(5100 lbf/in²). 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/in²) and ranged from 12 to 27.4 MPa (1710 to 3970 lbf/in²). Three job-made cylinders had strengths of 22.7 to 46.2 MPa (3300 to 6700 lbf/in²) and cement contents of 288 to 625 kg/m³ (5.1 to 11.1 bags/yd³). 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/in²) at 3 months age to about 20.7 MPa (3000 lbf/in²) 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/m³ (620.4 lb/yd³) 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 sections of the pavement on an experimental basis. Lignosulfonic and hydroxylated carboxylic acid type, water-reducing set retarders were each added at a rate of 2 percent (2 oz/sack of cement). Where these were used, the water-to-cement ratio was reduced to 0.44, and the cement content was reduced to 340.2 kg/m^3 (573.4 lb/yd^3). Control sections of pavement were placed where the set retarders were not used. The concrete mix was typical for the proportions used, with a slump of about 44.45 mm (1.5 in) and a unit mass of 81.89 kg/m^3 (138 lb/ft^3). Everything appeared to be normal, and no problems were anticipated from the concrete for many years.

The first hint of a developing problem was in 1964. In the D-cracking survey made late that year, this pavement, which was less than 2 years old, exhibited many short transverse cracks that showed a distinct but narrow band of staining (1, Figure 7). Most of the joints themselves showed little or no staining. Water was present at the pavement surface seeping from many cracks and joints. Otherwise, the pavement was in good condition (1, Figure 8).

Staining is a matter of concern but not necessarily an indicator of serious problems to come. However, by 1969 the joints showed heavy stain that appeared to be D-cracking and by 1970, there were dark stains adjacent to the joints. Cores from such badly stained areas usually have many cracked limestone particles and nearly parallel horizontal cracks mostly limited to the lower half of the core.

Cores were obtained from near several of the heavily stained transverse joints. Close examination of their top surfaces showed a fine pattern of cracking, but internally the cracks were primarily vertical and extended only a short distance downward into the concrete. There were no cracks in the lower halves of the cores. Gel was found in many of the cores, which showed that a reaction was taking place. It was obvious from the cores that the limestone coarse aggregates were not directly involved in the deterioration. They were sound, and the short surface cracks went around rather than through them. There was no evidence in the cores that freeze-thaw damage had occurred. The Republican River sand gravel used contained its usual volume of reactive particles, such as opaline siltstone, opaline sandstone, opaline limestone, and glassy volcanics (4). Steam treatment of some of the cores produced more gel, which showed that the reaction was not complete.

The evaluation of the cores in 1970 led to the conclusion that the problem was one of map cracking, rather than one of D-cracking. This is supported by the 1970 report of Bukovatz and others (2), who found that Towanda limestone (from same quarry as that used here) used as a control was itself not effective in preventing alkali-aggregate expansion. Thus, although the limestone coarse aggregate was not directly involved in the cracking, it was indirectly involved because it was not adequately functioning as a sweetener.

Within 2 years, there was a distinct pattern of cracks roughly parallel to the sawed transverse joints at several locations. These cracks were interconnected by a pattern of short cracks. This crack pattern was not typical of D-cracking, but it was present only near the transverse joints and transverse cracks, primarily at the joints. Thus, it was present at typical D-crack locations. The best description of the pattern is modified map cracking (8) or hybrid map cracking and D-cracking (11).

A new set of cores was taken in 1972, many 0.3 m (1 ft) or less from the locations of the 1970 cores. Those taken from the cracked areas showed that the internal concrete was much deteriorated, including freeze-thaw deterioration of the limestone aggregate that was limited to the top halves of the cores. In addition to the previously existing vertical cracks, there was now a set of roughly parallel horizontal cracks in the upper halves of the cores. The limestone aggregates were now badly cracked. Several cores broke at the level of the mesh, with the cracking primarily above that level. Thus by 1972, it was evident that D-cracking was also involved in the deterioration process.

With both modes of deterioration, map cracking and D-cracking, operating synergistically during 1973, 1974, and 1975, there was rapid deterioration, and blowups at the joints became a common occurrence. By the end of 1974, only 3 percent of the more than 900 joints were still good. Seventy percent of the joints showed staining and cracking, and the remaining 27 percent had required repairs varying from asphalt patching to complete replacement of the concrete at the joint because of pavement blowups.

Cores were obtained again in 1974. The D-cracking conditions were more widespread by then and deeper into the concrete. The concrete next to the joint had progressed to the stage of hybrid map cracking and D-cracking at which freeze-thaw deterioration was also occurring. Farther away from the joint, but still within the stained surface area, map cracking was occurring in the surface, but freeze-thaw deterioration of the limestone aggregates had not yet begun. Cores taken some distance from the joint and outside the stained area showed some evidence of map cracking through surface crazing and minor internal gel deposits. There was no indication of freeze-thaw damage in the nearly sound concrete areas outside the stained zone. A series of cores taken successively farther away from the sawed joint showed the deterioration history as it had been observed through time near the joint.

First, there is a fine surface crazing that can be seen only by close scrutiny. This is accompanied by, or more often preceded by, the staining of the surface near the joints. The staining is a result of deposits left at the surface through evaporation of water that seeps to the surface near the joints. This continuous wetting and drying at the surface by water coming from below causes surface map cracking to occur because of the reactive sand gravel and the ineffective sweetener.

After the surface is opened by the map cracking, water can enter from above more readily, and water from below has easier access to a greater number of limestone coarse aggregate particles. These then soon become critically saturated, and freeze-thaw deterioration in the form of D-cracking begins. This means that deterioration continues summer and winter alike.

Map cracking occurred in this pavement, even though the total alkalies of the cement were less than 0.6 percent and a limestone sweetener was used. Gibson had observed several years ago that the Republican River sand gravel, when used as a single aggregate, has produced severe cracking in concrete constructed with cement having an alkali content of less than 0.60 percent (3). He reported at the same time, however, that the most economical and satisfactory method to stop this map cracking was to add 30 percent durable, absorptive crushed limestone. At that time, he was unaware that this particular source of Towanda limestone is not effective as a sweetener, which was first reported by Bukovatz and others in 1970 (2).

The particular quarry from which the limestone sweetener was obtained has been closed by the construction of Milford Lake. However, it is not known whether other limestone sweeteners in use may also fail to prevent map cracking. There are other locations at which D-cracking began at the surface and worked downward to the bottom of the slab, and some of these will be studied to see whether a reactive sand-gravel material was used in them.

The water-reducing, set-retarding admixtures did not appear to influence the development of either map cracking or D-cracking. The pavement condition is approximately the same where either set retarder was used and where none was used. Therefore, they apparently neither contributed to nor hindered the development of deterioration.

The area in question has an average precipitation of 686 mm/year (27 in/year), but a pan evaporation rate of about 1626 mm/year (65 in/year) (4). It is subjected to an average of 5000 summer degree hours above 29.4°C (85°F) in a normal year (7). This high number of degree hours is experienced only in central and eastern Kansas, southeastern Nebraska, central and western Missouri, most of Oklahoma outside the panhandle, and north central Texas. The highest average number of such degree hours, more than 7000, is experienced only in portions of Kansas and Oklahoma. The maximum expected in these areas is more than 12 000 degree hours above 29.4°C (7).

The mean annual snowfall on the pavement is about 610 mm (24 in) (6, Figure 4), and more than 68 freeze-thaw cycles can be anticipated each year (9, Figure 2). In 1974, an average of 1500 passenger vehicles and 500 large trucks traveled the road daily (10).

It is reasonably certain that the high summer temperatures, the many freeze-thaw cycles, and the salt and melted snow all contributed to the deterioration. The daily pounding by heavy trucks would add to the problem by loosening and removing the cracked surface concrete. However, the occurrence of blowups because of internal expansion has done the most damage. The plane of splitting of the concrete in the blowups was the level of the load-transfer bars.

Thus, the Towanda limestone sweetener failed to prevent the reactive Republican River sand gravel from causing map cracking. The map cracking allowed the limestone coarse aggregate to become critically saturated, and freeze-thaw led to D-cracking. Together, these effects increased the process of pavement expansion, and with the help of water and hot weather, blowups at the joints soon followed.

REFERENCES

1. J. E. Bukovatz, C. F. Crumption, and H. E. Worley. Study of D-Cracking in Portland Cement Concrete Pavements. Field Phase. State Highway Commission of Kansas and Federal Highway Administration, Rept. 1, 1973.
2. J. E. Bukovatz, W. H. Wendling, and H. E. Wallace, Jr. Expanded Shale Used as an Admixture for Sand-Gravel Aggregate Concrete. State Highway Commission of Kansas and Federal Highway Administration, 1970.
3. W. E. Gibson. Field Experience With Alkali-Aggregate Reaction in Concrete: Central United States. HRB, Res. Rept. 18-C, 1958, pp. 6-7.
4. D. W. Hadley. Field and Laboratory Studies on the Reactivity of Sand-Gravel Aggregates. Portland Cement Association, Journal of the Research and Development Laboratories, Vol. 1, No. 3, Sept. 1959, pp. 17-33.
5. K. Mather, A. D. Buck, and W. I. Luke. Alkali-Silica and Alkali-Carbonate Reactivity of Some Aggregates From South Dakota, Kansas, and Missouri. HRB, Highway Research Record 45, 1964, pp. 72-109.
6. Pavement Traffic Marking Materials and Application Affecting Serviceability. NCHRP, Synthesis of Highway Practice 17, 1973.
7. R. G. Nevins. Environmental Control for Health, Comfort, Productivity, and Learning. Research Foundation of Kansas, Jan. 1968.
8. W. M. Stingley, J. D. McNeal, and R. L. Peyton. The McPherson Test Road: Tenth-Year Report. Proc., HRB, Vol. 39, 1960, pp. 191-204.
9. V. R. Sturrup and T. G. Clendenning. The Evaluation of Concrete by Outdoor Exposure. HRB, Highway Research Record 268, 1969, pp. 48-61.
10. Traffic Flow Map. Planning and Development Department, State Highway Commission of Kansas, and Federal Highway Administration, 1974.
11. L. V. White and R. L. Peyton. Condition of Concrete Pavements in Kansas as Affected by Coarse Aggregate. Proc., HRB, Vol. 25, 1945, pp. 129-146.

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Resilient Response of Railway Ballast

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The resilient responses of five typical open-graded aggregate materials (dolomitic limestone, blast-furnace slag, granitic gneiss, basalt, and gravel) that are used for railway ballast were measured in a triaxial cell. Three levels of compaction and seven stress levels were used. The results were used in regression analyses to develop equations relating the resilient modulus of a specimen to its first stress invariant. They were also used in correlation analyses attempting to relate the resilient response to the physical properties (particle index, specific gravity, Los Angeles abrasion, gradation, flakiness, soundness, and crushing index), but no consistent relations were established. It is concluded that (a) the resilient response of a specimen is essentially independent of its stress history, (b) the resilient moduli of no. 4 and no. 5 ballast-gradation specimens are