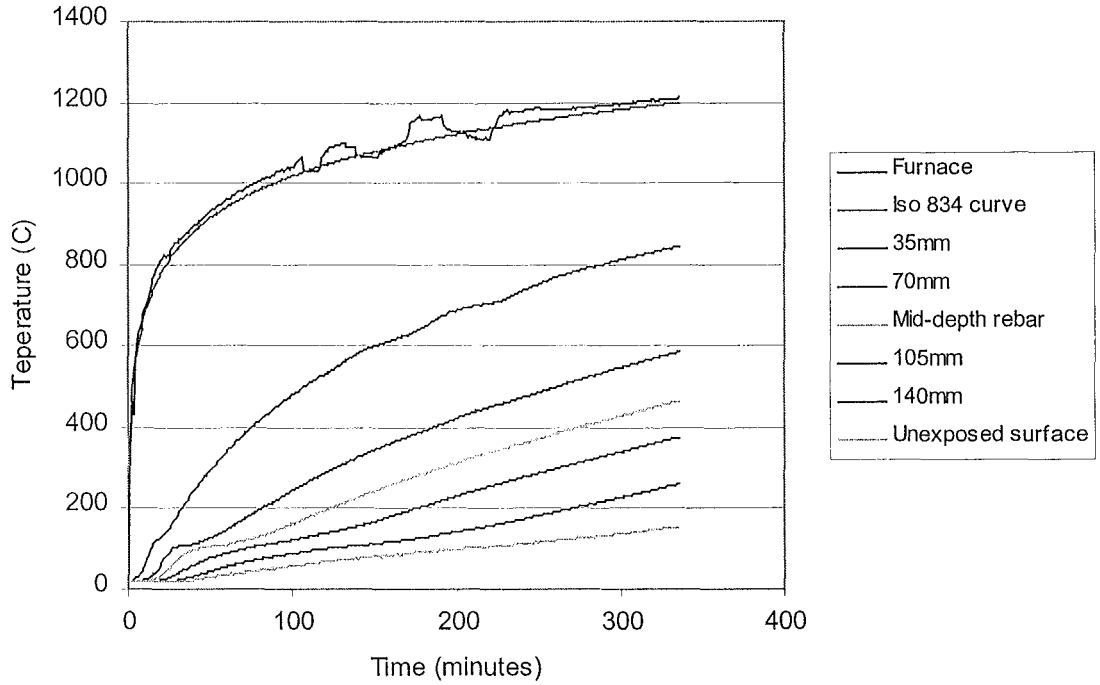


Figure 3.18: 175mm high strength concrete slab with polypropylene fibres.

3.4. *Observations*

Uniform Thickness Slabs

During the progression of the test, the majority of the specimen showed only minor spalling in the form of blistering and aggregate splitting, as shown in Figure 4.4. The 175mm plain high strength slab was the only exception for the uniform thickness specimens. It lost a large area from the exposed surface. This is shown in Figure 4.5.

With the thicker specimens sustaining much longer exposure to the elevated temperatures within the furnace, the effects were much more pronounced. Figure 3.19 shows a large melted area on the exposed face of the 175mm lightweight specimen. The concrete appeared to become liquefied and started sliding down the exposed face. Similar areas of melting were noted on the 175mm high strength specimens.

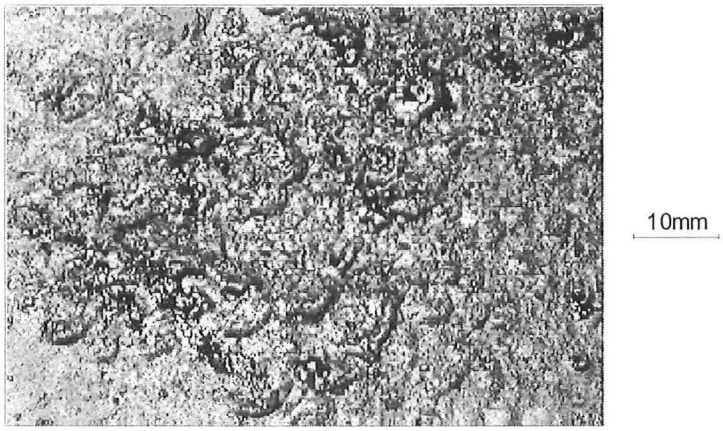


Figure 3.19: Melted area on the face of the 175mm lightweight specimen. (exposure time: 5 hours 48 minutes)

The thin slabs showed the effects of the large thermal gradient that occurred between the exposed and unexposed surfaces. Figure 3.20 shows a crack that formed across the unexposed surface of the 60mm high strength slab containing fibres. The crack appeared at the centre of the slab after 48 minutes and quickly propagated across the unexposed surface. The specimen was also heavily cracked on the exposed face, as

shown in Figure 3.21. These cracks occurred as the slab began to cool after the furnace was opened.

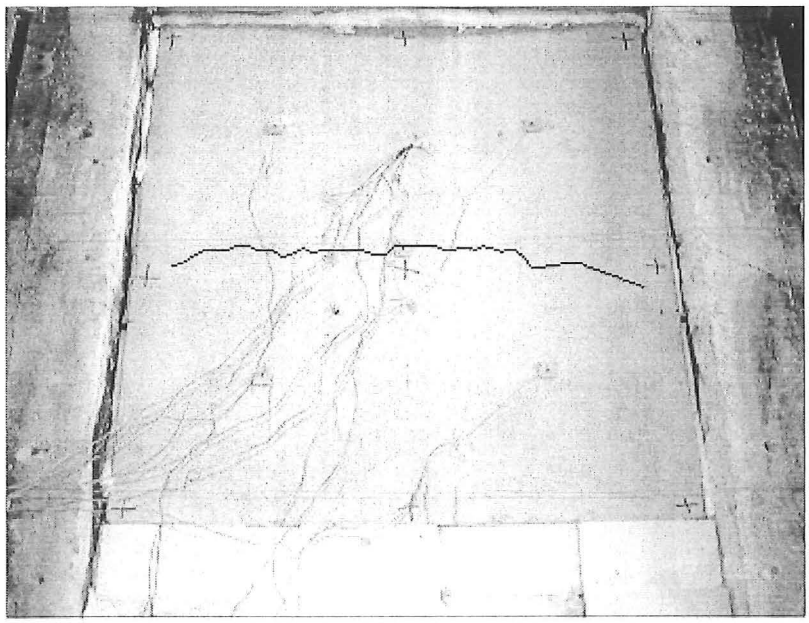


Figure 3.20: Crack across unexposed surface of 60mm high strength slab with fibres (photo at 50 minutes from test commencement)

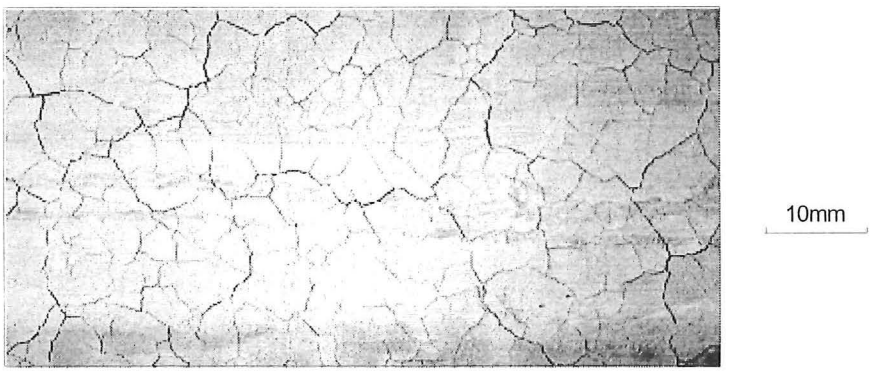


Figure 3.21: Cracking that occurred during cooling of 60mm high strength specimen.

Ribbed Slabs

The one ribbed slab that did not contain the polypropylene fibre suffered destructive spalling due to the high moisture content in the slab at the time of testing. Figure 3.22 shows that a sufficient amount of concrete was spalled from the exposed surface to expose the reinforcing steel in the top of the slab. The steel tray fell from the face of

the slab after 20 minutes, leaving the bare concrete face exposed until the test finished 50 minutes later. This would have largely contributed to the damage that occurred to the exposed surface. The slab containing the polypropylene fibres only showed minor buckling of the steel tray. This is shown in Figure 3.23.

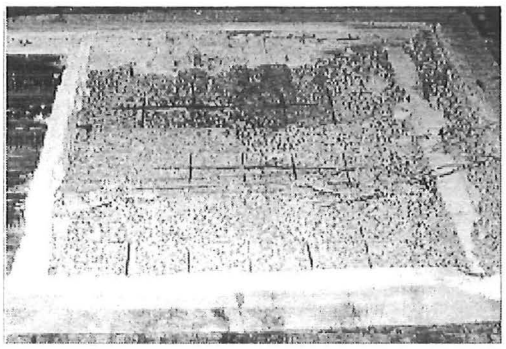


Figure 3.22: Damage caused to ribbed slab with no polypropylene fibres caused by spalling. (exposure time: 1 hour 30 minutes)

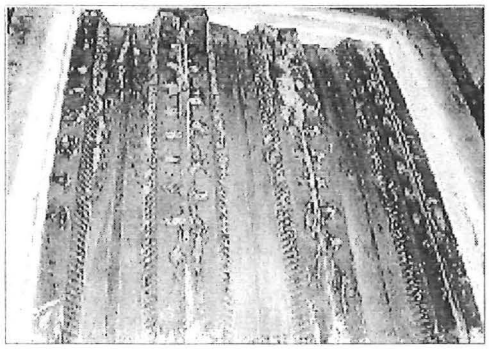


Figure 3.23: Bucking of steel tray on specimen containing polypropylene fibres. (exposure time: 1 hour 30 minutes)

3.5. Analysis

Correction for Non-Standard Moisture Content

At testing not all the specimens had reached a mid-depth relative humidity close to the required 75% standard mid-depth relative humidity. To allow the results to be comparable with previous tests a correction had to be applied to the mean time for a 140°C temperature rise on the unexposed face. The method used is described in ASTM E119 (1995). The uncorrected and corrected values for the mean times for a 140°C temperature rise on the unexposed surface are given in Table 3.4.

Statistical Variation of Results

To find the accuracy of the mean time to reach a 140°C temperature rise on the unexposed face the 95% confidence interval was evaluated. The 95% confidence limits were evaluated using a T distribution analysis that takes into account of the small size of the sample. The 95% confidence limits are listed in Table 3.4.

Table 3.4: Experimental results for insulation fire resistance tests on lightweight and high strength concrete.

Specimen Type	Nominal Thickness (mm)	Fibres	Relative Humidity (%)	Mean Time for ΔT>140°C (minutes)	Mean Time for ΔT>140°C corrected to 75% RH (minutes)	95% Confidence Interval (minutes)
Lightweight	60	n	76	75	71	±27
	60	y	77	75	71	±34
	130	n	86	309	305	±5.7
	130	y	88	320	315	±15
	175	n	89	Cold surface at 90.2°C after 350 minutes		±1.7
	175	y	88	Cold surface at 90.2°C after 350 minutes		±2.9
High Strength	60	n	73	54	54	±18
	60	y	73	50	50	±22
	130	n	86	187	183	±4.5
	130	y	85	181	178	±2.1
	175	n	87	335	329	±4.4
	175	y	86	340	334	±11
HiBond (ordinary)	130	n	90	59	55	no result[1]
	130	y	92	89	84	no result[1]

[1] insufficient number of thermocouples to give confidence interval

Variation of Insulation Fire Resistance with Aggregate Type and Thickness for a Slab of Uniform Thickness

The relationship between the thickness insulation fire resistance of a slab can be approximated with an empirical correlation proposed by Abrams and Gustaferro (1968). They described the following power curve expression.

$$R=Ct^n$$

where R = insulation fire resistance

t = thickness

C,n = constants determined from least squares regression analysis

For concretes containing different aggregate types differing values for the constants C and n are derived from least squares regression.

The correlation could only be directly applied to the results from the tests on the high strength concrete, as only two different data points were available for the lightweight concrete. The curve for the lightweight concrete was derived from comparison with the curves produced by Wade (1992), shown in Figure 4.1. Figure 3.24 shows the curve that was derived for the lightweight and high strength concretes.

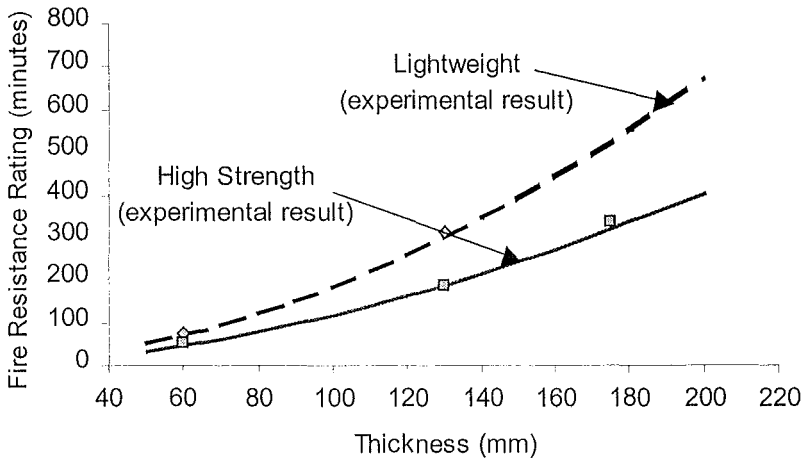


Figure 3.24: Variation of insulation fire resistance with aggregate type and nominal thickness for the lightweight and high strength concretes

Due to the large variations in the size of the 95% confidence interval for the corrected values of time to reach a 140°C temperature rise on the unexposed surface the corrected mean values were used to fit the curve. The constants derived from the curve fit were used to approximate the thickness at given fire exposure periods listed in Table 3.5.

Table 3.5: Values of C & n determined by least square regression.

Aggregate Type	Constants	
	C	n
Lightweight (estimated)	0.034	1.870
High strength	0.032	1.780

4. Discussion

4.1. Experimental Results

Relationship Between Thickness and Insulation Fire Resistance

Figure 4.1 shows the comparison of the curves derived from the experimental results for the lightweight and high strength concretes with those obtained by the initial BRANZ tests (Wade 1992).

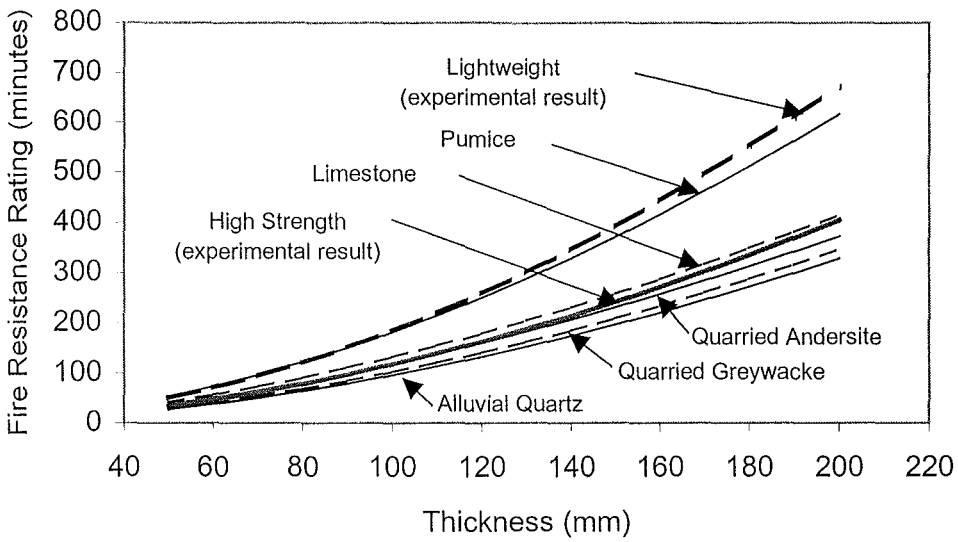


Figure 4.1: Thickness of a concrete element to provide insulation fire resistance made from New Zealand aggregate. (Wade, 1992)

As can be seen from the graph the lightweight concrete provides the greatest insulation fire protection when compared to other concretes types. Lightweight concretes have always been noted for their high insulating properties, and these results provide additional experimental data to support this.

The high strength concrete also performed well, providing better fire insulation protection than the equivalent ordinary concrete (quarried greywacke). This is due to the increased amount of water held inside the cement matrix by the higher cement content of the high strength concrete. The cement contents were 535kg and 295kg for

the high strength and ordinary concretes respectively. This additional amount of water held in the cement matrix increases the amount of energy required to heat the concrete.

Comparison of Experimental Results with NZS3101 (1995)

The New Zealand Standard NZS3101 specifies minimum thicknesses required to provide a level of insulation fire resistance. Figures 4.2 and 4.3 show a graphical comparison of the minimum thickness requirements of NZS3101 with curves derived from the experimental results.

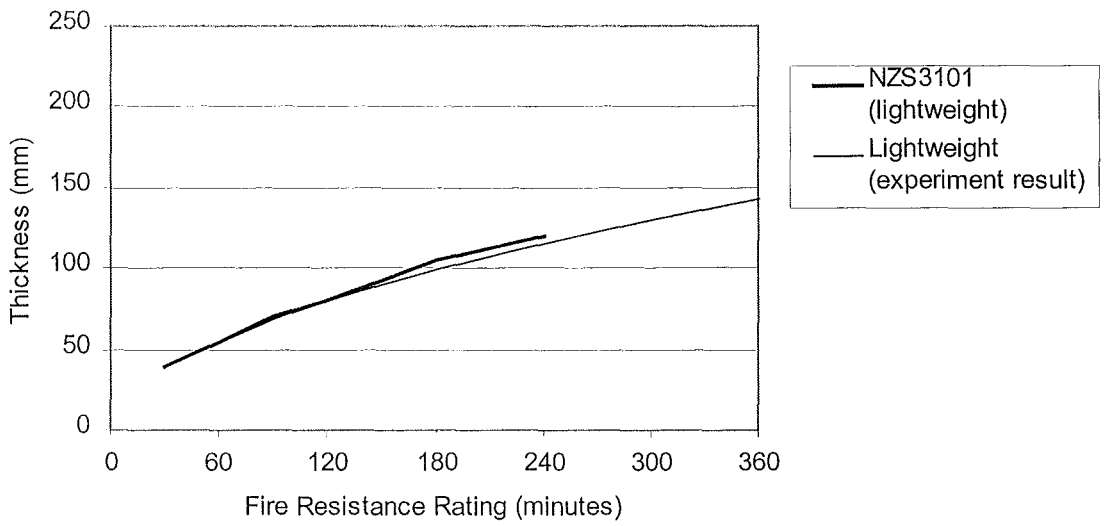


Figure 4.2: Comparison of minimum thickness requirement for lightweight concrete from NZS3101 with the experimental results

Figure 4.2 shows that the current minimum thickness requirements for insulation fire resistance from NZS3101 are comparable to the curve obtained experimentally for lightweight concrete.

Figure 4.3 shows the comparison of the results for high strength concrete with the values for ordinary siliceous concrete from NZS3101. This was selected because NZS3101 does not make a differentiation between ordinary and high strength concrete. The values for siliceous concrete were chosen for comparison as the high strength concrete used greywacke (siliceous) aggregate. The graph shows that the high strength concrete performs better than ordinary siliceous concrete due to the

increased amount of water held in the cement matrix by the higher cement content, as mentioned in section 2.3.

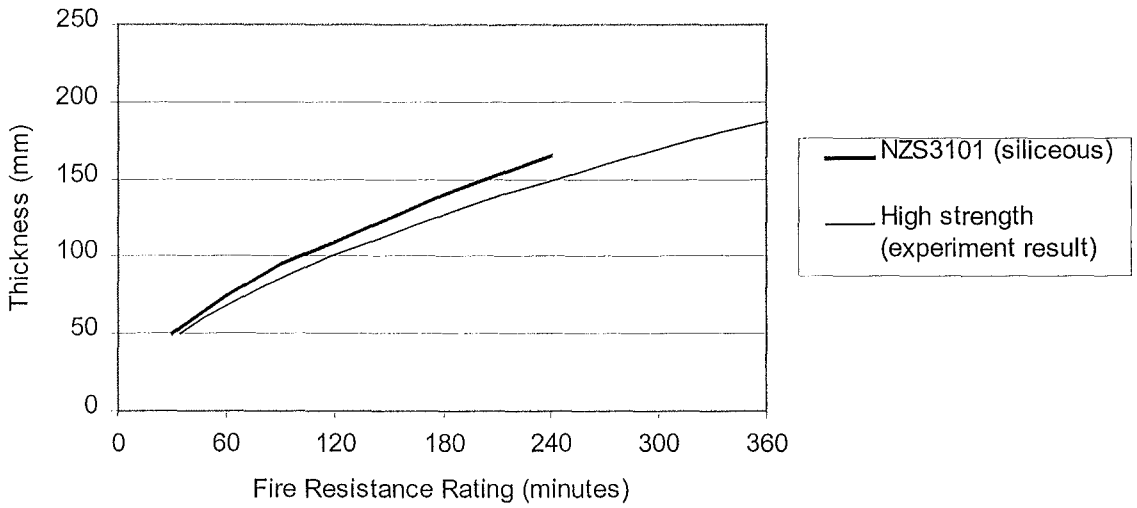


Figure 4.3: Comparison of the minimum thickness requirements for siliceous concrete from NZS3101 with the experimental results for high strength concrete.

Spalling and Effect of Polypropylene Fibres

From the 12 slabs that were tested most showed some minor spalling in the way of blistering or aggregate splitting, as shown in Figure 4.4. Only the 175mm HS plain concrete slab lost a substantial area of concrete that would be considered to effect the insulation of the slab. This is shown in Figure 4.5.

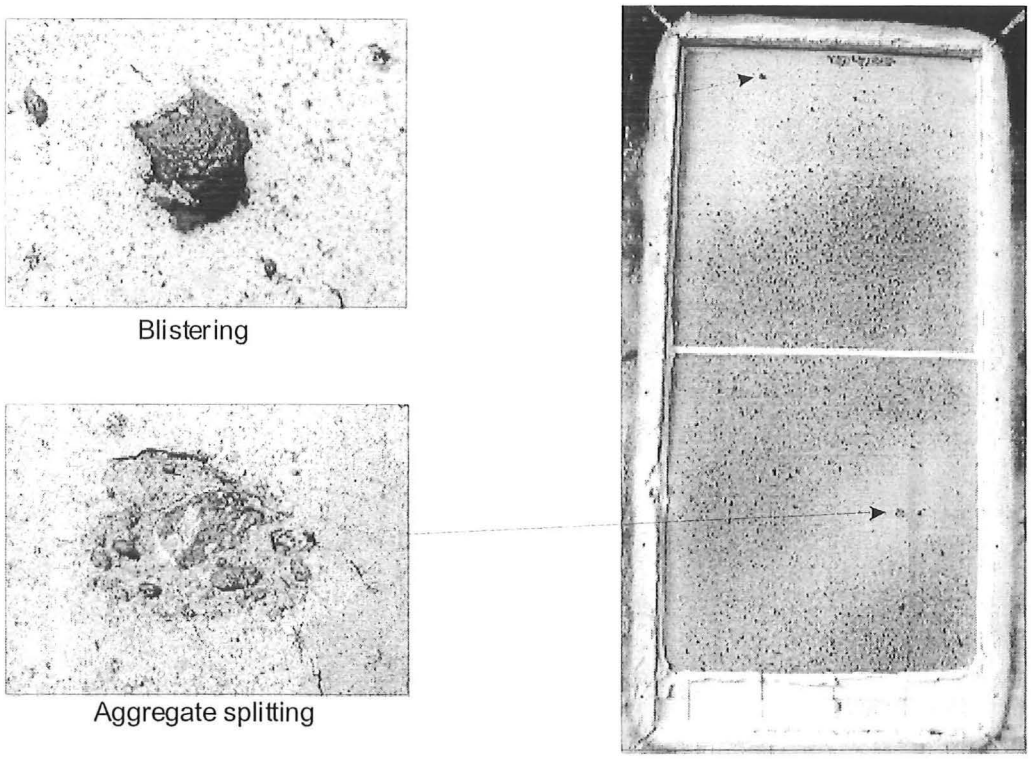


Figure 4.4: Blistering and aggregate splitting on exposed face of 130mm lightweight slab

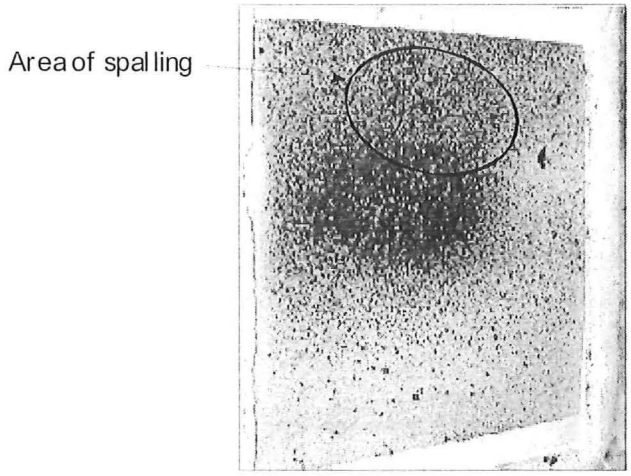


Figure 4.5: Spalling from exposed face of 175mm high strength slab

The time and progression of the spalling that occurred from the face of the 175mm plain high strength slab is unknown. From inspection of the area of spalling no split aggregate was found, suggesting that aggregate splitting was not the cause of the spalling. This suggests excessive pore pressures caused the spalling from evaporation

of moisture inside the slab, as no external loads were applied to the slab during testing.

The polypropylene fibre admix was the only variation between the pair of 175mm high strength slabs. As no significant spalling occurred from the slab with the polypropylene fibres, they may be considered to have contributed to the control of possible spalling. With only one incidence of significant spalling it cannot be proven from these tests that the polypropylene fibres can control the occurrence and severity of spalling.

Effective Thickness of Dimond HiBond Ribbed Floor Slabs

There are several methods currently in use for determining the effective thickness of a ribbed concrete element such as a floor slab. These are listed below in Figure 4.6.

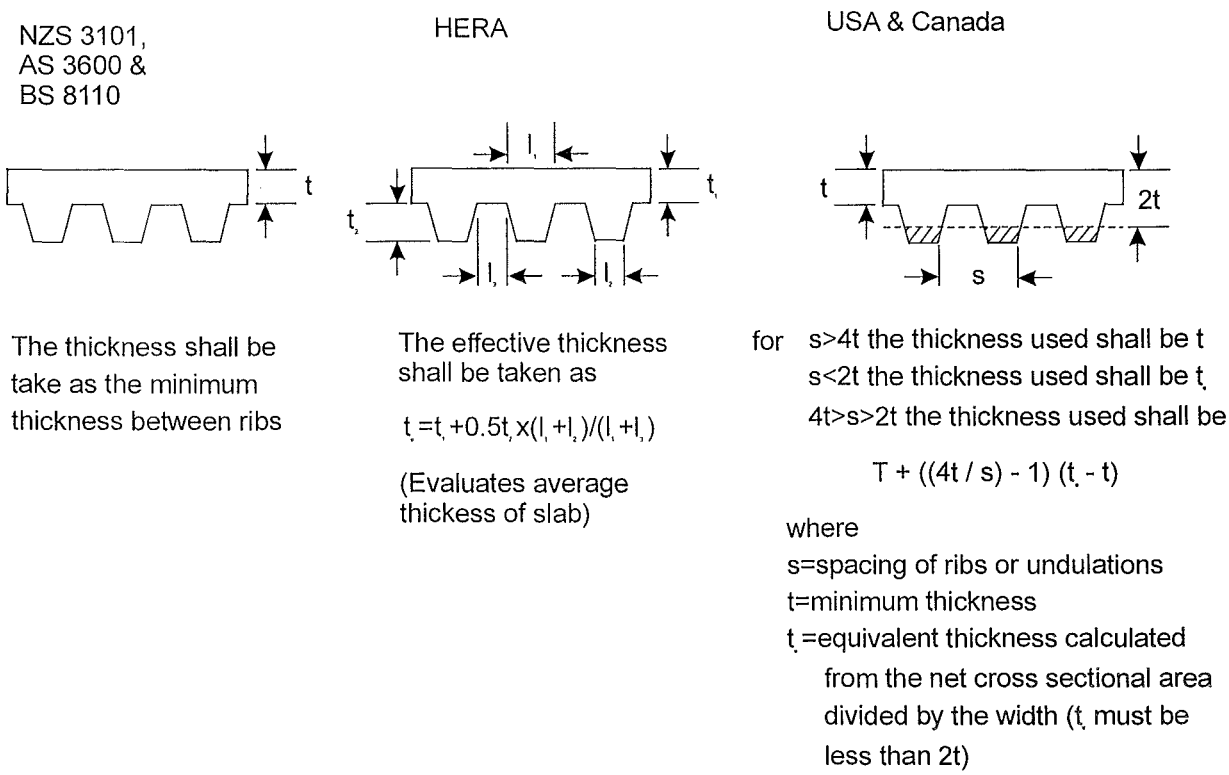


Figure 4.6: Method for calculating the effective thickness of a ribbed concrete element.

The test found that a 130mm thick Dimond Hi-Bond ribbed floor slab has an insulation fire resistance of 85 minutes. Figure 4.7 shows that an 85mm slab of

uniform thickness is required to obtain an insulation fire resistance of 85 minutes for ordinary siliceous concrete. This is the experimentally determined effective thickness of the 130mm thick Dimond Hi-Bond ribbed floor slab.

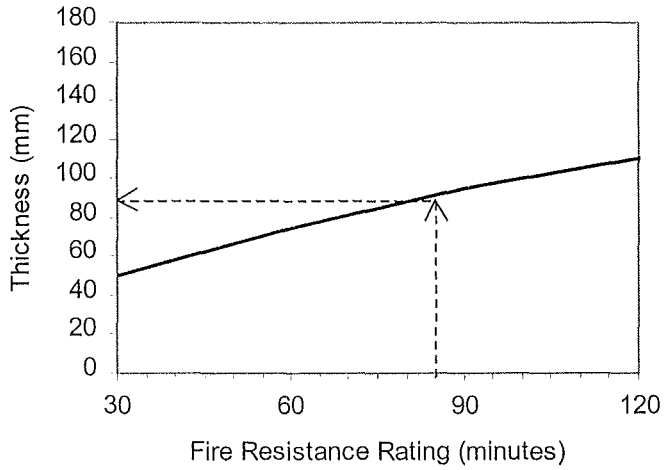


Figure 4.7: Evaluating equivalent thickness of Dimond HiBond from NZS3101 thickness requirements for siliceous concrete to provide insulation fire resistance.

Table 4.1 lists the effective thickness evaluated from each of the methods mentioned and their corresponding insulation fire resistance with those from the test.

Table 4.1: Comparison of estimated effective thickness and insulation fire resistance rating.

Origin	Effective thickness (mm)	Insulation Fire Resistance Rating (min)
Experimental	85	84
NZS 3101	75	60
US/Canadian	75	60
HERA	103	106

As can be seen from Table 4.1 the method from HERA considerably over predicts the effective thickness of the 130mm Dimond Hi-Bond ribbed slab. This corresponds to a higher insulation fire resistance, well above that found from the test.

Both the NZS and US/Canadian method give adequately conservative values.

5. Conclusions

5.1. General Conclusions

The following points summarise the conclusions and experimental findings on the applicability of the existing recommended values and minimum requirements given in NZS3101 to lightweight and high strength concrete.

Conclusions from Existing Literature

- The strength and elastic modulus curves given in NZS3101 can be applied to lightweight concrete.
- The current concrete strength reduction curve given in NZS3101 can not be applied to high strength concrete.
- The elastic modulus curve given in NZS3101 can be applied to high strength concrete.

Conclusions from Testing

- The lightweight expanded clay aggregate produced by Firth showed excellent behaviour in fire conditions.
- The values given in NZS3101 for insulation fire resistance can be applied to high strength and lightweight concrete.
- Using the minimum thickness of a ribbed slab to evaluate the insulation fire resistance, as defined by in NZS3101, gives conservative values.
- The HERA method for defining the effective thickness of a ribbed slab over-predicts the insulation fire resistance.
- No serious spalling problem is apparent in New Zealand high strength and lightweight concrete.
- Polypropylene fibres do appear to have some effect in minimising the occurrence and severity of spalling. The finding of overseas researchers supports this.

5.2. Recommendations and Further Work

From the findings of this project the following additional research and recommendations are suggested.

- Produce a strength reduction curve for New Zealand high strength concrete at elevated temperatures to be integrated into NZS3101.
- Develop a user friendly heat transfer computer package for solid and ribbed concrete slabs.
- Produce a more effective method for estimating the effective thickness of ribbed slabs.

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Appendix A

Comparison of New Zealand Standard NZS3101 with;

Australia (AS3600, 1994)

Europe (ENV 1992-1-2, 1995)

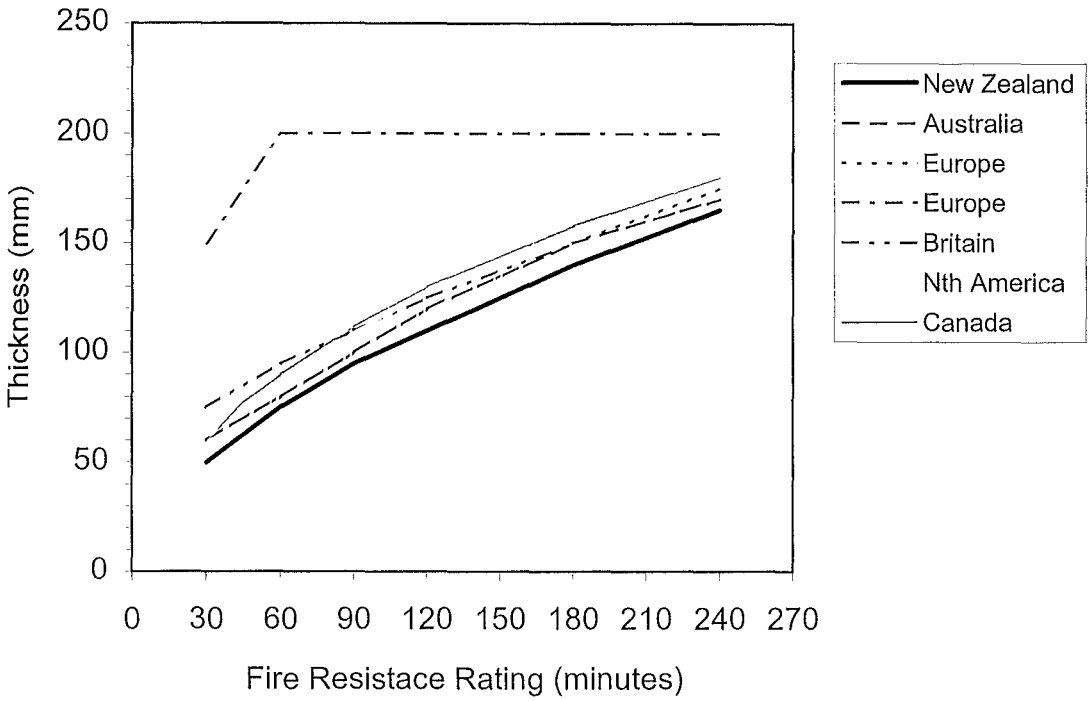
Britain (BS8110, 1985)

North America (UBC, 1997)

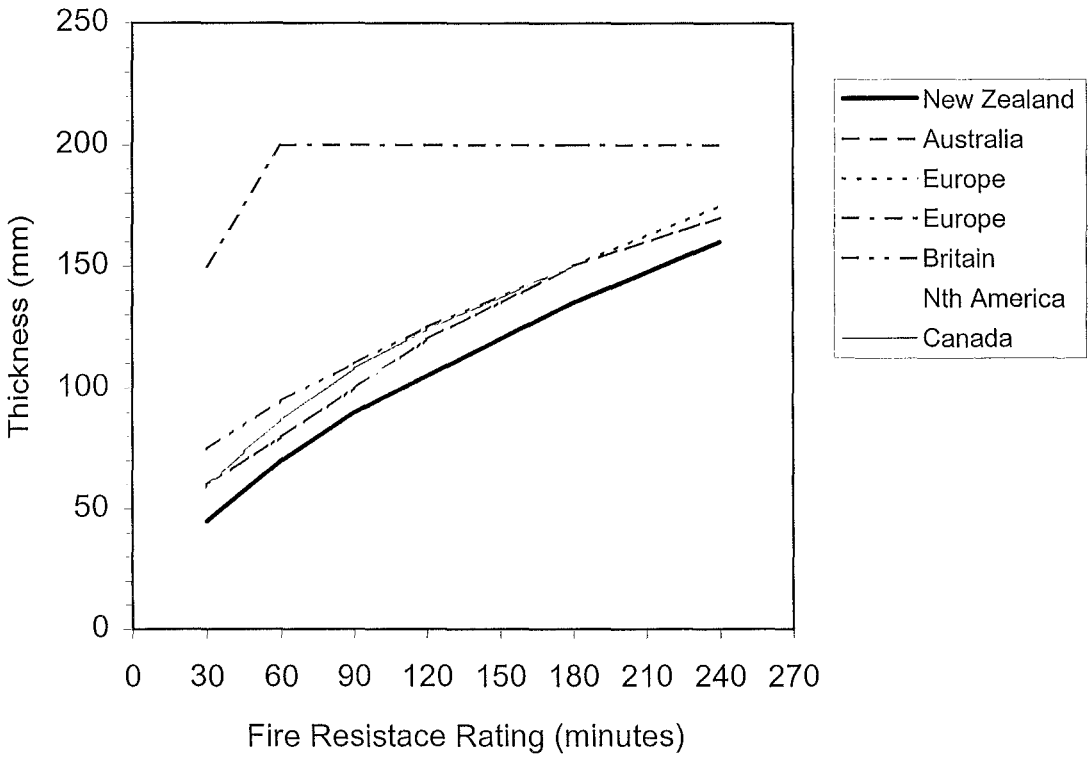
Canada (NBCC, 1995)

in regard to minimum thickness to provide insulation fire resistance.

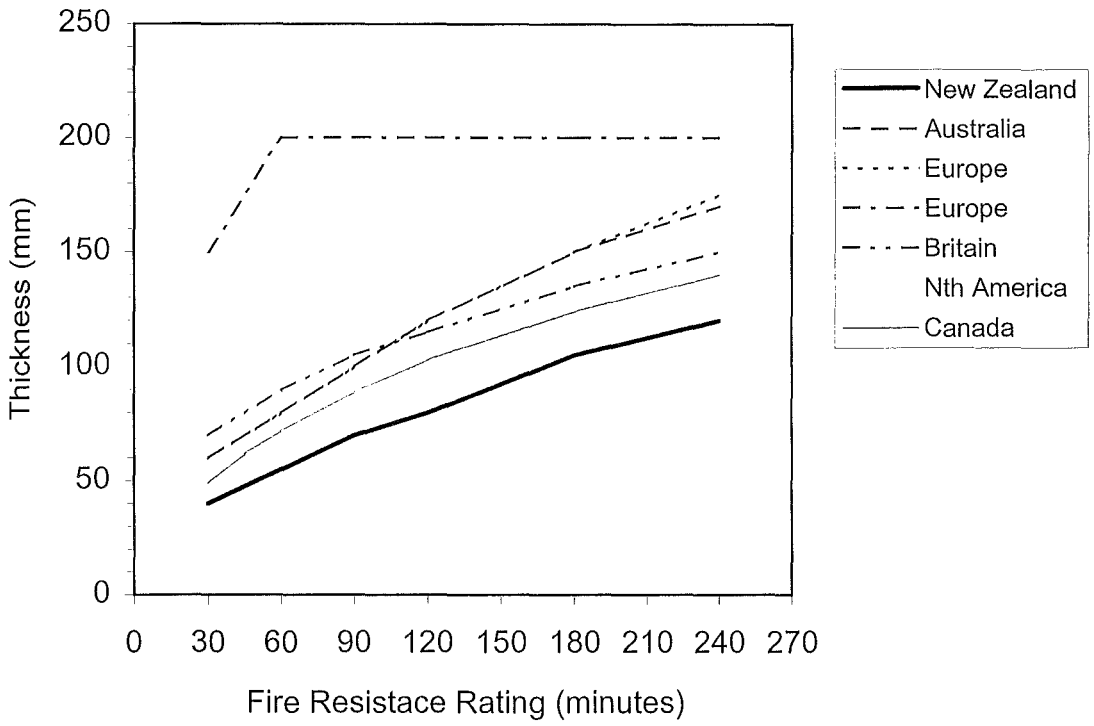
Plain Floor Slab, Siliceous



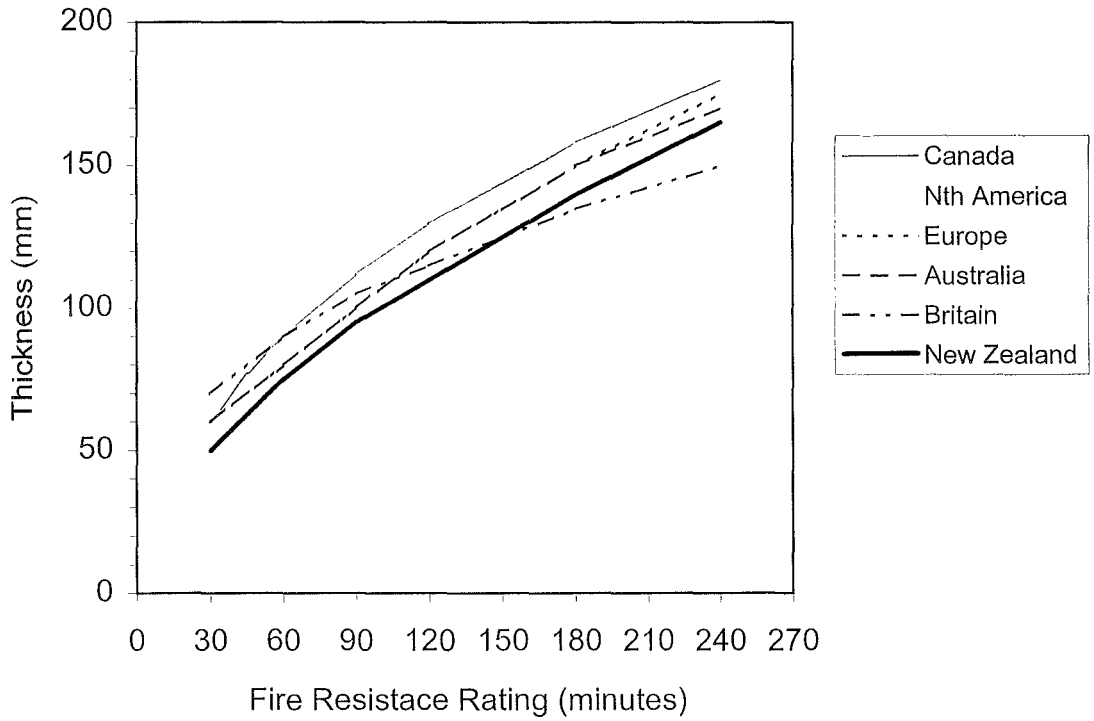
Plain Floor Slab, Calcareous

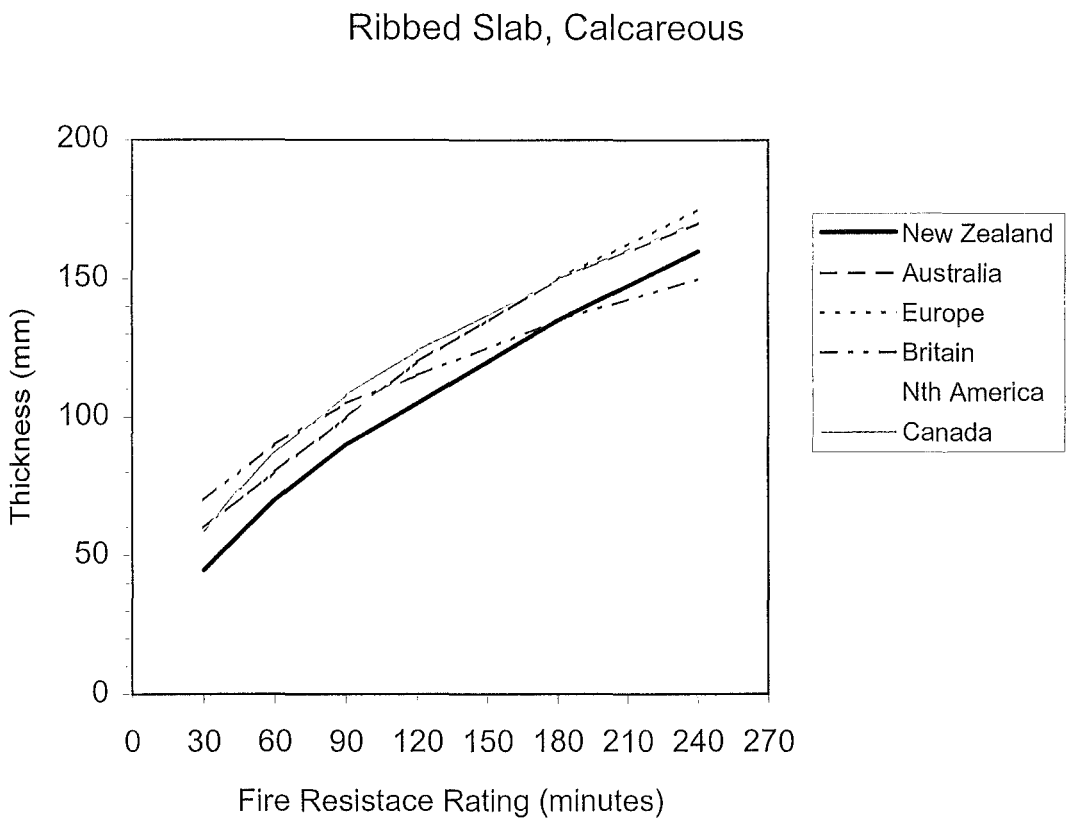


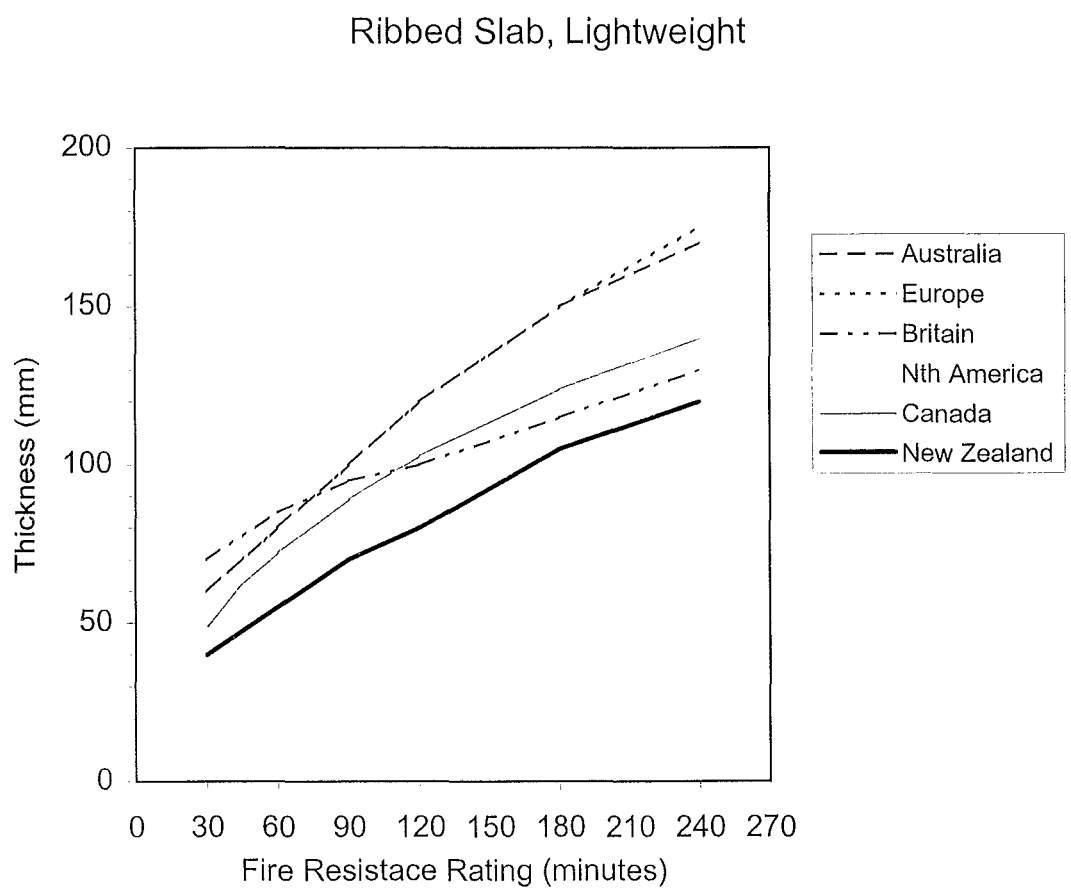
Plain Floor Slab, Lightweight



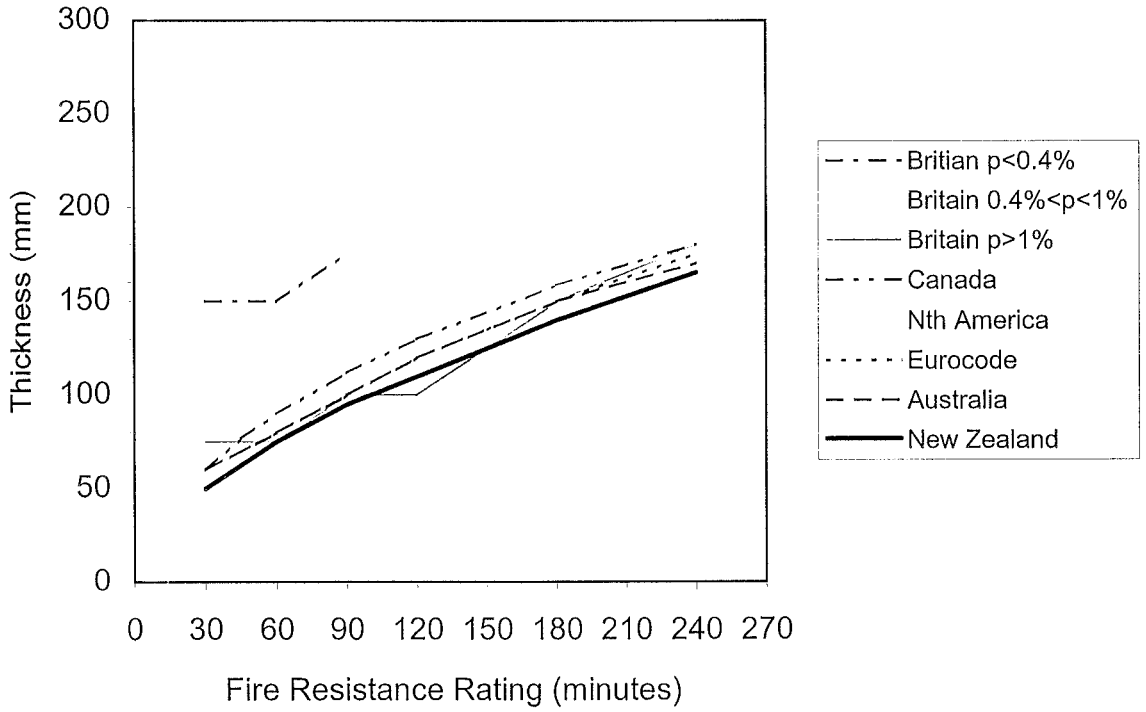
Ribbed Floor Slab, Siliceous



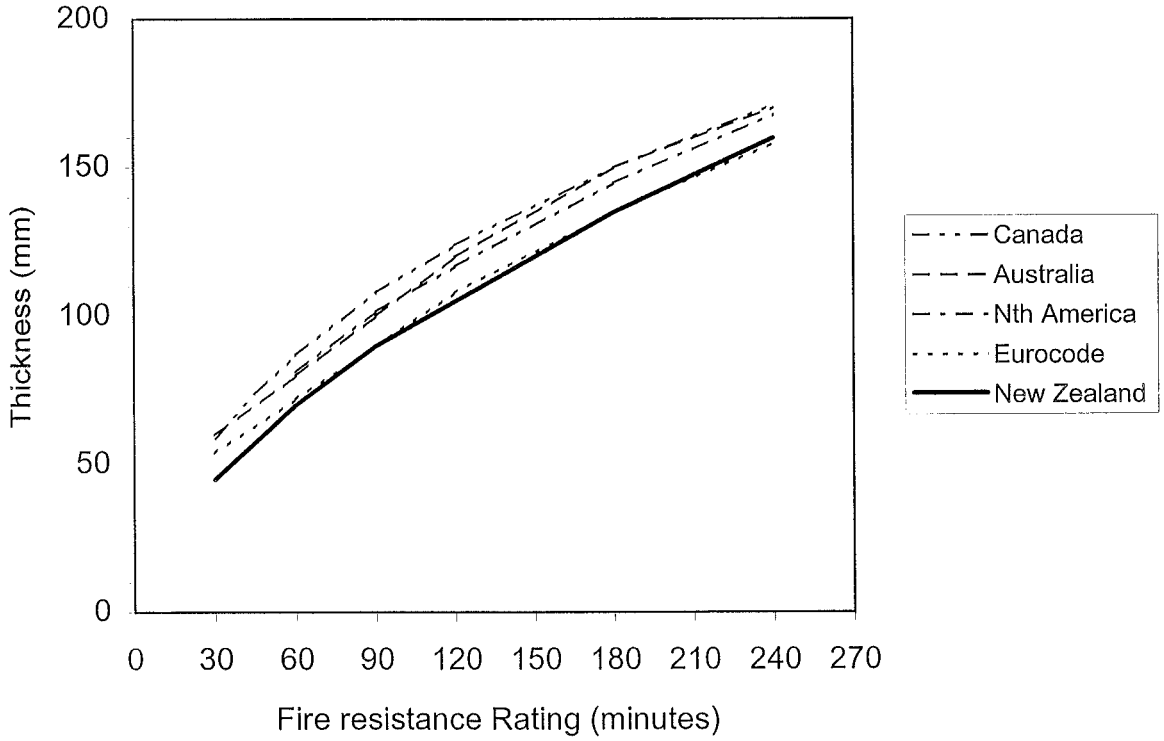

Ribbed Slab, Calcareous
200
150
100
50
0
Thickness (mm)
0 30 60 90 120 150 180 210 240 270
Fire Resistace Rating (minutes)
New Zealand
Australia
Europe
Britain
Nth America
Canada

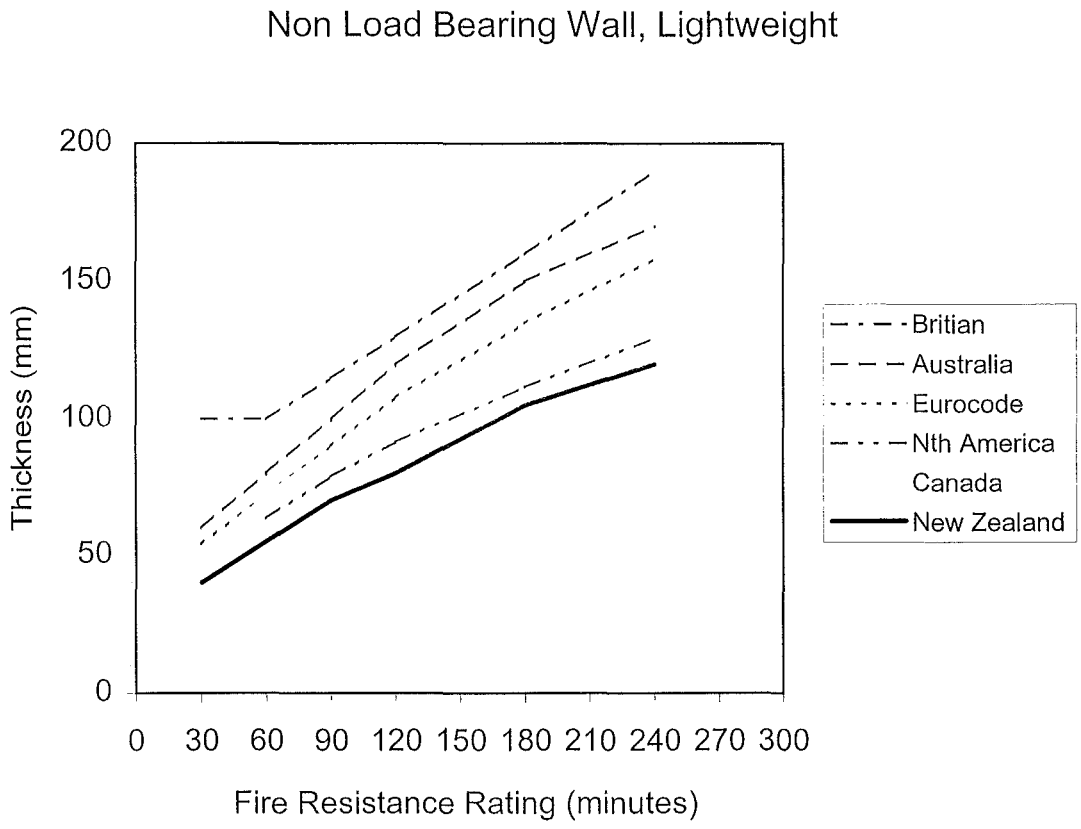

Ribbed Slab, Lightweight
200
150
100
50
0
Thickness (mm)
0 30 60 90 120 150 180 210 240 270
Fire Resistace Rating (minutes)
Australia
Europe
Britain
Nth America
Canada
New Zealand

Non Load Bearing Wall, Siliceous



Non Load Bearing Wall, Calcareous




Non Load Bearing Wall, Lightweight
200
150
100
50
0
Thickness (mm)
0 30 60 90 120 150 180 210 240 270 300
Fire Resistance Rating (minutes)
Britian
Australia
Eurocode
Nth America
Canada
New Zealand

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97/8	Uncertainty Analysis of Zone Fire Models	A M Walker
97/9	New Zealand Building Regulations Five Years Later	T M Pastore
98/1	The Impact of Post-Earthquake Fire on the Built Urban Environment	R Botting
98/2	Full Scale Testing of Fire Suppression Agents on Unshielded Fires	M J Dunn
98/3	Full Scale Testing of Fire Suppression Agents on Shielded Fires	N Gravestock
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98/5	Comparison Studies of Zone and CFD Fire Simulations	A Lovatt
98/6	Bench Scale Testing of Light Timber Frame Walls	P Olsson
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