Case Histories of Unsatisfactory and Abnormal Field Performance of Concrete During Construction

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Case histories are given relating to concrete produced in connection with the construction of three different projects. One was a major building, the second was an airfield pavement, and the third was a highway bridge deck. The projects are located in three different states on the East Coast of the United States. These case histories have in common that a major part of the problem in each was low strengths of test cylinders. In the first case, defective concrete containing the wrong aggregate and made to the wrong mixture proportions was removed and replaced. In the second case, the problem was traced to the presence in the aggregate of aluminum particles from the bodies of the dump trucks in which it had been transported. In the third case, there were many causes of loss of control of the concrete properties; an instance was found of greater variation of air content in a smaller volume than ever reported previously. In this case, the safety factor in design allowed the concrete to remain in the structure.

CASE 1: BUILDING

This case history began in August 1973. In the construction of a reinforced-concrete building for which a 34.5-MPa (5000-lbf/in^2) concrete had been specified and a mixture proportioned by an approved commercial testing laboratory was being used, the tests on the mixture gave values of more than 41.4 MPa (6000 lbf/in^2), but on the job there were many results in the 24 to 31-MPa (3500 to 4500 lbf/in^2) range.

The concrete in question had been placed between April 10 and June 16. Air temperatures during and after this period had been as high as 46°C (115°F), but by using ice, the temperature at which the concrete was placed was kept to a maximum of 29°C (85°F). Some probe tests and some core tests had been made by the contractor before the beginning of this study. The cylinder-test data were evaluated, and a program of core sampling and rebound-hammer testing was developed and carried out. The table below shows the 28-d cylinder-test results for the three strength classes of concrete involved (1 MPa = 145 lbf/in^2).

Strength (MPa)	
Nominal	Observed
28	22.5, 24.4, 26, 27.5, 27.8, 27.8, 27.9, 28, 28.1, 30, 30.4, 34, 40.8
34.5	19.6, 23.9, 23.9, 24.2, 24.3, 24.4, 27.3, 37.6, 27.9, 28.6, 29.8, 29.9, 30.2, 30.8, 31.8, 32.6, 32.6, 33.6, 34.3, 34.4, 34.5, 34.6, 34.7, 34.7, 34.8, 34.8, 35, 35.5, 36, 36.2, 36.7, 37.9, 38.2, 42.6, 46.2, 47
38	20.6, 22.3, 30, 33.3, 34.2, 38.2, 38.2, 39, 42.8, 43.4, 46.5, 50.3

These results are summarized below.

Nominal Strength (MPa)	n	$\bar{X}$ (MPa)	R (MPa)	Percentage Below Nominal Strength
28	13	28.3	18.4	31
34.5	36	32.5	27.5	55
38	12	36.6	29.7	42

The following table shows the results of strength tests on cores drilled from portions of the structure represented by specific cylinder-test results.

Strength (MPa)		
Nominal	Cylinder	Core
28	22.5	23.2, 21.5
	26	28.2, 24.8
	27.8	24.4, 37.2
	27.9	29.9, 27.7
	30	25.7
	30.4	37.2
34.5	19.6	25
	23.2	33.1, 26
	24.2	24.5
	32.6	42.6
	33.6	35.8
	34.3	30.5
	34.4	40.8, 48.1, 30.7
	46.2	34.3
	47	41.8
38	20.6	18.6
	22.3	19.8, 21
	30	20.3
	33.3	29.9
	34.2	30.8, 29.1
	38.2	25.1, 38.1
	46.5	33.2
	50.3	54.4

These cores represented concrete 60 to 124 d old when tested. In 5 of 21 locations where the cylinder strength was below the specified level, the core strength was above; in 5 of 12 locations where the cylinder strength was above the specified level, the core strength was below.

Rebound-hammer tests were conducted on areas adjacent to the locations of the pairs of drilled cores. Calibration curves were made for the rebound-number versus core-strength relation for each class of concrete (Figures 1, 2, and 3).

The calibration curves were used to predict the range of probable core strengths at a 99 percent confidence level in locations not represented by cores. The table below shows the predicted core strengths in such areas.

Nominal Strength (MPa)	Range (MPa)		Nominal Strength (MPa)	Range (MPa)	
	High	Low		High	Low
34.5	6.2	37.5	38	3.72	34.8
	6.2	37.5		3.72	34.8
	10.3	37.8		4.62	34.8
	11.5	37.8		4.62	34.8
	12.4	37.9		4.68	34.8
	13.1	37.9		6.21	34.9
	14.1	38.0		11.3	35.4
	18.7	38.4		12.4	35.7
	21.2	38.7		12.8	36.4
	22.3	38.8		15.2	36.7
	23.0	39.0		15.9	37.0
	24.1	39.3		18.4	38.5
	24.1	39.3		20.7	41.3
	26.8	40.3		22.1	43.3
	31.5	45.3		22.6	46.4

Figure 1. Core strength versus rebound-number calibration: 28-MPa mixture.

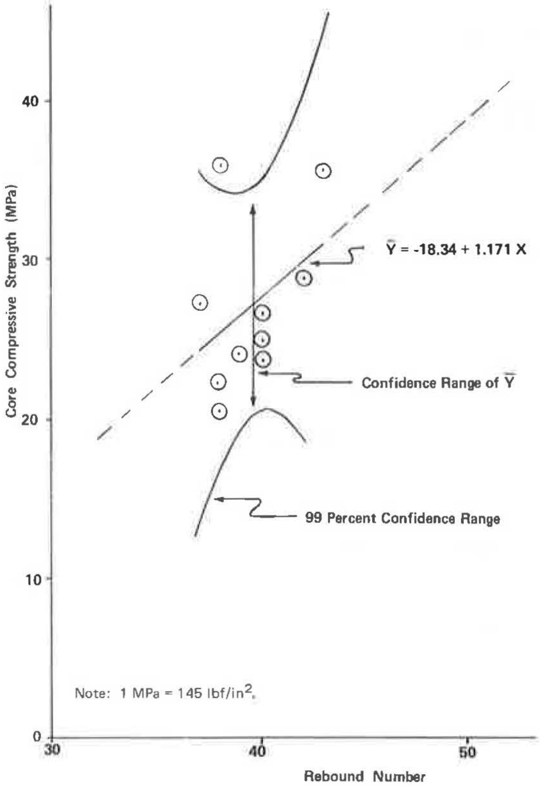


Figure 2. Core strength versus rebound-number calibration: 34.5-MPa mixture.

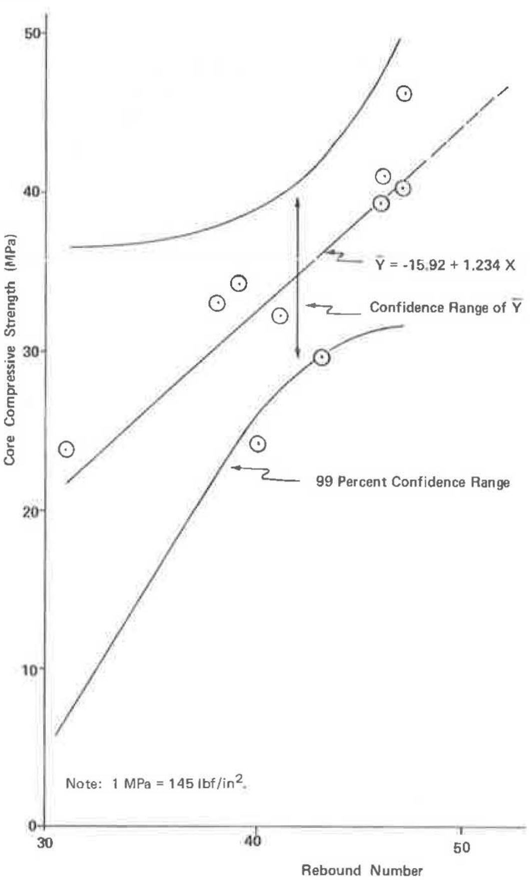
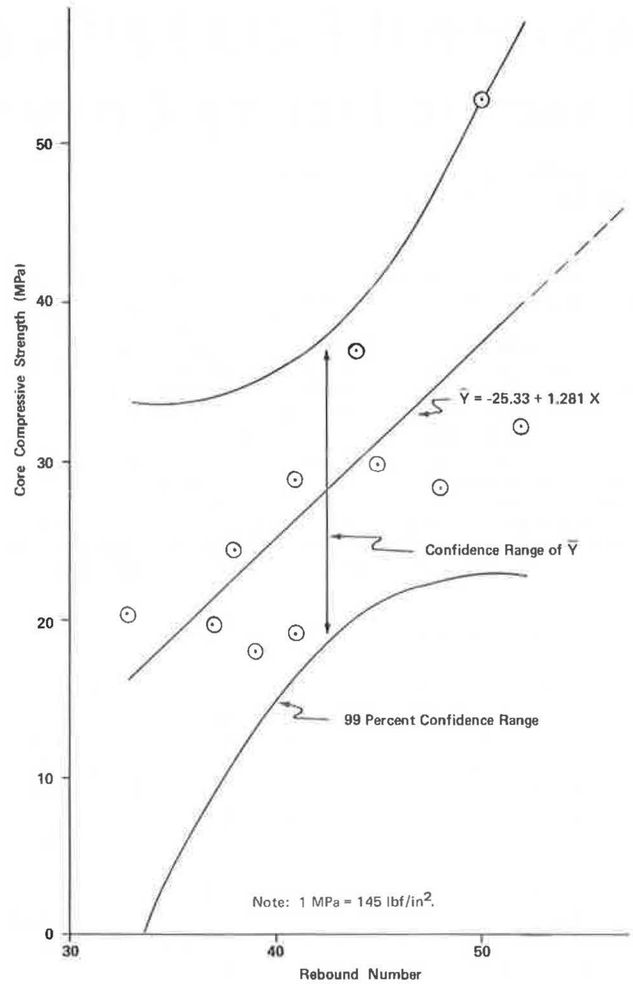


Figure 3. Core strength versus rebound-number calibration: 38-MPa mixture.



Nominal Strength (MPa)	Range (MPa) High	Range (MPa) Low	Nominal Strength (MPa)	Range (MPa) High	Range (MPa) Low
34.5	32.0	46.3	38	22.6	46.4
	32.8	51.3		22.6	48.0

In the case of the 34.5-MPa concrete, there were no areas in which the rebound values indicated, within 99 percent probability, either that the concrete must be stronger or that it must be weaker than its nominal value. However, in the case of the 38-MPa (5500-lbf/in²) concrete, while in no area did the rebound values indicate a 99 percent probability that the concrete was at least nominal strength, in 11 of the 17 areas tested, there was a 99 percent probability that it was less than nominal strength. Additional rebound-hammer tests were made but are not included here.

The procedure followed was to evaluate areas based on the cores by using the following standard (1):

> Concrete in the area represented by the core tests will be considered structurally adequate if the average of the three cores is equal to at least 85 percent of [the ultimate compressive strength] f_c' and if no single core is less than 75 percent of f_c'.

On the basis of the core-test results and a few areas where the cylinder-test results indicated low strengths, which were confirmed by rebound-number estimates, 20 of 29 columns specified to contain 38-MPa (5500-lbf/in^2) concrete were determined to be not in compliance with respect to strength. A few core samples were laboratory tested to determine cement and air content. The results (given below) for samples having specified cement and air contents of 439 kg/m^3 (752 lb/yd^3) and 4 percent respectively make it clear that the deviation of strength, at least in certain cases, from that required by the contract and capable of being produced from the approved mixtures, resulted from unauthorized changes in aggregate type, aggregate grading, and cement content from those expected for these mixtures (1kg/m^3 = 0.062 lb/ft^3).

Strength (MPa)		Cement Content	Air Content
Nominal	Core	(kg/m^3)	(%)
34.5	24.5	233	8.7
	40.8	603	3.7
38	20.4	316	2.2
	37.5	628	5.7

Figure 4 shows cores of concrete containing the approved aggregate. Figure 5 shows cores of concrete containing inferior aggregate from a different source and of a different grading.

This experience suggests a method for using a nondestructive test—i.e., the rebound hammer—to estimate the range of strength of concrete in a structure. In this case, the contractor was notified as to the known extent of nonacceptable concrete by drilling and testing cores, and he then removed and replaced those portions of the structure.

Figure 4. Cores from concrete made with approved aggregate.

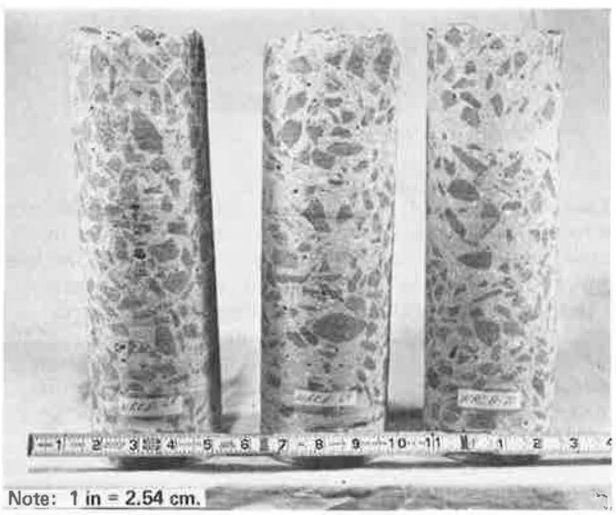
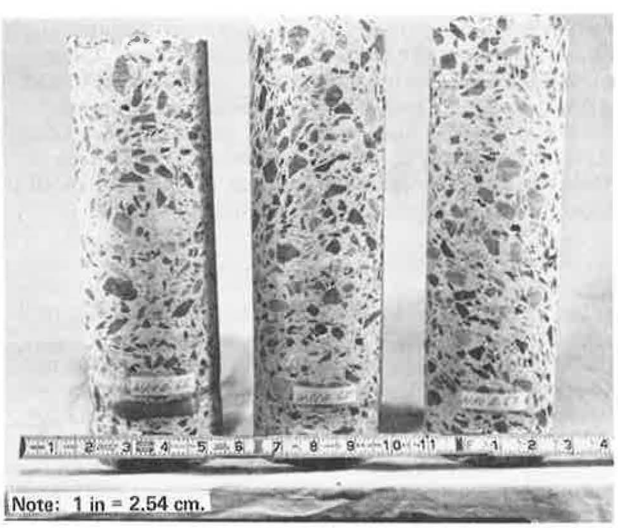


Figure 5. Cores from concrete made with inferior aggregate.



CASE 2: AIRPORT PAVEMENT

This case history began in February 1974. The problem was described as involving unusually large fluctuations in flexural-strength results for no apparent cause. Initially, it seemed that the following assumptions could be made:

1. Since the problem was encountered with both type 1 and type 1P cement, it is apparently not a function of cement type.
2. Since the aggregates used have a long history of satisfactory ability to produce concrete having a high flexural strength, the problem is apparently not a function of the intrinsic quality of the aggregate.
3. Since many specimens were of satisfactory strength, the problem appears to relate to specific batches or specific specimens.

Figure 6. Aluminum particles from aggregate stockpile.

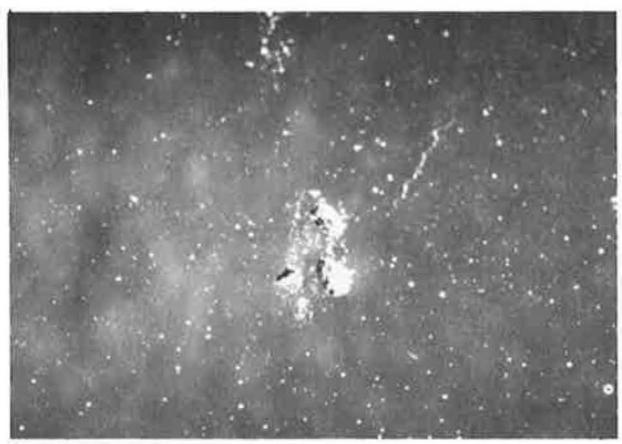


Figure 7. Hydrogen evolution by aluminum particles in alkaline solution.

Figure 8. Slice through full thickness of core showing zones of different air content: (a) 12 percent and (b) 3 percent.

Figure 9. Enlarged view of 12 percent air-content zone (Figure 8a).

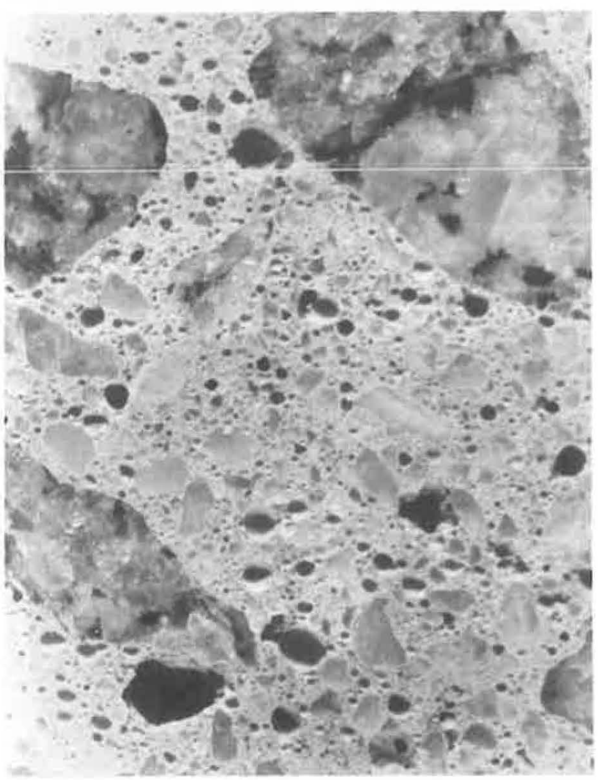
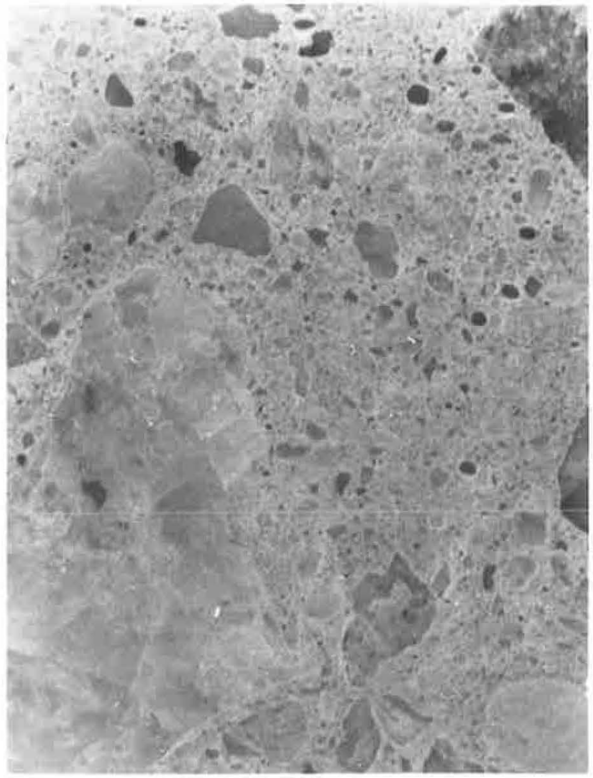


Figure 10. Enlarged view of 3 percent air-content zone (Figure 8b).



On these assumptions, the first possibility considered was based on a case history of concrete having an increased gas-bubble content after placement that had been pumped through aluminum pipe, but no pumping was involved here. However, the aggregate was a hard, tough, angular, crushed granite that was transported by truck for a considerable distance from the production site to the project site in large aluminum-body dump trucks, and an analogous mechanism might be involved. By March 18, work was resumed, it having been found that satisfactory strengths were obtained when the coarse aggregate was rewashed after delivery to the project site.

Meanwhile, it was established that grains of metallic aluminum were present among the fines accompanying the coarse aggregate as it was dumped into stockpiles at the job site (Figure 6). It was also confirmed that these particular aluminum particles evolved hydrogen gas when immersed in an alkaline solution (Figure 7) and that there is a direct relation between decreased flexural strength and increased air content of hardened concrete as determined by microscopic examination. So far as is known, this is the first recorded case of generation of hydrogen gas in concrete by aluminum particles present as a contaminant in the aggregate after transportation in aluminum-body trucks.

CASE 3: BRIDGE DECK

The third case began in August 1971. At first, the problem was one of bridge-deck deterioration, but by September, it had become one of quality assurance for a replacement deck. The replacement was done during the winter of 1971/1972 and used more than 2270 Mg (50 000 cwt) of cement and 680 Mg (1 500 000 lb) of reinforcing steel. The first mixture proportions selected used a type 3 cement to give a 7-d design strength of 31 MPa (4500 lbf/in^2), i.e., an average of 35.2 MPa

(5100 lbf/in^2). Problems were encountered because the type 3 cement had been selected merely to reduce the duration of the required period of wet curing. The specifications were changed to membrane curing, type 1 cement, and a 28-d design strength of 31 MPa. In November, about a month after placement was begun, problems with the 28-d strength were encountered. The concrete had been dry batched, hauled about 37 km (23 miles), mixed, discharged into a pump, pumped to the deck, spread, consolidated, finished, and coated with curing compound. The problems involved control of slump and air content and seemed related to the length of time between batching and discharge and to be aggravated by the procedure of putting all the air-entraining admixture in half of the wet sand that goes over half of the stone and under the first half of the cement. An extensive series of tests using the rebound hammer and core drill were made, and 45 cores were tested in the laboratory. The core strengths averaged 19 MPa (2750 lbf/in^2) and ranged from 12 to 27.4 MPa (1710 to 3970 lbf/in^2). Three job-made cylinders had strengths of 22.7 to 46.2 MPa (3300 to 6700 lbf/in^2) and cement contents of 288 to 625 kg/m^3 (5.1 to 11.1 bags/yd^3). Other cores were tested for air content and frost resistance. The air contents of nine cores ranged from 5.7 to 10.2 percent, but one having a value of 7.2 percent (Figure 8) had two zones, one with an air content of about 12 percent (Figure 9) and the other with an air content of about 3 percent (Figure 10). By May 1972, the average core strength had increased from about 17.9 MPa (2600 lbf/in^2) at 3 months age to about 20.7 MPa (3000 lbf/in^2) at 7 months age.

REFERENCE

1. Building Code Requirements for Reinforced Concrete Concrete. ACI, 1971, paragraph 4.3.5.1.

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Map Cracking in Limestone-Sweetened Concrete Pavement Promotes D-Cracking

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A Portland cement-concrete pavement constructed in north central Kansas in 1963 showed map cracking near the sawed transverse joints by 1970. The pavement had been built using Republican River sand gravel, which is known to be reactive, and 30 percent Towanda limestone from near Milford, Kansas, had been added to the concrete mix as a sweetener to prevent the map cracking. The limestone did not prevent the map cracking, but was not otherwise involved in the deterioration that occurred before 1970. By 1972, however, after the surface near the joints was opened by the map cracking, the limestone aggregate particles became directly involved through the freeze-thaw deterioration type of the D-cracking. By 1974, deterioration was rapidly spreading outward from the transverse joints and blowups at the joints were requiring considerable repair. The synergistic effects of map cracking during the summer [with an average of 5000 degree hours, above 29.4°C (85°F)], and D-cracking during the winter (with more than 68 freeze-thaw cycles) promoted even more rapid deterioration and joint blowups. The progress of the deterioration was followed by the study of pavement cores obtained in 1970, 1972, and 1974.

For many years, a standard method for the prevention of serious map cracking in Kansas concrete has been to allow the addition of 30 percent limestone sweetener to unapproved sand-gravel materials. (Unapproved indicates that the sand gravel is known to be reactive and produce map cracking or that no information is available concerning its service record either in the field or from laboratory tests.) The sweetener is used with either type 1 or type 2 cement.

In 1963, a 17.7-km (11-mile) long two-lane pavement was constructed in north central Kansas. The contractor chose an unapproved Republican River sand-gravel material from Scandia, Kansas, with a fineness modulus of 3.52 and a specific gravity of 2.62 as the basic aggregate. For the sweetener, he chose Permian Towanda limestone from Milford, Kansas. This limestone coarse aggregate was all through a 38.1-mm (1.5-in) sieve and had a fineness modulus of 7.24, a specific gravity of 2.47, and a water absorption of 4 percent. The particular limestone used is one of the better ones available in Kansas in terms of its resistance to the freeze-thaw type of D-cracking. Even so it has produced D-cracking (1), and would be rejected in many states. Studies made by the Corps of Engineers in their investigations of aggregates for the Milford Dam showed that the Towanda limestone was physically superior to most of the coarse aggregates from sources economically available (5). Thus, the limestone chosen seemed the best of those readily available. The aggregate mix ratio was the standard 70 percent sand gravel and 30 percent limestone sweetener. The contractor used a type 1 cement that was near a type 2 and had the following properties:

Property	Value (%)
Total alkali	0.59
C_3A content	8.9
Autoclave soundness	0.33
C_3S content	51.5
C_2S content	23.4
MgO	2.35
SO_3	1.82

The concrete mix used 368.1 kg/m^3 (620.4 lb/yd^3) of cement with a 0.47 water-to-cement ratio. The concrete was air entrained with a target value of 6.5 percent. Two admixtures were used separately in some