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In situ measurement of the effect of partial Portland cement replacement using either fly ash or ground granulated blast-furnace slag on the performance of mass concrete

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The Paper presents the results of an investigation to monitor the effect of partially replacing Portland cement with either ground granulated blast-furnace slag or fly ash on the performance of massive concrete pours. Temperature and strain were monitored in three pours each 4·5 m deep which form part of the foundations for a grinding mill at the Frodingham Cement Works in Scunthorpe. The total cementitious material content in each pour was 400 kg/m^3. In one case 75% of the OPC was replaced by granulated slag and in another, 30% was replaced by fly ash. Strengths were measured under British Standard and temperature matched curing conditions to observe the effect of the in situ heat cycle. To enable a theoretical assessment of thermal stresses at early age to be made, laboratory tests on similar concrete were carried out to determine adiabatic temperature rise, thermal movement, elastic modulus and creep characteristics. Results of the laboratory and site investigations are assessed in relation to existing data.

Introduction

Background

In November 1976 the Taylor Woodrow Research Laboratories started a programme of testing on behalf of the Building Research Establishment to examine the performance of concretes containing either fly ash or ground granulated blast-furnace slag as partial Portland cement replacements. For some time there had been debate as to the effectiveness of these materials in reducing the temperature rise resulting from heat of hydration and hence the likelihood of cracking in massive concrete pours. In situ temperatures[1–8] in concrete pours up to 3·75 m deep with total cement contents of 300–465 kg/m^3 indicated that a reduction in temperature rise could be achieved. However, one school of thought maintained that all cements, including those containing granulated slag or fly ash, should be treated alike when considering problems associated with heat of hydration.[9,10] In particular there was doubt as to whether, in larger pours with higher cement contents, the effect of partial Portland cement replacement would be as significant as that monitored in relatively smaller pours.

2. Shortly after starting the laboratory programme, which primarily involved

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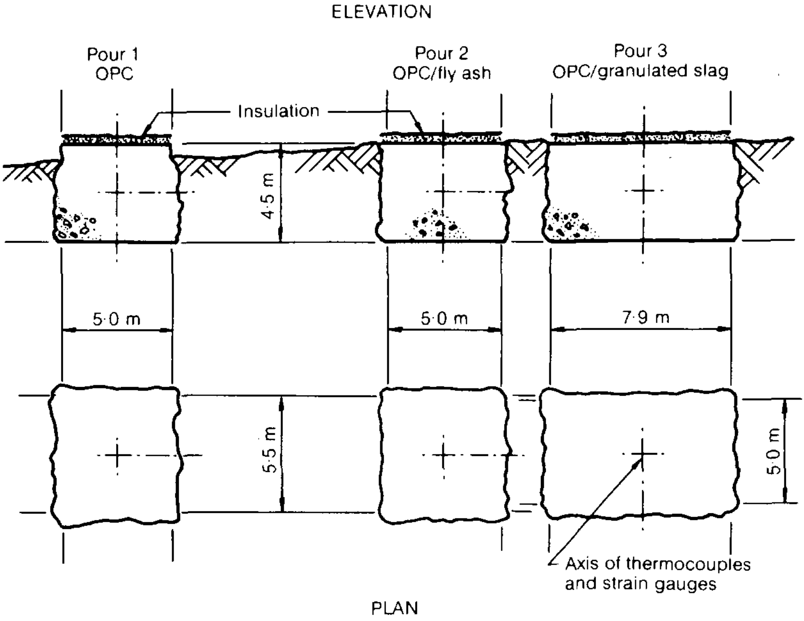

Fig. 1. General layout of instrumented pours and nominal pour dimensions

the measurement of heat of hydration under adiabatic conditions, strength development and deformation behaviour, the opportunity arose of instrumenting three large concrete pours, each 4·5 m deep, which formed part of the foundations for a new grinding mill at the Frodingham Cement Works at Scunthorpe (Fig. 1). Frodingham Cement Co. Ltd were willing to permit the use of different concretes in each pour, provided that they each met the specification requirements for characteristic cube strength (25 N/mm^2 at 28 days) and workability (75 mm slump). A programme of instrumentation was therefore carried out as part of the Building Research Establishment extra-mural contract, in association with Frodingham Cement Co. Ltd and Pozzolanic Ltd.

3. Three concretes were examined: a control mix with a Portland cement content of 400 kg/m^3, a mix with 75% of the OPC replaced by Cemsave,[11]—a ground granulated blast-furnace slag—and a mix with 30% of the OPC replaced by Pozzolan[12]—a selected fly ash. The high cement content in relation to the strength requirement was chosen in order to provide data in an area which had not previously been investigated (i.e. cement-rich structural concrete) and in order to be compatible with concretes already being examined as part of the laboratory programme.[13]

4. In addition to monitoring temperature variation during the early age heat cycle, movements were monitored through the depth of each pour. Although measurement of temperature gradients can give a rule of thumb indication as to the proneness of cracks in concrete,[9,10] it is relative strain changes and associated thermal stresses which if excessive, cause cracking.

Objectives

5. The site instrumention was carried out primarily to observe the effect of either fly ash or granulated slag on the in situ performance of mass concrete. In particular, the following factors were examined

(*a*) the rate of hydration in terms of temperature development
(*b*) the peak temperature occurring at the centre of each pour
(*c*) the magnitude of temperature differentials
(*d*) thermal movements and differential thermal strains
(*e*) the effect of temperature cycling on concrete strength development.

It was thus hoped that many of the differences of opinion related to this area of concrete technology could be resolved.

Construction details

Concrete

6. The volumes of concrete cast into each pour are given in Table 1, together with the mix proportions and details of the materials used. The cementitious materials were chemically analysed and the compound phase composition for the OPC was calculated (Table 2). The high C_3A content (13·6%) indicated that a substantial early heat evolution could be expected from the OPC.

Concrete supply

7. Dry batched ready-mix concrete was used. The degree of variation in workability was similar for all three mixes (Table 3). Concrete was discharged by chute directly into each pour.

Table 1. Details of concrete mix proportions and materials used

Material	Mix proportions, kg/m^3, saturated surface dry weights		
	Pour 1	Pour 2	Pour 3
Portland cement: Rugby OPC from South Ferriby Works	400	280	100
Quality controlled fly ash: Pozzolan from West Burton Power Station (Pozzolanic Ltd)	—	120	—
Ground granulated blast furnace slag: Cemsave from Appleby Steel Works (Frodingham Cement Co. Ltd)	—	—	300
Coarse aggregate: 20–5 mm irregular gravel from Steetley–Denniff (Whisby) and Redland (Besthorpe and North Scar) quarries	1260	1235	1250
Fine aggregate: Zone 3 natural sand Steetley–Denniff and Redland quarries	615	620	605
Water: Mains supply	180	165	165
Volume of concrete placed, m^3	144	147	212

Table 2. Chemical analysis of OPC, granulated slag and fly ash

	OPC	Granulated slag	Fly ash
Silica, SiO_2	20·20	34·65	49·10
Alumina, Al_2O_3	6·73	13·62	28·10
Ferric oxide, Fe_2O_3	2·50	0·28	9·50
Calcium oxide, CaO	64·58	38·36	1·80
Magnesium oxide, MgO	0·91	10·15	1·70
Sulphate, SO_3	2·93	0·23	0·30
Sodium, Na_2O		0·44	1·80
Potassium, K_2O		0·81	3·70
Titanium, T_1O_2	0·26	0·42	1·00
Loss on ignition	1·01	−1·2 (gain)	3·0
Potential phase composition			
C_3S	52·2%		
C_2S	18·6%		
C_3A	13·6%		
C_4AF	7·6%		

Table 3. Details of samples taken during casting

Volume of concrete delivered, m^3	Number of cubes cast		Slump, mm	Sample taken by
	BS curing	Temperature matched curing		
Pour 1				
36	3	—	70	Frodingham Cement Co.
78	7	10	70	Frodingham Cement Co.
78	3	—	125	Steetley-Denniff
120	3	—	60	Steetley-Denniff
144	11	—	120	Frodingham Cement Co.
Pour 2				
36	8	—	70	Frodingham Cement Co.
54	3	—	75	Steetley-Denniff
85	21	15	80	Pozzolanic Ltd
91	8	—	100	Frodingham Cement Co.
108	3	—	60	Steetley-Denniff
120	8	—	50	Frodingham Cement Co.
Pour 3				
42	8	—	55	Frodingham Cement Co.
96	3	—	80	Steetley-Denniff
108	—	12	70	Frodingham Cement Co.
144	8	—	40	Frodingham Cement Co.
174	24	—	85	Frodingham Cement Co.
174	3	—	65	Steetley-Denniff
198	8	—	50	Frodingham Cement Co.

8. For pour 2 the supply of fly ash ran out after placing 129 m^3. The rest of the pour (18 m^3) was completed with an OPC concrete with modified mix proportions (375 kg/m^3 OPC, 675 kg/m^3 sand, 1275 kg/m^3 gravel and 100 kg/m^3 water).

9. The pours were cast on three successive days to minimize the effects of ambient temperature changes and materials variation. The weather remained stable for three days and concrete mix temperatures were generally about 20 ± 2°C.

Insulation

10. To reduce excessive thermal gradients, insulation in the form of a fine foamed slag/air-cooled slag blend was spread on to heavy duty polythene laid on the concrete. The average depth of insulation was 100 mm, applied in each case at about four hours after completion of casting.

Sampling

11. Samples of concrete were taken throughout each of the three pours to check the workability (and hence variability) of the fresh concrete and also to produce cubes for the assessment of strength, under standard water curing and temperature matched curing conditions (Table 3). The latter were cured in heated water baths controlled by thermocouples cast into the centre of each pour. The slump was generally in the range 40–125 mm.

In situ temperature and strain measurement

Instrumentation

12. Copper/Constantan thermocouples were used in conjunction with two 24 channel Honeywell Electronik roll chart recorders to monitor continuously the temperature variation at 15 locations within each pour. Thermal movement was measured at five locations using vibrating wire strain gauges (Fig. 2). A battery-operated strain reader and data logger provided a digital print-out at preselected time intervals.

Temperature rise

13. The temperature of the concrete at placing was recorded at each thermocouple location immediately after being covered with concrete. Concrete placing temperatures were generally of the order of 20°C but recorded values ranged from 16·5°C to 24°C. The higher temperatures generally occurred during the middle of the afternoon when the ambient temperature was high and the constituent materials had been warming up during the earlier period of the day.

14. The early age temperature variation recorded in each pour at mid-height is shown in Fig. 3. The maximum temperature rise recorded at the centre of each pour was

(*a*) pour 1: 54·5°C rise, from placing temperature of 20·5°C
(*b*) pour 2: 47·5°C rise, from placing temperature of 21·5°C
(*c*) pour 3: 46·0°C rise, from placing temperature of 18°C.

15. The effect of the cement replacement materials was in each case to reduce the peak temperature rise by about 8°C (15%). This reduction was small in relation to the observed effect of cement replacement materials in smaller

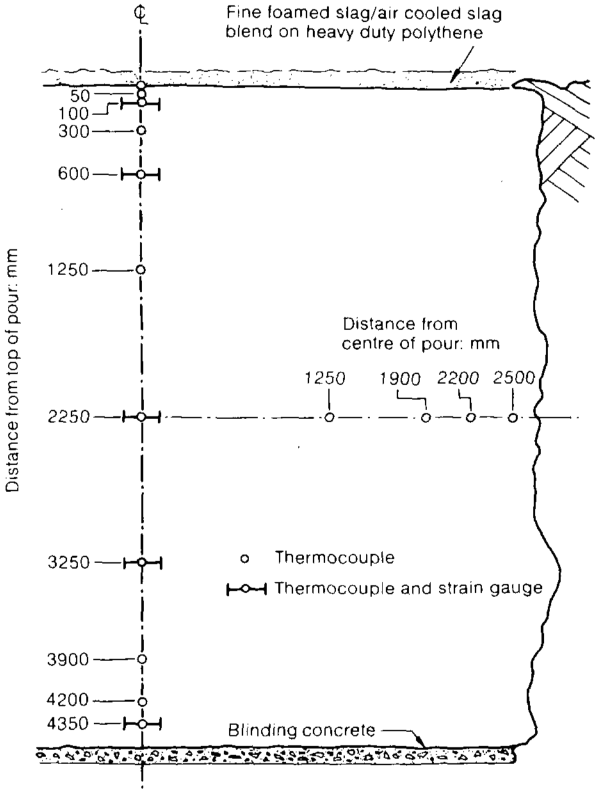

Fig. 2. Location of thermocouples and strain gauges within each pour

pours with lower cement contents.[1–8] The temperature rises per unit weight of cementitious material were 13·6°C/100 kg for OPC, 11·7°C/100 kg for OPC/fly ash and 11·6°C/100 kg for OPC/granulated slag. The value for OPC was higher than the 12°C/100 kg generally quoted for large pours,[9] but this was not totally unexpected in view of the pour size and the chemical composition of the cement.

Relationship with existing data

16. The temperature rise which occurs as a result of hydration in a concrete pour depends on a number of factors, the most significant of which are total cementitious material, type of cement, size of pour, concrete placing temperature and environmental conditions. The interrelationship between these factors is complex, but for general engineering purposes certain assumptions can be made to simplify the prediction of temperature rise in concrete. For example, a survey of data[1, 3,6,8,14] relating to the use of granulated slag has shown that the maximum temperature rise, expressed in °C/100 kg of cementitious material, can be approximately related to the minimum pour dimension and the level of replacement (Fig. 4). As the size of the pour is increased, the effectiveness of the granulated slag in lowering the peak temperature is reduced. However, in pours

about 2 m deep, reductions in temperature rise of 50% have been achieved by a 70% replacement.[1-3]

17. The curves are related to a concrete mix temperature of 10–20°C. In the centre of large masses of concrete which are in an adiabatic condition for, perhaps, several days, variations in the initial placing temperature will have only a marginal effect on the peak temperature rise. In smaller pours, in which the rate of heat evolution is more important in determining temperature rise, the initial mix temperature is of greater significance, and at casting temperatures below 10°C the peak temperature rise will also be reduced.

18. A similar set of curves has been developed for concrete containing fly ash (Fig. 5). The information available is more limited[4,5] but curves for replacement levels up to 30% have been produced. Again the advantage in terms of reduced temperature rise was reduced with increased lift height.

19. It is clear, therefore, from the data in Figs 4 and 5 that Portland cement

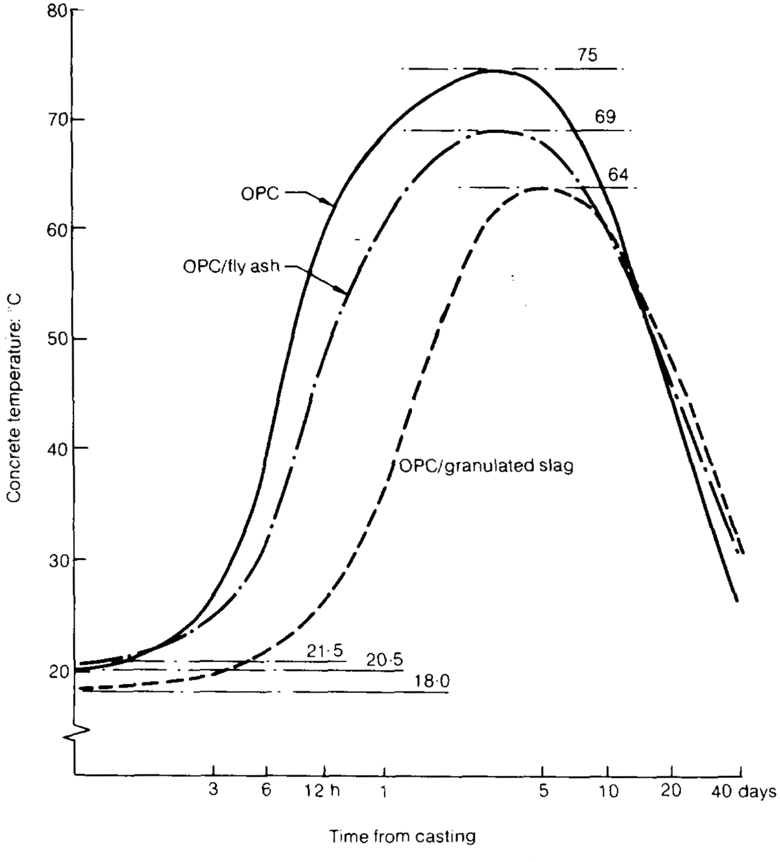

Fig. 3. Variation in concrete temperature recorded at mid-height

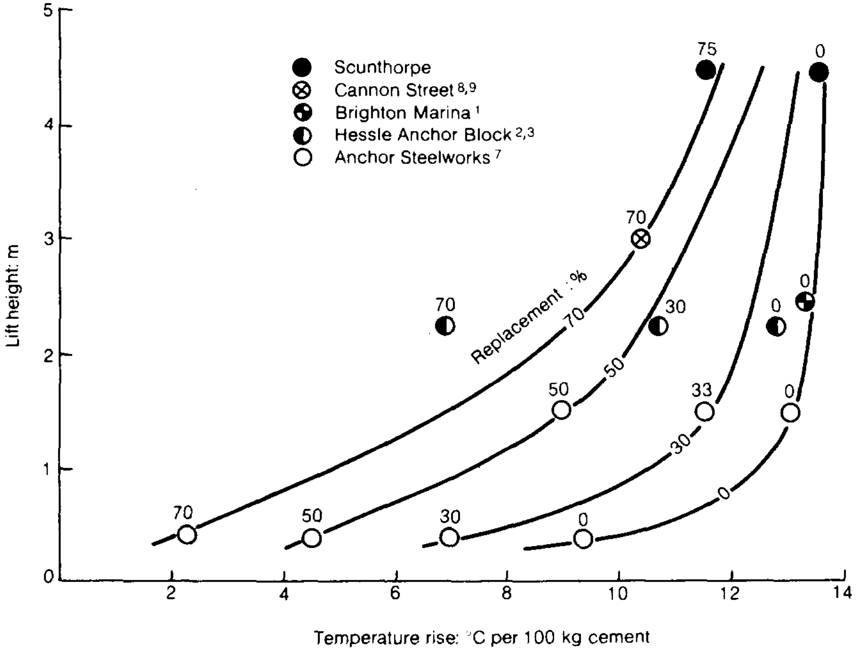

Fig. 4. Relationship between lift height, temperature rise and level of Portland cement replacement using granulated slag

Table 4. Recorded values of temperature differentials between mid-height and the top, side and base of each pour

Pour number	Top			Base	Side
	Before insulating	Four hours after insulating	At maximum temperature	At maximum temperature	At maximum temperature
1	30	20	17	25	17
2	24	9	17	25	14
3	20	12	17	25	10

replacement using either fly ash or granulated slag can be adopted to effect a reduction in the temperature rise which occurs during hydration.

Thermal gradients

20. Temperature distributions through each pour, recorded at the time of maximum temperature rise, are shown in Fig. 6. The application of the insulation 4–5 hours after the completion of concreting resulted in a significant reduction in the magnitude of the temperature differentials to the top surface of each pour (Table 4). Had insulation not been used, differentials approaching 50°C might have been expected. The maximum differentials were in fact between

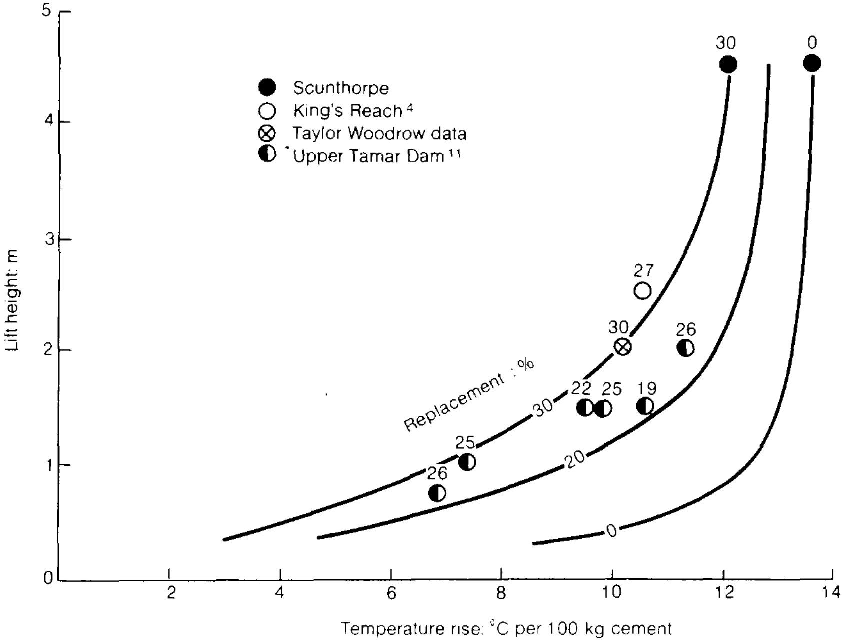

Fig. 5. Relationship between lift height, temperature rise and level of Portland cement replacement using fly ash

mid-height and the base of the pour: about 25°C. It has been recommended[9,10] that, as a rule of thumb, the temperature differential should be kept below 20°C to avoid cracking. Dunstan and Mitchell[5] confirmed this figure; they observed cracking in the Upper Tamar Dam with a temperature differential of 25–26°C. It was reasonable to assume, therefore, that with a 25°C differential recorded in all three pours cracking would have been likely to occur.

Strain measurements in situ

21. Whether or not cracking will occur at a specific location within a concrete pour when subjected to a change in temperature $\Delta\theta$ will depend on the difference between the actual movement which occurs at that point and the potential or free thermal movement associated with the same temperature change. Expressed in terms of the coefficient of thermal movement for the concrete, the stress-inducing strain ε_s (i.e. the strain that, if excessive, can cause cracking) can be calculated from

$$\varepsilon_s = \Delta\theta(\alpha_f - \alpha_a) \qquad (1)$$

where α_f is the free thermal movement coefficient and α_a is the apparent thermal movement coefficient measured in situ.

22. Values of α_a have been obtained from plots of strain against temperature at each gauge location (Fig. 7). The variation of α_a through the depth of each pour can be approximately represented by a parabolic curve enabling the mean

value to be calculated mathematically (Fig. 8). If the general equation for the parabola is

$$\alpha_a = (h^2/a) + b \qquad (2)$$

where h is the distance from the mid-height in metres, and a and b are constants, then the mean value of α_a (in units of microstrain per degree centigrade) can be calculated using

$$\alpha_{a(mean)} = (H^2/3a) + b \qquad (3)$$

where H is half the depth of the pour.

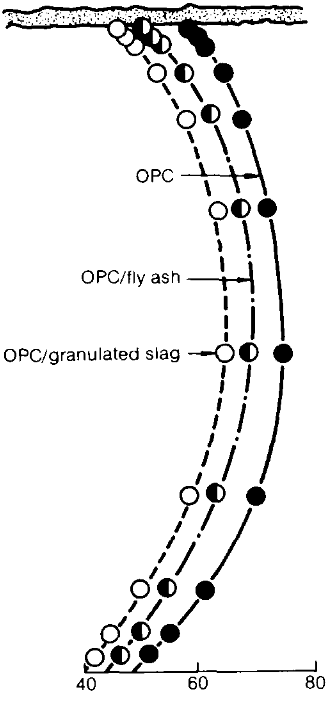

Fig. 6. Vertical temperature distributions recorded at the time of maximum temperature rise

Table 5

Pour	a	b	$\alpha_{a(mean)}$
1	1·53	10·0	11·1
2	1·05	10·5	12·1
3	1·05	10·1	11·7

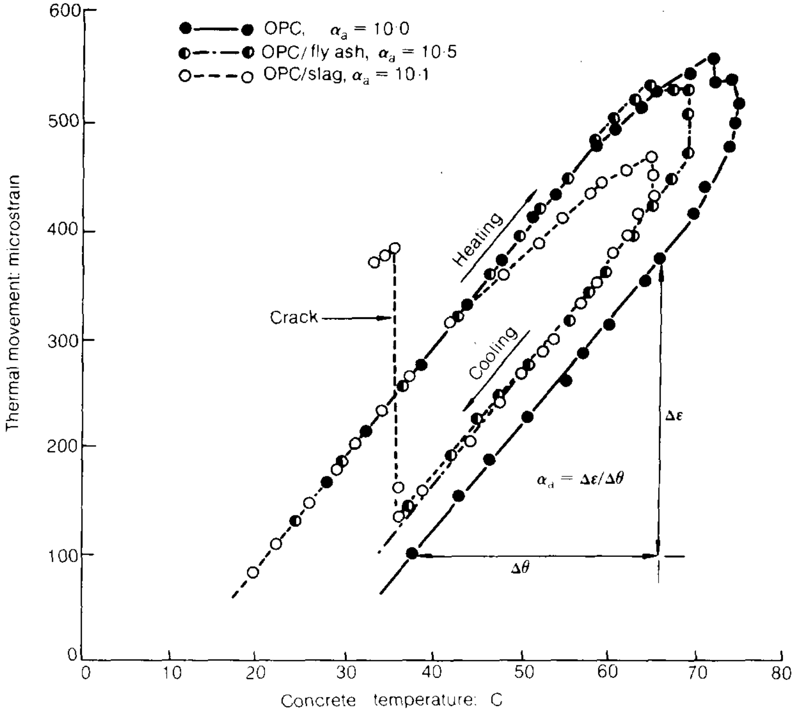

Fig. 7. Relationship between strain and temperature recorded at mid-height

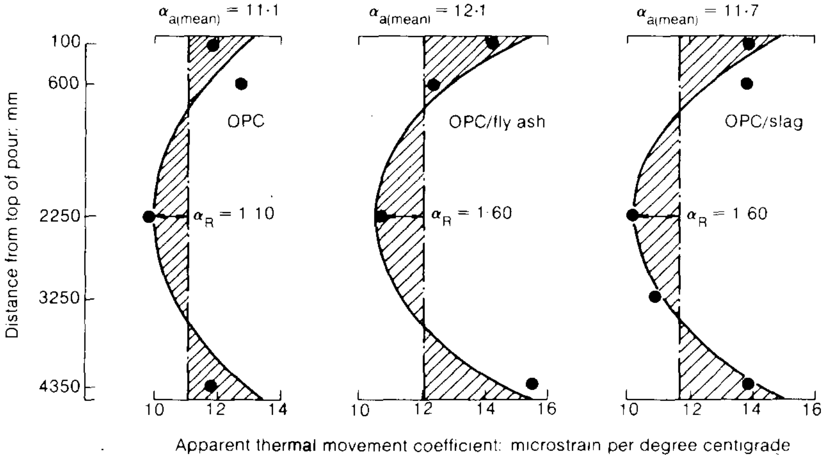

Fig. 8. Variation of the coefficient of thermal movement measured in situ

23. Values of a, b and $\alpha_{a(mean)}$ were as shown in Table 5. The value of $\alpha_{a(mean)}$ in a pour with no external restraint represents the free thermal movement coefficient of the concrete. Values obtained for pours 2 and 3 were in fact close to the value of 12 microstrain per degree centigrade normally expected for a flint gravel aggregate concrete;[15] the value obtained for pour 1 was marginally lower.

24. The variation of α_a within each pour was due to the vertical temperature variation, the central core being subjected to the greater temperature rise, thus imposing movement on the cooler surface regions. By action and reaction the core would therefore be subject to some restraint, the level of which has been calculated in terms of the stress-inducing or restrained strain (equation (1)) to provide a quantitative assessment of the likelihood of cracking (Table 6). The value of ε_s in pour 1 was relatively low in relation to the tensile strain capacity of concrete; values of strain capacity of 65–75 microstrain have been measured for flint gravel aggregate concrete at an early age[16–18] and cracking would not, therefore, have been expected to occur, despite the 25°C temperature differential recorded. However, in pours 2 and 3 tensile strains of 78 and 74 microstrain were recorded, indicating that the strain capacity may have been exceeded and that cracking was likely to occur.

25. A vertical hairline crack was detected in pour 3 at mid-height and developed to a width of 0·03 mm. This was not considered to be structurally important in view of the nature of the pour and self-healing as a result of continued hydration is likely to have occurred.[19] It was surprising that cracking occurred in pour 3, in which a high level of Portland cement replacement was used to reduce the temperature rise, whereas no cracking was detected in pour 1. An explanation of this phenomenon became apparent as results became available during the laboratory test programme.

Load deformation

26. As part of the laboratory programme three concretes were tested to measure early age and long-term load deformation behaviour. The concretes tested had mix proportions similar to those used on site. The tests were carried out on sealed cylindrical specimens 150 mm in diameter and 450 mm long which were heat-cycled during their early life to ensure that the laboratory concrete represented as closely as possible the concrete in a massive structure. The imposed heat cycles are shown in Fig. 9 and are representative of the temperatures likely to occur in a 2·5 m deep pour. This depth was selected

Table 6. Values of potential stress-inducing strain, developed at the centre of each pour during cool-down

Pour number	Thermal movement coefficient microstrain/°C			Temperature change $\Delta\theta$	Stress-inducing strain $\alpha_R \times \Delta\theta$, ε_s microstrain
	Free α_f	Actual α_a	Restrained α_r		
1	11·1	10·0	1·1	54·5	60
2	12·1	10·5	1·6	47·0	78
3	11·70	10·1	1·6	46·5	74

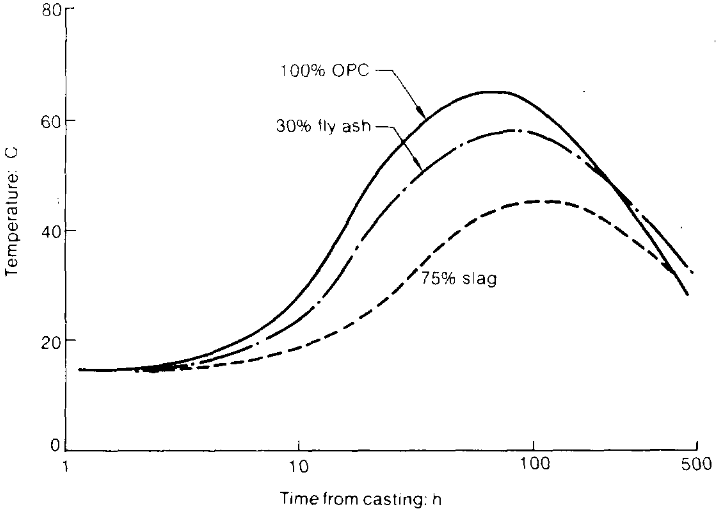

Fig. 9. Temperature cycles imposed on laboratory test specimens used for the measurement of elastic and creep strain

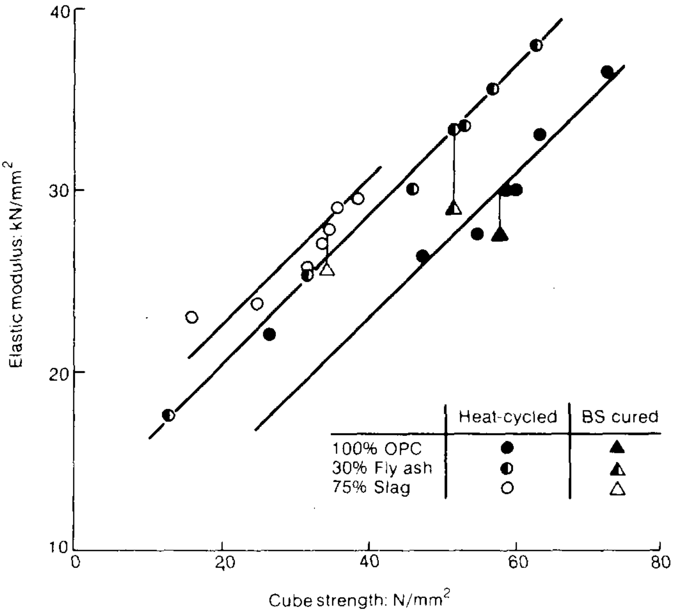

Fig. 10. Relationship between elastic modulus and strength for concretes with and without Portland cement replacement

before details of the pours at Scunthorpe were known and as being more typical of general site practice. Specimens were loaded at ages from one day to one year.

Elastic modulus

27. The development of elastic modulus varied for the three concretes and would perhaps have been expected to be related to the development of strength. However, the relationship between modulus and strength indicates that the concretes with OPC replacement yielded a higher modulus in relation to strength (Fig. 10). The effect was most significant for the slag concrete, which for a given strength had a modulus value about 8 kN/mm^2 in excess of the OPC concrete. An increase of 6 kN/mm^2 resulted from the use of fly ash. A similar effect was observed for concrete used in the top cap of the Hartlepool Pressure Vessel;[20] a 5 kN/mm^2 increase in modulus resulted from the use of 50% granulated slag with SRPC compared with an OPC mix of similar strength. Neville and Brooks[21,22] also recorded an increase in modulus using granulated slag.

28. It would appear, therefore, that partial Portland cement replacement using either fly ash or granulated slag increased the stiffness of the concrete in relation to its strength. One reason for this may have been the reduced water demand resulting from cement replacement, although this would have been expected to influence the strength to a perhaps greater extent than the modulus. It is more probably the result of a modification to the hydration products caused by the introduction of the replacement materials.

Creep

29. The cylindrical specimens were maintained under load and summaries of the creep curves are given in Fig. 11. For concrete loaded at an age greater than 24 hours, the effects of fly ash and slag are to reduce significantly the magnitude of creep at a constant stress–strength ratio.

Slag concrete

30. The relationship between creep strain after 100 days under load and the proportion of granulated slag is shown in Fig. 12. A reduction in creep of about 70% occurred for concretes with 75% granulated slag. A similar effect was observed by Neville and Brooks[22] for levels of replacement of up to 50% for concrete stored under moisture stable, mass concrete conditions. Taylor Woodrow[20] and Okada *et al.*[23] also recorded reduced creep with slag concrete; these results are summarized in Fig. 12. It is clear that under mass concrete conditions, the reduction in creep is in proportion to the level of replacement using slag. This is not true for concrete which is drying out; in this case creep is largely unaffected by the use of slag.[24]

Fly ash concrete

31. The relationship between strain after 100 days under load and the proportion of fly ash is shown in Fig. 13. A reduction in creep of about 50% occurred at a replacement level of 30%. A similar effect was recorded for concretes used in the bridges tested on the M56:[25] there was a reduction in creep of 30% when 25% of the OPC was replaced by fly ash.

32. Other workers have also observed the influence of fly ash on creep. Ross[26] tested concretes with up to 25% fly ash by weight of total cement and

recorded a reduction in creep of up to 15%. However, Lohtia *et al.*[27] showed a marginal increase in creep with levels of fly ash up to 25%. The variation of results was perhaps attributable to the difference in the curing regimes, variations in materials or the quality of the ash. However, it is clear that a significant reduction in the creep of heat-cycled concrete containing 30% Pozzolan fly ash was achieved under moisture stable mass concrete conditions.

33. The increased stiffness of blended cement mixes has obvious benefits, particularly in relation to prestressed concrete structures. However, at an early

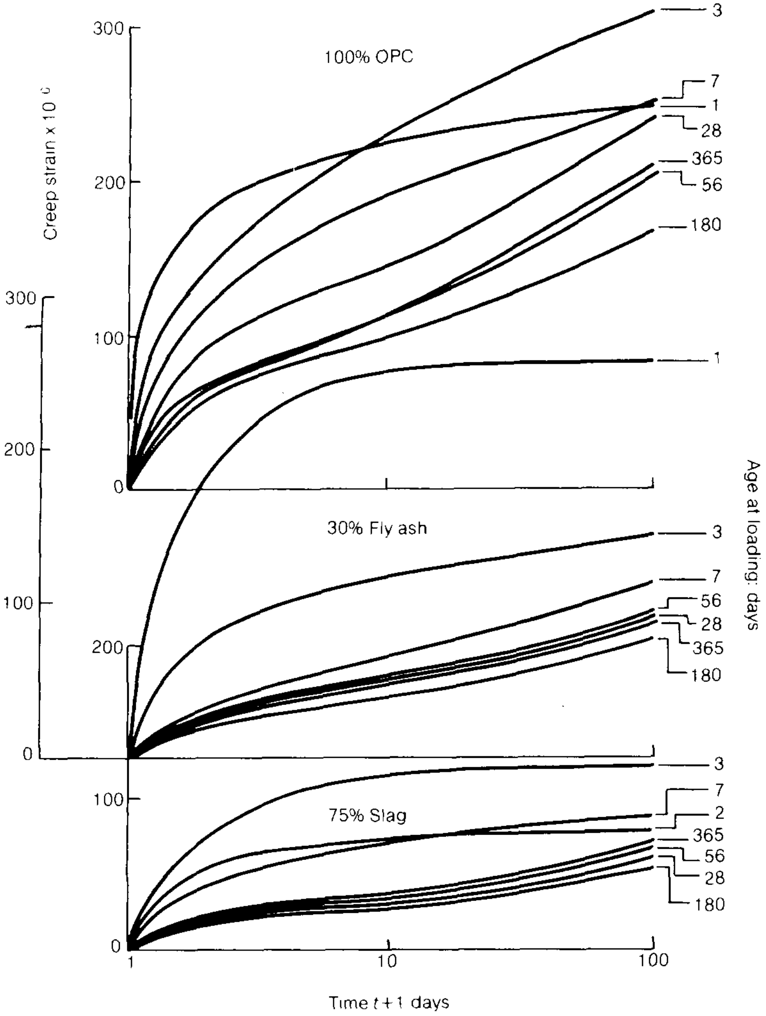

Fig. 11. Creep of concretes with and without Portland cement replacement loaded to a constant stress–strength ratio of 25%

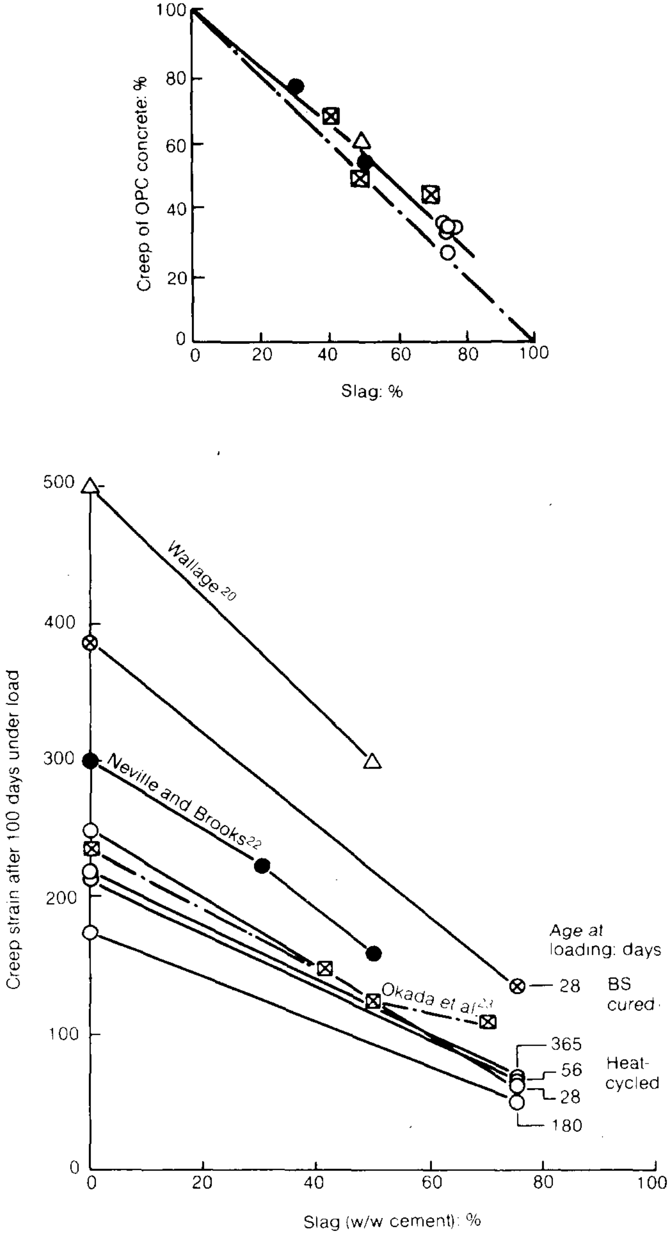

Fig. 12. Influence of Portland cement replacement using ground granulated slag on the creep of concrete

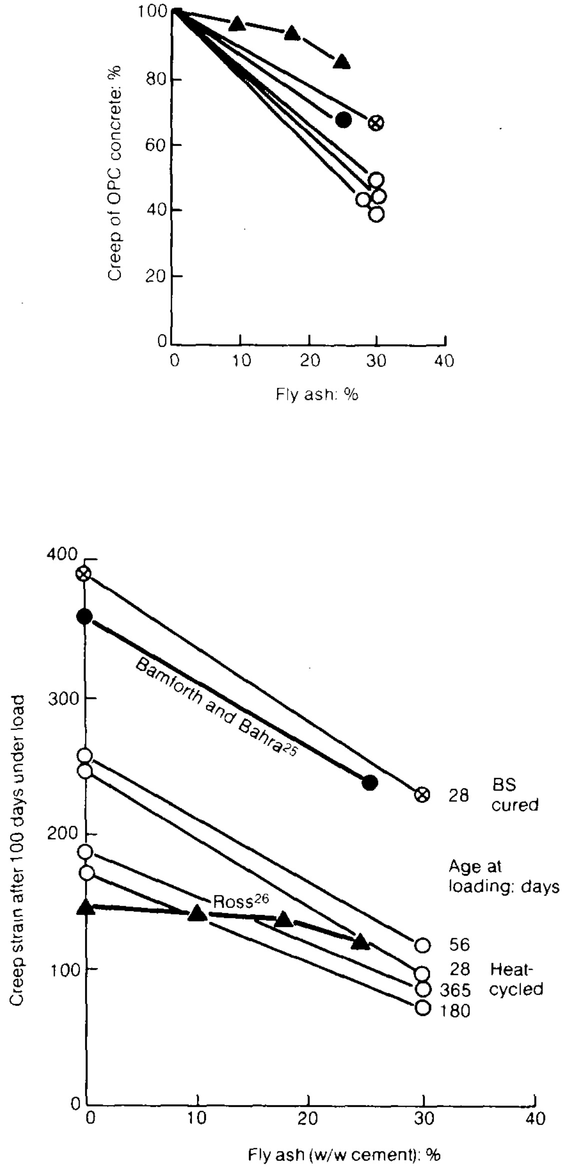

Fig. 13. Influence of Portland cement replacement using fly ash on the creep of concrete

Table 7

Lift height	Minimum level of replacement percentage	
	Fly ash	Blast furnace slag
Up to 1 m	20	40
1·0–1·5 m	25	50
1·5–2·0 m	30	60
2·0–2·5 m	35	70

age, when thermal stresses are developed, it is generally desirable that the modulus be low and the creep be high in relation to the concrete strength. Restrained or differential thermal movement would then result in low and easily relieved stresses, and the strength of the concrete would be sufficiently high to tolerate these stresses. The fact that this phenomenon has not been noted previously suggests that in the more conventional sizes of pour (i.e. up to about 2·0 m) the effect of the temperature reduction outweighs the increased stiffness, resulting in lower thermal stresses.

Recommended replacement levels of OPC replacement

34. To determine the required levels of Portland cement replacement for different pour sizes, the reduction in temperature rise has been balanced against the increased stiffness of the concrete for different levels of replacement and lift height. The results indicated that the minimum replacement levels shown in Table 7 are necessary to achieve a benefit in terms of reduced thermal stress for different size pours. For lifts greater than 2·5 m high, the benefit in terms of reduced temperature is unlikely to be sufficient to offset the increased stiffness in relation to strength and no overall benefit may be achieved. However, there are other benefits which arise from the use of fly ash and granulated slag in massive or cement-rich structures.

Concrete strength development

35. Tests carried out as part of the Scunthorpe instrumentation programme showed that, in terms of the 28 day BS 1881[28] standard cured cube strength, the three concretes examined were essentially similar although the use of the fly ash and particularly the granulated slag caused a reduction in the early strength gain (the granulated slag was used at a much higher replacement level, Fig. 14). The strength development characteristics were significantly different under temperature matched curing conditions. In each case the effect of the early age temperature cycle at the centre of the 4·5 m deep pour was to accelerate the early rate of strength gain.

36. At 28 days, however, although the strengths of the concrete containing replacement materials were enhanced by the temperature cycle, the strength of the OPC concrete was significantly impaired such that the heat-cycled strength was about 30% below the standard water-cured value. The reduction in strength of the OPC concrete was not unexpected, as a similar effect had been observed previously.[29] However, the magnitude of the reduction was greater than

previously recorded and so the test was duplicated in the Taylor Woodrow Laboratories using materials obtained from site. An effect of similar magnitude was observed (Fig. 14).

37. It is unlikely that the reduction in the strength of the OPC concrete was caused simply by the magnitude of temperature rise as all three concretes were subjected to similar increases (45–55°C). It is more probable that the rate of hydration or changes in the hydration process were of greater significance. The following theories might therefore be advanced to explain the effect of the early age temperature cycle.

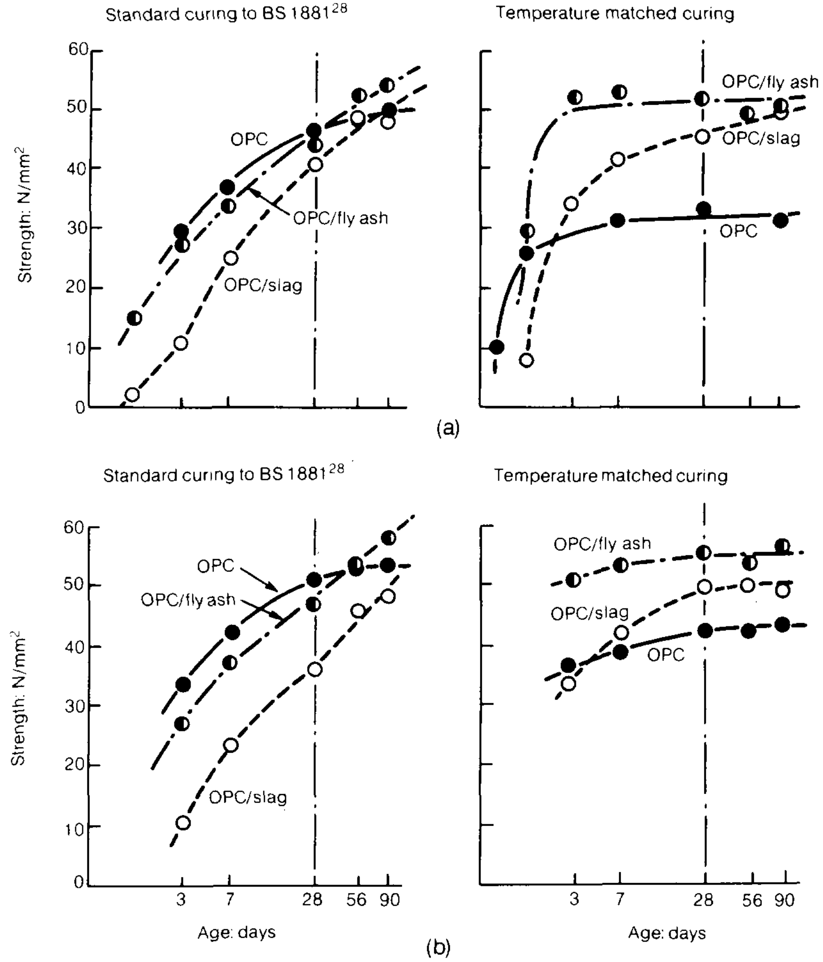

Fig. 14. Strength development of 100 mm cubes subjected to BS 1881[28] and temperature matched curing: (a) cubes cast in situ; (b) cubes cast in the Taylor Woodrow laboratory

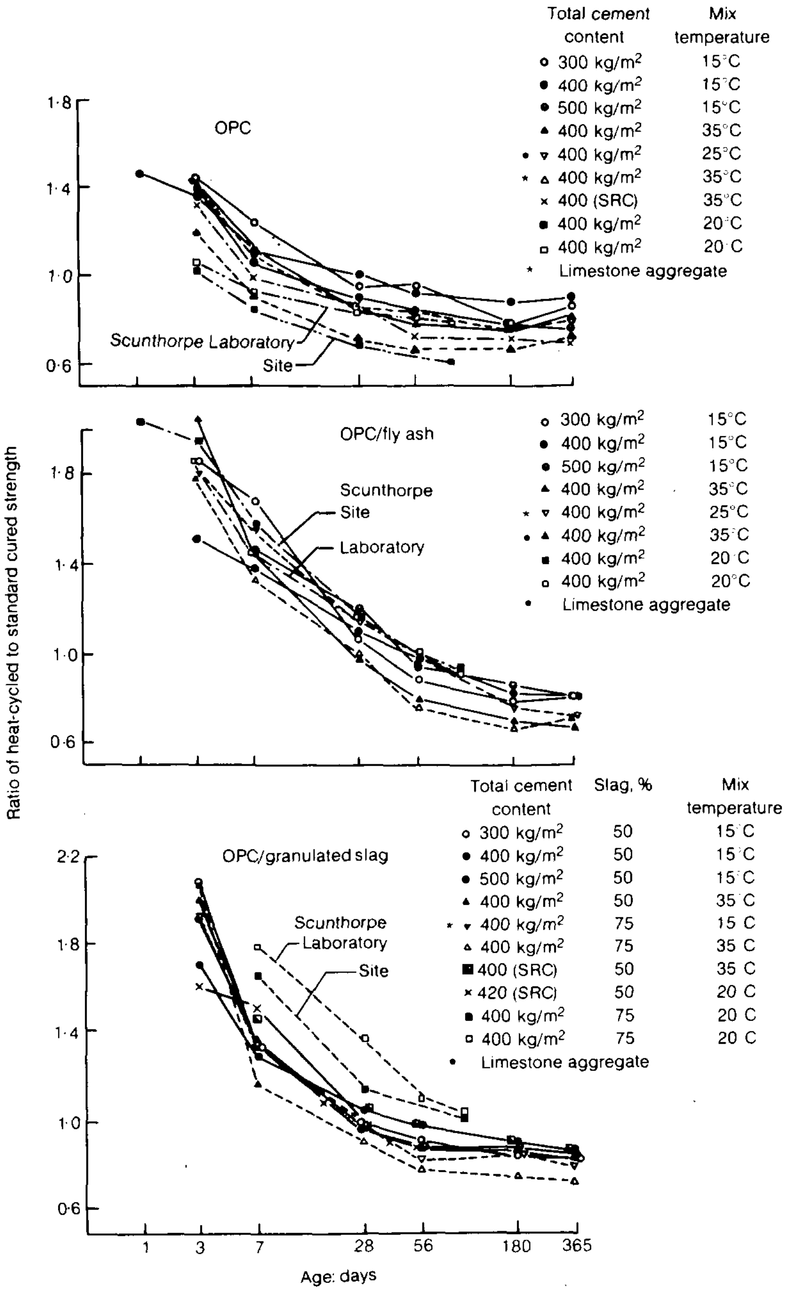

Fig. 15. Ratio of temperature matched to BS 1881[28] cured concrete

Thermal stressing and dislocation

38. The materials comprising concrete have different thermal expansion coefficients and therefore by heating the composite materials, internal stresses can be induced as a result of differential expansion.

39. It is certainly the case that when hardened concrete is heated to 75°C, the magnitude of these stresses can be sufficient to cause aggregate/cement paste bond failure and reductions in the 28 day strength of as high as 40% were obtained when a low thermal expansion limestone aggregate was used.[30] If the rate of heating is reduced, particularly at early age, the internal stresses have more chance to be at least partially relieved by creep, reducing the thermal shock and hence the reduction in strength. However, lower creep in the replacement mixes would tend to exacerbate dislocation in relation to OPC mixes, to some extent counteracting the influence of reduced rate of temperature rise.

Changes in hydration product

40. The strength gain in cement is primarily the result of the formation of tricalcium silicate and dicalcium silicate hydrates. The tricalcium silicate provides the early strength and the slower-reacting dicalcium silicate provides the long-term strength.[31] However, the two hydrates are of the same type[32] and when the curing temperature is increased, although the early rate of hydration is accelerated, both silicates have shown a decrease in hydration at later ages. Lea[32] suggests that this may arise from changes in the lime–silica ratio of the hydration product as hydration proceeds, and the thickness and permeability of the gel coatings that are formed. Idorn[33] concluded that the formation of small amounts of other more crystalline calcium silicate hydrates may be at temperatures above 50°C which appear to be detrimental to strength at later ages.

Laboratory results

41. Tests undertaken as part of the main laboratory test programme also indicated that the early age heat cycle

(*a*) reduced the strength of OPC concrete at 28 days by up to 30%
(*b*) increased the strength of concrete with 30% fly ash by up to 20%
(*c*) marginally changed the strength of concrete with 75% blast-furnace slag within the range ±10%.

These results are shown in Fig. 15. So although there may be little or no advantage in using fly ash or blast-furnace slag to reduce the likelihood of early thermal cracking in pours greater than 2·5 m deep, in terms of in situ performance these materials may help maintain strength under heat-cycled conditions.

Conclusions

Temperature rise

42. Experience from the Scunthorpe experiment and the results of previous in situ measurements have shown that the temperature rise developed during the hydration of cement can be reduced significantly by partially replacing Portland cement with fly ash or ground granulated blast-furnace slag in pours up to 2·5 m deep. In pours about 4·5 m deep, in which the centre will remain in an adiabatic condition for perhaps several days, the advantage in terms of temperature reduction will be small.

43. The rate of heat evolution with the use of cement replacement materials has been shown to be reduced significantly, enabling greater benefit in terms of reduced temperature rise to be achieved in smaller pours from which the heat developed is more rapidly dissipated.

Early age thermal stresses

44. In pours about 4·5 m deep, in which the temperature rise may be reduced only marginally by partial Portland cement replacement, the magnitude of thermal stresses may be increased by Portland cement replacement. This is due to an increase in elastic modulus resulting in higher levels of thermal stress, and a reduction in creep preventing adequate stress relief. In pours less than 2·5 m deep, the reduction in temperature rise can offset the increased stiffness of the concrete, resulting in a reduction in the level of thermal stress and possible cracking.

Strength development

45. By using up to 30% fly ash or 75% granulated slag by weight of cementitious material it is possible to produce concretes with 28 day standard cured cube strengths and workabilities comparable with those normally achieved with OPC concrete.

46. Subjecting concretes to a temperature cycle similar to that which will occur in a massive concrete pour will significantly accelerate the early strength development. In the long term, the strength of OPC concrete may be significantly impaired as a result of the early age heat cycle, such that at 28 days the strength may be up to 30% lower than the standard cured value.

47. In contrast, heat cycling has been found to increase the 28 day strength of concretes containing fly ash by up to 20% and to have little effect on concrete containing granulated slag.

Crack proneness

48. In assessing the use of Portland cement replacements to reduce the likelihood of cracking at early age it is, therefore, not only the reduction in temperature rise which should be considered but also the influence of the replacement material and the early age heat cycle on the development of the physical properties of the concrete, particularly in relation to strength and deformation behaviour. There is clearly a need for further work in this area if Portland cement replacement materials are to be used more widely.

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