Figure 9. Surface-temperature distributions for Sections 23 (fine-grained base course), 8 (normal), and 26 (coarse-grained) during winter seasons 1977-1978 to 1979-1980.

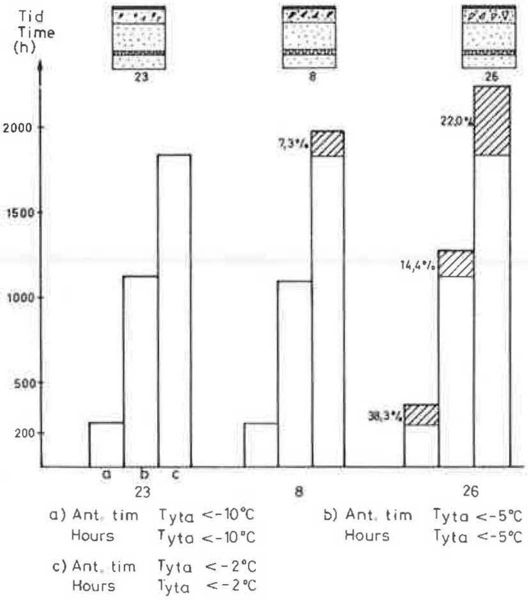
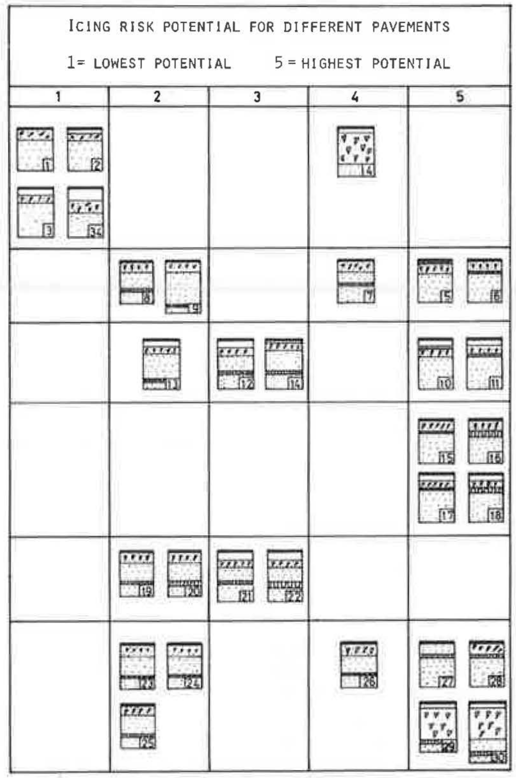


Figure 10. Icing risk potential of road pavements represented at Test Field Linköping 1976.



When insulation is laid high up in the road structure, the thickness of the insulation layer has a certain, but comparatively small, effect on icing risk potential, whereas the significance of the thickness is even smaller when the insulation is at a greater depth. Thicker insulation layers involve a somewhat higher risk of icing than thinner layers.

In the case of road types provided with thermal insulation, the materials in the course above the insulation have great significance with regard to the occurrence of slippery conditions. Pavements containing coarse and relatively dry materials such as ungraded crushed rock and coarse-grained base-course material have a higher icing risk potential than roads containing finer-grained materials of greater water content, such as fine-grained base-course material and sand.

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Effect of Moving Traffic on Fresh Concrete During Bridge-Deck Widening

HOWARD L. FURR AND FOUAD H. FOUAD

Traffic in lanes adjacent to a deck that is being widened or reconstructed causes deflections and vibrations in the fresh concrete deck. A study of the effects of these disturbances in concrete decks is reported here. Decks in service for years were inspected for signs of deterioration; deflections and vibrations were mea-

sured during concrete placement and initial curing; bridge-deck cores were analyzed for cracks and signs of bonding problems in the reinforcing steel and tested for strength and soundness. Laboratory beams were constructed and tested to simulate a transverse strip of a deck slab. Periodic deflections and vibrations were applied from time of casting to one day. No deterioration that could be attributed to traffic during construction and curing of the decks was found in existing decks. The study of the cores showed no difference in cracking in cores taken from disturbed areas of the deck from that in cores taken from undisturbed areas. There was evidence of creation of a void in the new concrete around certain rebar dowels bent at right angles in a horizontal plane on emerging from the old concrete. This situation was found only in cores taken at the joint between old and new concrete, and only one bridge of all that were studied had this bent-dowel detail. The study shows that no detrimental effects should be expected in deck concrete supported by steel and prestressed concrete beams spanning up to about 100 ft when the decks are widened or reconstructed under normal traffic.

Bridge-deck concrete that is placed adjacent to moving traffic undergoes considerable disturbance from vibrations and deflections caused by the traffic. A study made to determine the effects of that disturbance is reported here.

Concrete, in a fluid state when placed, rapidly becomes plastic and then solid within a period of 2 or 3 h after mixing. Disturbances at very early ages could cause changes in its physical character that would influence its behavior in service. Puddling of the fluid material causes segregation, which causes problems in durability and reduced strength. If the reinforcing steel moves about in the plastic concrete, it could wallow out a void, depending on the relative movement and the plasticity of the concrete. Such areas reduce or destroy bond, and they can cause durability problems if invaded by water. The early-age solid concrete (that only a few hours old) has little strength and little capacity for strain. It is easily cracked when the strain becomes great.

In both widening and deck replacement, new concrete is joined to older concrete by a construction joint. In deck widening, the curb and a short width of deck are cut back, usually to the outside beam of the original structure. The reinforcing steel at that joint, referred to here as the break line, is left to form dowels extending from the old deck into the concrete of the new portion of the deck. In replacement, construction is done in stages to permit continued use of the bridge. A portion of the old deck is removed and traffic is routed over to the other lanes until the broken-out part is replaced and cured out. Traffic is then moved over to the new part and another part is broken out and replaced. This is continued until the entire deck has been replaced. Some of the earlier work of this kind provided no tie between the older and the new concrete (1), but the current practice ties the two parts together with reinforcing bar dowels (2,3). Some of the typical joints are shown in Figure 1.

A vehicle moving over the structure causes the loaded lanes to deflect with respect to the portion being replaced. This causes bending in the new slab in a manner somewhat like that shown in Figure 2. Negative bending moment develops over the first new beam and puts the top of the deck over that beam in tension. If the curvature exceeds the capacity of the fresh concrete, the slab will crack. Another occurrence in the new concrete bay might be relative movement between reinforcing steel and the fresh concrete. If such movement does occur and if the concrete is plastic, there is a danger that a wallowed-out area will form around the steel.

Both the bending over the beam and the relative movement between steel and concrete make the inside bay of the fresh new concrete a critical area. For purposes of differentiating it from the much more stable areas beyond that bay and near supports, it is referred to as the disturbed region. The more stable areas are called undisturbed regions.

The investigation reported here consisted of a field and laboratory study to determine whether moving traffic on a deck under construction caused any of the problems discussed above or any other recognizable problem. The field study included a visual inspection, core sampling, and deflection measurements of existing bridges that had been widened or had had decks replaced under traffic. All these bridges except four had been in service at least two years when inspected. Following this inspection, nine bridges, four of which were under construction, were studied for the effects of moving traffic on the new concrete deck. In addition to the field study, model beams were studied under laboratory-controlled conditions to gather information on properties of fresh concrete and to test theories about the behavior of early-age reinforced beams. In the context used here, early age refers to ages not exceeding 24 h.

INVESTIGATION

Visual Inspection

A visual inspection of 30 bridges was made to determine the overall condition of the decks that had been widened while traffic was being carried in an adjacent lane. Bridge types consisted of simple continuous and overhanging steel beams, simple reinforced and prestressed concrete beams, and reinforced concrete simple slabs. The spans ranged from 25 ft for slab types to 110 ft for prestressed-concrete-beam types. The overall condition of the top deck surface was of concern, but greater emphasis was given to the region at the break line and the region over the new beam adjacent to that joint. In most of the bridges, the underside of the deck was also inspected. All the bridges were in service during the inspection, and in most cases no effort was made to control traffic during the inspection.

Vibration and Deflection Measurements

Recordings were made of midspan deflections of beams on five existing bridges as random trucks crossed in the lane adjacent to the break line. A 10-channel recorder was used to record the deflections measured by 10 linear potentiometers connected as shown in Figure 3. Figures 4 and 5 show a typical record of a truck crossing adjacent to the break line on a three-span continuous steel I-beam bridge. During measurements, the widened portion (the newer concrete) was closed to traffic, but no other control was imposed. Records made under normal traffic were collected only for larger trucks by using, in most cases, the lane adjacent to the break line. Transverse curvature of the deck at midspan was calculated from the deflection records, and the vibrational frequency was determined from the same traces.

Deflection and vibration measurements were made on four bridges during deck replacement in stages. In all except one, the same procedure was used as that followed for existing bridges. In that one exception, deflections were measured at selected midspan locations on reinforcing steel and concrete forms as well as on beams. The data were used to determine whether there was relative movement between the steel, the forms, and the concrete as well as in determining curvature of the new concrete. A typical record of deflections in the exception cited is shown in Figures 6 and 7.

Bridge-Deck Cores

Cores were drilled from the decks for purposes of

Figure 1. Types of construction that connect new concrete to existing bridges.

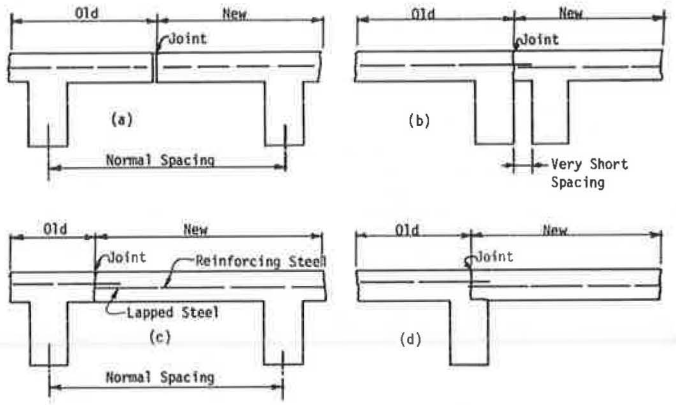


Figure 2. Relative movements between old and new concrete when traffic moves across bridge.

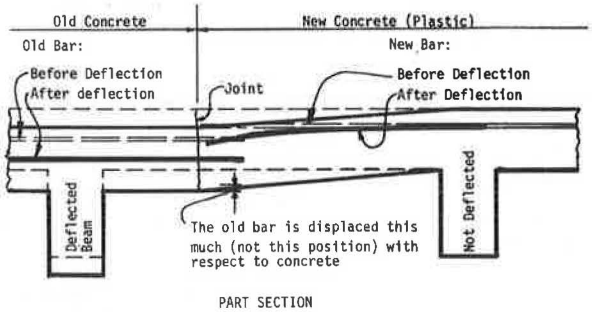


Figure 3. Detail of attachment of cable to bridge beam.

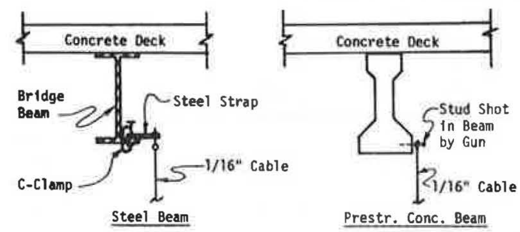
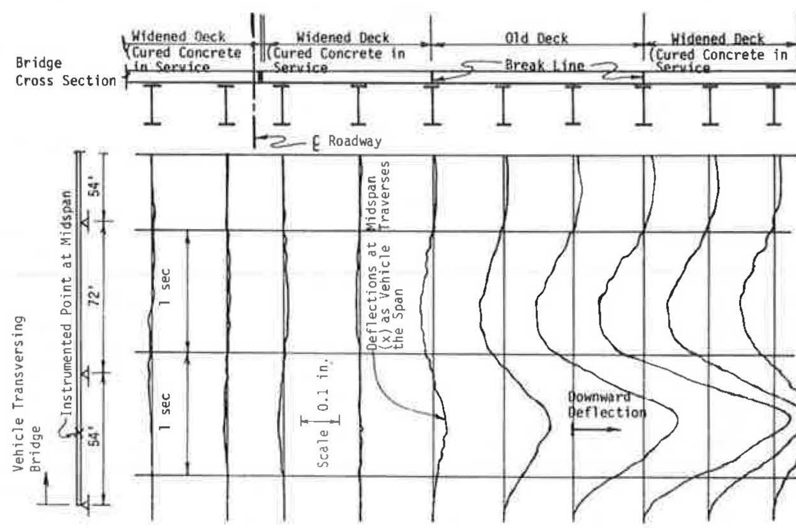


Figure 4. I-45 and FM-517, Dickenson (Houston): midspan deflections of beams due to one crossing.



determining the condition of the concrete and to detect any differences that might be evident in the concrete from different areas of the deck. A total of 109 nominal 4-in-diameter cores was taken from nine bridges. Five of these bridges had been in service for some time when cored, but no traffic had been permitted on the new parts of the other four. More than half of the cores were taken from the midspan disturbed areas, as defined earlier, and the others were taken from the undisturbed regions at or near supports. A schedule of the cores is given in Table 1, and Figure 8 shows locations of cores removed from one deck.

Each core was carefully inspected for visual evidence of flaws. Some of them were then sawed through perpendicular to their axes, polished, and inspected through a magnifying glass. These polished surfaces were then treated with a crack-detecting substance that was fluorescent under black light; they were then studied further for crack patterns. The cracks on one of these surfaces have been inked in for emphasis in Figure 9.

Other cores were trimmed to a length of about 4 in to include the reinforcing steel from either the dowel bar crossing the break line or the reinforcing steel from the top or bottom mat. These cores were immersed in red ink and subjected to 14 psi negative pressure for about 3 h. They were then broken to reveal the steel and inspected for dye stains that might have penetrated between the steel and the con-

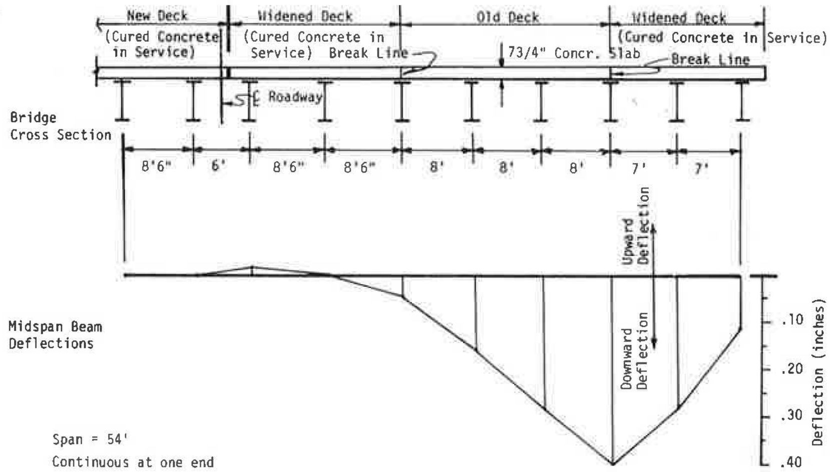


Figure 5. I-45 and FM-517, Dickenson (Houston): maximum midspan beam deflections for one crossing.

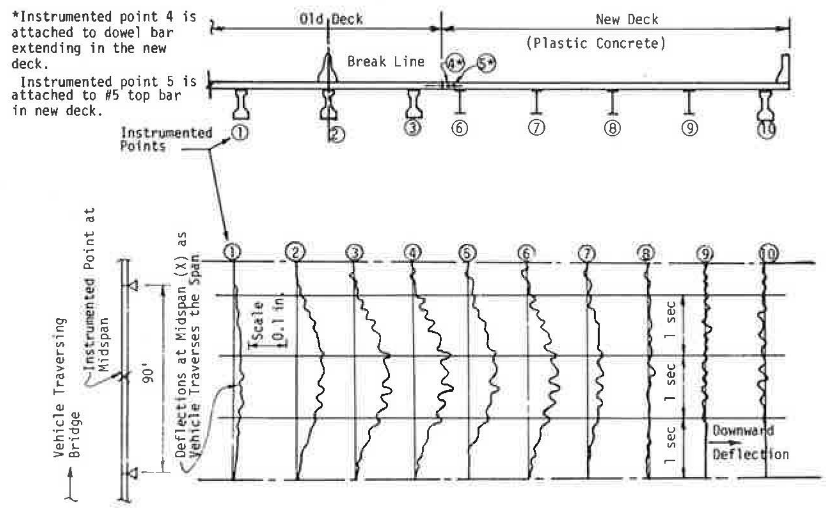


Figure 6. US-75 and White Rock Creek, Dallas, northbound (90-ft span): midspan deflections of beams and rebars due to one crossing.

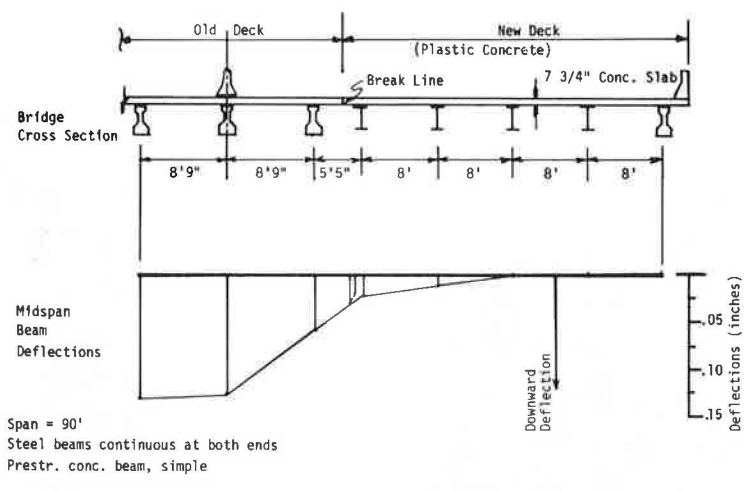


Figure 7. US-75 and White Rock Creek, Dallas, northbound (90-ft span): maximum midspan beam deflections for one crossing.

crete. Figure 10 shows a specimen being broken to reveal the steel; the grooves in the core were sawed after removal of the core from the vacuum pot.

Ultrasonic pulse-velocity soundness tests were made on 15 cores by passing a 50-kHz longitudinal pulse through polished surfaces on the ends. Some of the cores came from disturbed regions; the others were taken from undisturbed areas. Following the ultrasonics, compression-strength tests were made on these cores.

Laboratory Beams

Five laboratory beams were fabricated and tested to gather further information on conditions that were revealed in the field study. The beams, designed to represent a transverse strip of bridge-deck slab, were 7 in deep, 12 in wide, and 10 ft 8 in long. One was not reinforced, and the other four were reinforced as shown in Figure 11. The beams were cast and tested in the forms shown in Figure 11. The

Table 1. Schedule of bridge-deck cores.

Bridge Location	Number of Cores: Disturbed Area	Number of Cores: Undisturbed Area	Beam Type[a]: Old	Beam Type[a]: New	Core Marks
I-35 and Avenue D, Temple	7	5	C-Stl	C-Stl	ADTD1-7 ADTU1-5
I-35 and Atchison, Topeka, and Santa Fe Railroad, Temple	7	5	C-Stl	C-Stl	RRTD1-7 RRTU1-5
I-45 and FM-517, Dickenson (Houston)	7	5	C-Stl	C-Stl	F7HD1-7 F7HU1-5
I-45 and FM-519, League City (Houston)	7	5	S-PC	S-PC	F9HU1-7 F9HU1-5
I-10 and Dell Dale Avenue, Houston	8	4	S-PC	S-PC	DDHD1-8 DDHU1-4
US-75 and White Rock Creek, Dallas					
Southbound	10	3	S-PC	C-Stl	WRDD1-10 WRDU1-3
Northbound	7	3	S-PC	C-Stl	WRDLD1-7 WRDLU1-3
US-84 and Leon River, Gatesville	10	6	OH-Stl	OH-Stl	LRGD1-40′ LRGU1-56′
TX-183 and Elm Fork, Trinity River, Irving (Dallas)	10	0	C-Stl	S-PC	EFDD1-10

[a]C = continuous; S = simple; OH = overhanging; PC = prestressed concrete; Stl = steel.

Figure 8. US-75 and White Rock Creek, northbound: core locations.

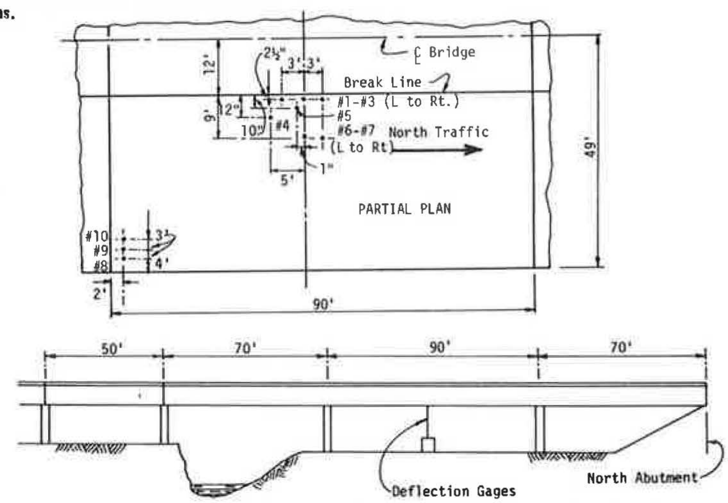
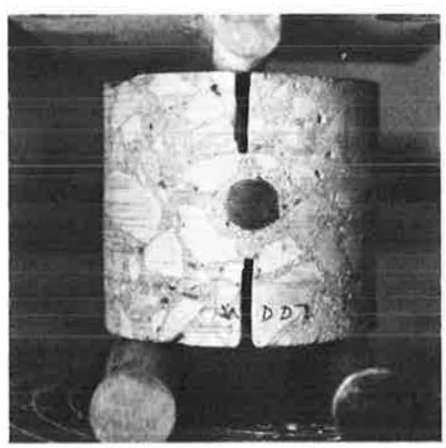


Figure 9. Trace of cracks on polished surface of 4-in-diameter concrete core.

Figure 10. Grooved core prepared for inspection of dye penetration.



vertical sides of the forms were cut into 6-in segments to permit flexibility during testing. The forms were supported by flexible supports at one end and at midspan to simulate bridge beams. The other end was loaded by a pulsating ram to simulate the behavior of the bridge slab during the crossing of the bridge by truck traffic. Figure 12 shows some details of the test setup. Each of the beams was deflected periodically by the ram, beginning at the time of casting and lasting until the concrete was 24 h old.

The first four beams, including the one not reinforced, were deflected 0.25 in at the unsupported end at 5-min intervals to simulate heavy truck traffic. The fourth beam of this series was subjected to a 0.020-in, 6-Hz end vibration superimposed on the 5-min-interval 0.25-in deflection. The end deflection of the fifth beam was reduced to 0.15 in, and it had no superimposed vibration.

The five beams were inspected continuously during testing to detect any cracks that might develop on the top surface, which was the only exposed surface, since the beam remained in its mold during the testing. After the close of the tests, 3-in-diameter cores were taken from each of the beams, some in cracked areas and some through the dowel and reinforcing bars.

RESULTS AND DISCUSSION

Visual Inspection of Bridges

All bridges except one had dowels crossing the break line. The joint in the one exception was open due to the differential deflection of the deck on the two adjacent sides. The joints in all of the other bridges were in excellent condition. Short lengths of hairline cracking along the joints were visible in a few of the bridges, but there was no serious spalling at any of them. The general condition of the decks was good. Crack patterns were generally the same on the new portions of the decks as those on the older portions. No deterioration was found that could be attributed to construction techniques during widening.

Deflection Measurements

Although the deflections were used only for curvature calculations, it is interesting to note that the minimum span-to-deflection ratio was approximately 1600, and the maximum was approximately 8100. The former occurred in the end span of a 54-72-54-ft three-span continuous steel beam unit on an Interstate route. The latter occurred in a prestressed concrete beam bridge that had a 90-ft span. The vibrational frequencies, counted from the oscillographs, ranged from 3.5 to 10 Hz. The frequencies, along with other information, are given in Table 2.

The laboratory beam deflections were used to calculate the beam curvature, which was designed to simulate the transverse curvature of the bridge slab. The procedure used in calculating curvature assumed that an arc of a circle passed through three successive points on the deflected slab. The three points used for the bridge slab were (a) the beam at the break line, (b) the adjacent beam toward the outer edge of the bridge, and (c) the next outer beam. Corresponding points for the laboratory beams were the two flexible supports and the loaded end; however, deflections measured at closer intervals were used in calculating curvature. The curvature found in this manner was checked against a parabolic fit, and the two were almost indentical. Curvature for the bridges is shown in Table 2.

The maximum curvature in fresh bridge concrete was found to be 0.114×10^{-4}/in. It occurred in the 7.75-in deck of the TX-183 Elm Fork, Trinity River, bridge. No longitudinal cracking at the point of calculated maximum curvature was found in the concrete of that new deck. Hilsdorf and Lott (4), reporting on statically deflected 6-in-thick reinforced concrete slabs in the laboratory, found appreciable cracking at 2-4.5-h age when curvature was 5×10^{-4}/in.

Figure 11. Layout of reinforcement and formwork.

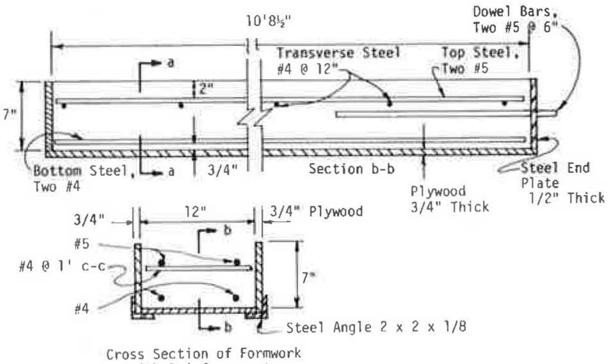


Figure 12. Side elevation of loading frame and test specimen details.

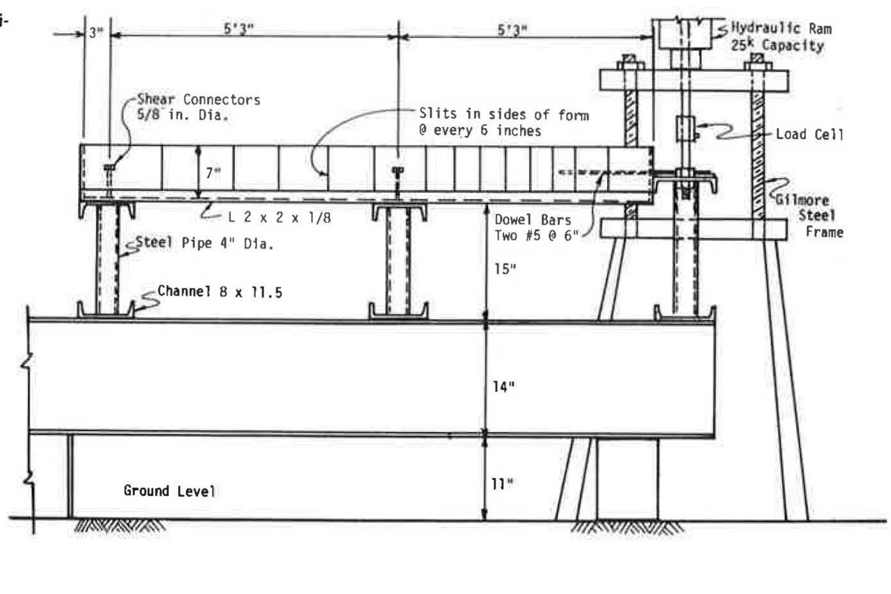


Table 2. Natural frequencies of bridges and maximum transverse bending curvature.

Bridge Location	Beam Type[a] Old	Beam Type[a] New	Span (ft)	Natural Frequency (Hz)	Maximum Differential Deflection[b] (in)	R_{min}[c] (in)	$(1/R_{min}) \div (1/10^5$ in)
I-35 and Avenue D	C-Stl	C-Stl	60	5	0.032	465 333	0.215
I-35 and Atchison, Topeka, and Santa Fe Railroad	C-Stl	C-Stl	70	5	0.041	300 976	0.332
I-45 and FM-517	C-Stl	C-Stl	54	5	0.120	405 140	0.247
I-45 and FM-519	S-PC	S-PC	110	3.5	No traffic on old deck		
I-10 and Dell Dale Avenue	S-PC	S-PC	87	4	0.060	118 580	0.843
US-75 and White Rock Creek							
Southbound	S-PC	C-Stl	50	10	0.032	429 684	0.233
Northbound	S-PC	C-Stl	90	5	0.058	120 869	0.827
US-84 and Leon River	OH-Stl	OH-Stl	67.5	4.5	0.058	145 455	0.688
TX-183 and Elm Fork, Trinity River	C-Stl	S-PC	50	6	0.040	87 803	1.139

[a] C = continuous; S = simple; OH = overhanging; PC = prestressed concrete; Stl = steel.
[b] Maximum differential deflection between outside existing beam and adjacent new beam under new deck.
[c] R = radius of curvature of deck in transverse bending.

In a theoretical elastic model, concrete curvature is inversely proportional to the distance from middepth to the top of the slab, one-half the thickness. It appears reasonable then that the slab thickness would have some bearing on the curvature of fresh concrete. In order to have a fair comparison between the curvature in Hilsdorf and Lott's work and that for the bridge, the former curvature must be modified to account for the different thicknesses. Assuming that the strains were the same at cracking in the two cases, Hilsdorf and Lott's curvature must be multiplied by 0.75 for comparison of values. If that is done, their curvature is about 33 times the maximum given above for the bridges.

Data from the laboratory beams showed that a curvature of approximately 0.36×10^{-4}/in was required to crack the 7-in-thick beams. This is about three times the maximum curvature found in the bridge decks. This finding and the comparison made with Hilsdorf and Lott test information indicate that cracking of fresh concrete on a bridge deck due to passage of traffic is not likely if the bridges are similar to those of this study.

Core Study

Examination of the bridge cores revealed numerous microscopic to hairline cracks, but the patterns were random and the depths were, for the most part, shallow. The few that extended through the slabs were attributed to shrinkage or a combination of shrinkage and flexure from traffic. Twenty-one of 55 randomly cracked cores were taken from undisturbed areas, and 34 came from disturbed areas. The 21 cores make up 58 percent of all undisturbed cores, and the 34 make up 47 percent of the disturbed cores. It is concluded from this that random cracking was not caused by traffic disturbance during placement and curing of the deck concrete. There is no indication that transverse bending caused longitudinal cracking in the decks.

The laboratory beams did crack at sections of maximum negative bending moment at approximately the age of setting, but their curvatures were far in excess of those in the bridge concrete. The cracks in the reinforced laboratory beams extended no further than the top steel, and it is most unlikely that if cracks did form in the fresh concrete of the decks, they would extend deeper than the top mat of steel.

Neither the pulse velocity measurements nor the compression tests revealed any differences in soundness between disturbed and undisturbed cores.

An exceptional core from the break line of the 90-ft center span of a three-span continuous unit provided the clearest evidence found in the entire study of disturbance in the fresh concrete. The top surface was cracked in the general direction of traffic, and the interface of steel to concrete had the appearance of puddled plastic concrete. All traces of bar deformation marks were lost. It is believed that intermittent deflections from traffic caused puddling of the fresh concrete. This puddling formed a layer of uncemented particles that was flushed out by the water from the core drill. Figures 13a-13d show details of the core.

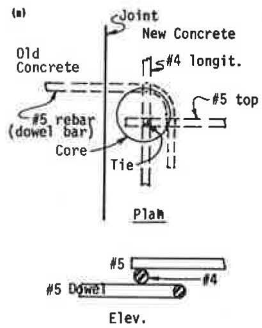
The bridge from which this core was taken had a dowel-bar detail that was different from any other in the study, and no record has been found of its use in any other bridge. This dowel was bent 90 degrees in the horizontal plane on emerging from the older concrete, and it was embedded when the new concrete was placed. In some spots either longitudinal steel or transverse steel or both were tied to or rested on the bent dowel prior to placement of the new concrete. The bent dowel can be seen in Figure 13. The dowels in all of the other bridges ran straight across the joint and were not bent. No problems were found with the straight dowels.

Four other cores from the bent-bar dowel bridges (they were twin structures) showed evidence of differential movement between the steel and the concrete. All of them were located within 12 in of the break line. Puddled areas or poor bar imprints were found at the dowel or at steel that was tied directly or indirectly to the dowel. No cores located more than 12 in from the joint showed any distress.

Under vacuum, dye penetrated around the bars of three of the four cores mentioned in the paragraph above. One of these is seen in Figure 14. In all of these, penetration along with poor bar imprints indicated poor or total absence of bond at the particular location. Some dye penetrated a fraction of an inch along the ends of all bars cut by the drill in coring, but bar imprints were good in all except those noted above.

A study by Larnach (5) in 1952 found that revibration of partially set concrete might cause some loss in bond but that vibration applied directly to the steel in fresh and partially set concrete had little effect on the bond. No discernible differences in the deflections of mat steel and fresh concrete were found in the present study, and none was shown from the study of the cores. This would indicate that there were no problems with the steel vibrating differently than the concrete, except in the situations described above. Only movement of the steel at the break line caused problems, and from Larnach's work one can say that such movement caused the problems after the concrete had partially set.

Figure 13. Core WRDD10F from southbound traffic bridge, US-75 and White Rock Creek: (a) layout, (b) side view, (c) lower section, and (d) upper section.



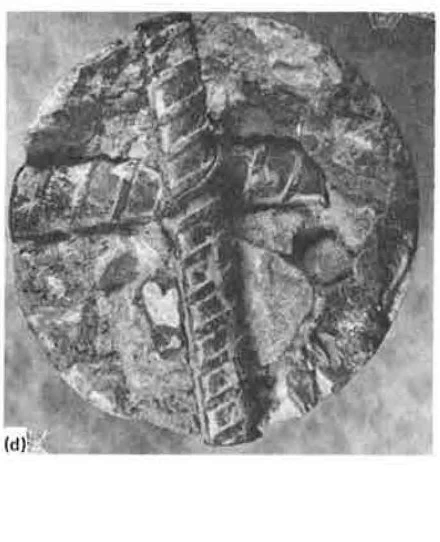
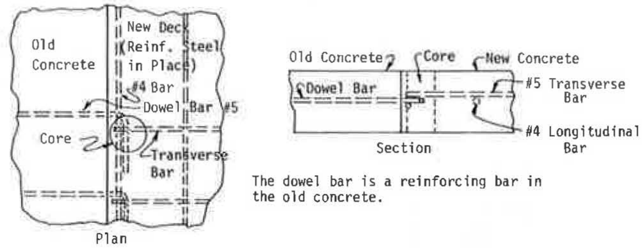
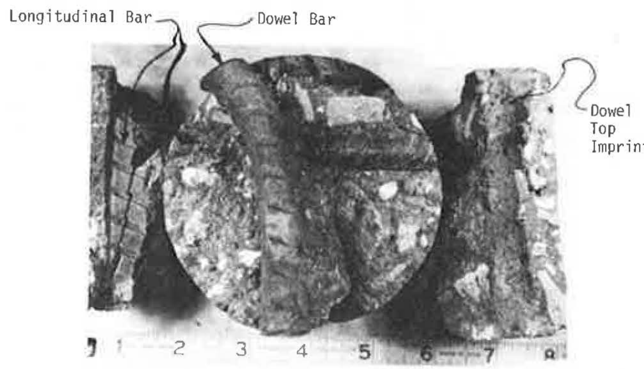


Figure 14. Core WRDD4: (top) schematic indicating location, (bottom) actual dowel bar and imprint.





CONCLUSIONS

A study was made of bridges that were widened while normal traffic was maintained in lanes adjacent to fresh concrete used for widening or staged reconstruction. Bridges that had been in service for years and bridges that were under construction were studied. Laboratory beams designed to simulate a new deck placed and cured during normal traffic use were studied. From the results of the study, the following conclusions may be stated for bridges made of prestressed concrete and steel beams spanning up to approximately 100 ft:

1. No evidence of problems in concrete placed and cured while traffic was maintained was found in bridges that had been in service for years after the new concrete had been placed.
2. Traffic can be maintained during placement and curing without causing flexural problems in the fresh concrete.
3. Voids develop in fresh concrete around dowels bent at a right angle in a horizontal plane on emerging from the old concrete. No surface evidence was found that these voids cause problems in the performance of the deck. No such voids were found in dowels that extended into the new concrete without bends.
4. Vibrations caused by normal bridge traffic have no detrimental effect on the concrete, the reinforcing steel, or the interaction between the reinforcing steel and concrete.

RECOMMENDATIONS

Extend all dowel reinforcing bars approximately 24 bar diameters straight into the new deck area. Lap splice these dowels 20 bar diameters with the reinforcing bars that serve as transverse reinforcing for the new concrete. Tie the top-mat steel system to the dowels to prevent relative vertical movement between the dowels and the top-mat steel. This detail will provide a space between the end of the top rebars and the break joint for a temporary filler, sometimes used in construction by stages, and sufficient lap length for transfer of flexural stress. The ties are needed to ensure that the dowels and the rebars move in harmony, thus preventing the occurrence of voids found at the bent dowels.

ACKNOWLEDGMENT

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Cathodic Protection of Continuously Reinforced Concrete Pavement

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Some sections of continuously reinforced concrete pavement (CRCP) in Minnesota are experiencing a spalling-type deterioration caused by corrosion of the reinforcing steel. In an attempt to develop a method of stopping this corrosion, a cathodic protection system was designed and installed along a 1000-ft section of Interstate CRCP just north of St. Paul. High silicone chromium iron alloy anodes energized by a constant-current output rectifier were placed at the edge of the 10-ft bituminous shoulder at 50-ft intervals. On half the project, the anodes were buried in a trench that was backfilled with coke breeze. On the other half of the project, canisters containing the anodes packed in coke breeze were placed in post holes and backfilled with additional coke breeze. It appears that both types of installation are providing at least partial cathodic protection to the pavement.

During the late 1960s, the Minnesota Department of Transportation (MnDOT) constructed considerable mileage of continuously reinforced concrete pavement (CRCP). In rural areas pavement thickness was generally 8 in and in urban areas 9 in. The steel reinforcement was 0.6-0.7 percent and was deformed wire mesh or deformed reinforcing bar.

During the past seven years, an increasing number of CRCP sections in Minnesota have begun to show a spalling-type deterioration. The frequency of this deterioration progressed from isolated and random in 1975 to widespread and concentrated on certain pavement designs by 1978. Pavements showing the most severe spalling are of the two-course construction type with a steel-to-concrete ratio of 0.60-0.65 percent. Reinforcement used was deformed wire mesh with specified clear cover of 2-4 in. In most cases the steel had been placed at the minimum specified cover of 2 in.

In 1976, a survey of a deteriorating section of CRCP on I-94 between downtown St. Paul and downtown Minneapolis was conducted. Tests performed during this survey included cover measurements, delamination detection, and half-cell potentials for corrosion detection. A visual survey was also performed. Strong evidence was found to support the theory that corrosion-induced spalling was occurring. Survey results showed that corrosion-potential measurements were generally at or well above the corrosion threshold of 350 mV relative to the copper sulfate electrode. Many measurements were in the range of 500-600 mV of active corrosion. The maximum potential noted on corrosion-damaged bridge decks in Minnesota was also about 600 mV. Delamination surveys revealed that 13 percent of the pavement tested was delaminated. Reinforcing steel cover generally measured 1.75-2.25 in.

At the time of the survey on I-94 (August 1976), nearly all noticeable spalling-type deterioration was confined to the oldest sections of CRCP in the metropolitan area. However, it was feared that eventually the problem could become very extensive.

A research study was undertaken to develop procedures for stopping or at least reducing the rate of corrosion in CRCP. One known method for preventing corrosion of steel is cathodic protection. It has been used successfully to protect buried pipelines for many years. More recently it has proved to be an effective means of arresting corrosion of rebars in concrete bridge decks (1,2). By using procedures developed in these two applications as a starting point, a design was developed to cathodically protect a section of CRCP.

The location selected for the cathodic protection installation is on I-35W a few miles north of St. Paul. Factors considered in making the selection were pavement type, traffic, state of deterioration, and convenience for monitoring. The pavement type is one that has exhibited the most frequent and