Durability of Steel-Formed and Sealed Bridge Decks

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A 2-year research project was carried out to investigate the freeze-thaw durability of concrete bridge decks cast on steel forms, which remain in place after construction, and sealed on the top surface with a waterproof membrane. The long-term durability of the forms was also studied. Laboratory freeze-thaw tests equivalent to a winter season and one winter of outdoor exposure tests were carried out on simulated bridge-deck slabs. These slabs covered all combinations of the following variables: (a) form type (wood and steel) and (b) surface treatment (none, linseed oil, and waterproof membrane). In addition, 25 bridge decks with steel forms and waterproof membranes and 1 bridge deck without steel forms but with a membrane were inspected. The bridge decks ranged in age from 1 to 13 years, and all but six were 8 or more years old. By using a variety of inspection techniques that ranged from visual examination to pulse velocity measurement, it was determined that steel-formed bridge decks with surface sealing are no more prone to freeze-thaw deterioration than wood-formed decks. Generally, the forms themselves were found to be in good condition when designed for proper deck-surface drainage.

The primary goals of the research described in this paper were to evaluate the effect of steel bridge forms on the durability of concrete bridge decks and to evaluate the durability of the forms themselves. The extensive use of steel beams for bridge decks, especially along the eastern seaboard, coupled with current Federal Highway Administration guidelines that require the use of waterproof membranes for resurfacing bridge decks provided the impetus for the first goal. The question of form deterioration due to corrosion has often been raised; however, few definitive data can be found in the literature. In a study of 20 steel-formed bridge decks in New York State (1, 2), it was reported that less than 5 percent of the total area of sections of steel deck forms removed for inspection showed any signs of corrosion. A 1970 survey of all 4-year-old bridge decks in Pennsylvania (249 bridges, 146 of which were constructed with steel deck forms), did not reveal any instances of form corrosion problems (3). However, the question of form durability persists.

The goals of this research were pursued in a research plan that included two major phases. The first phase involved exposing sections of simulated bridge decks to laboratory and field freeze-thaw conditions. The second phase consisted of a program for inspection of bridge decks.

SIMULATED BRIDGE DECK SPECIMENS

Experimental Design

A complete factorial experimental design was employed in which the variables consisted of three levels of surface treatment and two levels of form type. Each condition was replicated twice to enhance the statistical significance of the experimental results. Therefore, the total number of specimens was 12.

Test Specimens

The test slabs were 76.2 cm (30 in) wide by 91.4 cm (36 in) long by 19.1 cm (7.5 in) deep. (This is the depth to the top of the ridges on the steel forms, i.e., minimum slab thickness.) The side forms consisted of 5.1 by 25.4-cm (2 by 10-in) timbers coated on the inside with 1.59-mm (16-gauge) galvanized sheet metal. The side forms remained in place throughout the testing to prevent moisture loss through the sides of the specimens. Calking compound was applied to the perimeter of the slabs to seal any separation that occurred between the side forms and the slabs. The bottoms of the wood-formed slabs were formed with plywood that was removed after curing. The steel-formed slabs were formed on the bottom sides with standard 0.80-mm (22-gauge) galvanized bridge forms with a 16.5-cm (6.5-in) pitch and a 5.1-cm (2-in) corrugation depth.

Relative humidity wells, to accept the Monfore gauge (4), were precast in each specimen by using brass rods and sleeves. Steel reinforcing bars were placed in two layers; 1.3-cm (0.5-in) number 4 bars were placed transversely, and 1.6-cm (0.6-in) number 5 bars were placed longitudinally. The lower mat was positioned 2.5 cm (1 in) above the steel- or plywood-formed bottom, while the upper mat was placed 5.1 cm (2 in) below the top of the form. Steel chair supports were used for position-

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ing and supporting the reinforcing steel.

Copper-constantan thermocouples were placed in seven specimens. One thermocouple was placed at middepth in each of the two replicate specimens to represent each combination of design parameters. One slab received three thermocouples: one at 2.5 cm (1 in) below the surface, one at middepth, and one 16.5 cm (6.5 in) below the surface or 2.5 cm (1 in) above the bottom surface.

The concrete for all 12 test specimens was mixed in one batch at a commercial ready-mix plant and delivered by a transit mix truck to eliminate batch-to-batch variation. Pennsylvania Department of Transportation class AA concrete (the type used in bridge decks in Pennsylvania) was specified. Slump and air content were measured before the slabs were placed and at the third points on the placement schedule (i.e., between placement of the fourth and fifth and the eighth and ninth slabs). A final determination for air content was made after all slabs had been placed. The air content determinations were slightly below specification. For the purpose of this experiment, this was not necessarily detrimental since it served to accentuate any differences in freeze-thaw behavior among the specimens.

All specimens were moist cured with wet burlap for a 28-day period, and polyethylene sheeting was used to retard evaporation. After completion of the curing period, eight slabs were given surface treatments in accordance with the research plan. The surfaces of these slabs were allowed to dry in the laboratory for 1 day to enhance the surface treatments.

A 50-50 mixture of boiled linseed oil and mineral spirits was applied to four slabs: two wood-formed and two steel-formed slabs. Application was accomplished by a hand sprayer. The first coat was applied at a rate of 1.0 L/m^2 (0.025 gal/ft^2) while the second coat was applied at the reduced rate of 0.6 L/m^2 (0.015 gal/ft^2). One-day drying time was allowed between applications.

A preformed moisture-proof membrane was applied to two wood-formed and two steel-formed slabs. The slabs first received a coating of liquid mastic supplied by the manufacturer. The membrane was then applied after the tack coat had dried. A special caulk was then applied to all outside edges to ensure a watertight seal. A hot mix of asphaltic concrete (PennDOT Specification ID-2) that was 3.8 cm (1.5 in) thick was applied and compacted over the membrane. The remaining four slabs, two steel-formed and two wood-formed, received no surface treatment.

Laboratory Freeze-Thaw Tests

To accomplish the objective of this research, certain criteria had to be established. First, it was necessary to ensure that the test slabs were saturated at all times, thus providing the worst possible condition one would expect in nature. This was accomplished by continuously ponding water on the slab surfaces and by monitoring internal hygrometric conditions throughout the test. A Monfore gauge for relative humidity was used for monitoring purposes. Second, the internal freezing of each slab had to be ensured, and this was accomplished by monitoring internal temperatures of the slab by using thermocouples and a temperature recorder. Third, if there was any freeze damage, it had to be detected and monitored, and this was accomplished by the use of a pulse-velocity meter for detecting cracking and delamination and by the use of linear variable differential transformers (LVDTs) for volume changes during freezing.

Before the freeze-thaw process was begun, initial readings were taken with the Monfore gauge and with the pulse-velocity meter to establish data values. The specimens were subjected to 75 freeze-thaw cycles under controlled conditions in the laboratory. The winter climate of Pennsylvania was used as the basis for setting the number of freeze-thaw cycles, since this climate is considered severe in the freeze respect. The decision to use 75 freeze-thaw cycles was based on data from previous research (5) that showed this number of freeze-thaw cycles to be the maximum found in Pennsylvania in an average winter.

Temperatures within the slabs and the air temperature of the cooling chamber were constantly monitored. The cooling rate within the slabs was approximately 1.4°C/h (2.5°F/h). Once the slabs had reached -3.9°C (25°F), the freeze cycle was considered complete and was reversed. The heating rate within the slabs was 2.8°C/h (5°F/h), and thawing was considered complete when slab internal temperatures reached 1.7°C (35°F).

To verify that the test slabs remained in a saturated condition throughout the test period, internal relative humidity measurements were taken in each slab 2.5 cm (1 in) from the top, the bottom, and at middepth before freeze-thaw testing began, after the fifteenth freeze-thaw cycle, and every 10 cycles thereafter. According to the literature (6), relative humidity values in excess of 80 percent are indicative of saturation of the capillary system but not of the entrained air voids, which, because of their large size relative to the capillary pores, saturated with great difficulty. In other words, the gel and capillary pores hold water tenaciously and completely outstrip the larger air voids in competition for available water. When the gel and capillary pores are saturated (a feat they are able to accomplish in the presence of liquid water or by capillary condensation at relative humidities above 80 percent), the air voids are prevented from filling except under hydrostatic pressure (e.g., during freezing). Throughout the laboratory test phase, the relative humidities in the test wells were never below 95 percent, and most of the time they showed 100 percent relative humidity (RH) or the presence of free water.

Pulse-velocity measurements were obtained before the freeze-thaw cycles commenced, after the fifteenth freeze-thaw cycle, and every 10 cycles thereafter. These readings were taken to evaluate the extent of progressive freeze-thaw damage in the slabs. Pulse-velocity decreases with increasing cracking and deterioration in concrete because of the longer flight paths of the sonic waves passing around discontinuities when the distance between the transmitting and receiving transducers remains constant. Pulse-velocity readings were taken at five locations on each slab—near the corners and at the center. The results that include the subsequent field exposure tests are shown in Figure 1. It is quite evident that, with the possible exception of slab 9, there are no consistent trends in a degradation of pulse velocity. In slab 9, the steel bridge form had become debonded from the concrete, and this created an air space, which readily explains the drop in pulse velocity.

Length changes of the test slabs during freezing that were measured parallel to the thickness of the slabs were obtained by using the highly sensitive LVDTs. Equipment limitations permitted monitoring at only one point on each of six test slabs that represented each of the six combinations of test variables.

At the instant of freezing, all concrete containing moisture expands because of the hydraulic pressure generated by the freezing water. This expansion, termed "dilation," is transitory and completely recoverable in concrete that is immune to frost damage. In frost-susceptible concrete, however, dilations are large and mostly nonrecoverable; i.e., permanent deformation and cracking results. The critical value of dilation is a function of the elastic properties of the concrete (modulus of elasticity, tensile strength, and Poisson's ratio), but for normal concretes it is approximated by the relation

$$D_c = 70L \tag{1}$$

where

D_c = critical dilation (micrometers) and
L = dimension of concrete in the direction that length changes are measured (meters).

Therefore, for the case at hand, $D_c = (70)(0.19) = 13.3$ μm (525 μin). Thus, dilations in excess of 13.3 μm (525 μin) must be consistently encountered if the concrete has failed because of freeze-thaw action. Larson and Cady (7) give a more detailed discussion of dilation as a measure of frost damage. Length change measurements were made on some or all of the instrumented slabs during the cycles given in Table 1. Considerable difficulty was encountered because of condensation and subsequent freezing of moisture in the cores of the LVDTs. Despite the efforts to circumvent this problem, the data obtained are somewhat meager. As given in Table 1, the dilation for any case did not exceed or even closely approach the critical value of 13.3 μm (525 μin). These data support the findings of the pulse-velocity determinations and confirm the absence of frost damage to the test slabs.

Field Tests for Exposure

After the 12 slabs were tested by simulating a winter season of freeze-thaw cycles in the laboratory, they were moved out-of-doors for field exposure tests. The major objective of this phase of the project was to monitor the performance of the slabs during one winter of actual field exposure. The information gathered during the 1974 to 1975 winter included weather data (daily high-low temperature, precipitation, and depth of snowfall), moisture content, pulse velocity, and visual inspection. Deicer salt (calcium chloride) was applied to the slabs at a rate of 141 kg/lane-km (500 lb/lane-mile) or about 0.039 kg/m^2 (0.008 lb/ft^2) as dictated by weather conditions. In all, 14 salt applications were made.

The site chosen for outdoor exposure of the test slabs is the oval-shaped, 1524-m (5000-ft) long, paved test-track facility that is situated approximately 8 km (5 miles) northeast of the main campus of the Pennsylvania State University in open, rolling country. A reinforced concrete slab was constructed at the site, and the test slabs were supported about 0.4 m (16 in) above the slab on concrete blocks.

From daily temperature data, it was determined that 39 freeze-thaw cycles occurred during the period from August 9, 1974, to March 14, 1975. This number of cycles appeared to be somewhat lower than the 57 freeze-thaw cycles that were determined to occur in an average winter in this part of Pennsylvania. However, it is reasonable to assume that 5 to 10 additional freeze-thaw cycles occurred during the remainder of the 1974 to 1975 winter after March 14. A freeze-thaw cycle is defined by the dropping of air temperature below -3.3° C (26° F) followed by a rise in temperature above 1.1° C (34° F). Total precipitation was about 15 percent above normal, and snowfall amounts were about 33 percent above normal during the test period.

It was originally intended that the degree of saturation within the test slabs would be monitored weekly during the winter season by using the Monfore relative humidity probe. However, some difficulties were encountered with the probe; e.g., probe test wells were plugged with insects, and a probe was disabled. Therefore, an effective acquisition of internal RH data was prevented until January. Nonetheless, all data obtained, as in the laboratory test phase, revealed internal relative humidities in excess of the critical 80 percent level necessary for the maintenance of capillary saturation.

Periodic pulse-velocity determinations were made on the test slabs to determine presence and extent of any freeze-thaw damage to the slabs during the exposure period. As shown in Figure 1 and mentioned earlier, the pulse-velocity determinations indicated no evidence of freeze-thaw damage.

BRIDGE INSPECTIONS

The laboratory and field exposure tests previously described were carried out to evaluate potential freeze-thaw damage on simulated bridge-deck slabs under controlled conditions. To provide the added dimensions of traffic loading and longer periods of exposure, a program of field inspection of bridges having steel forms and membrane-sealed deck surfaces was undertaken.

It was decided that the oldest bridges having the same (within the limits of practicality) superstructure type and traffic loadings would be selected from diverse geographic locations. Also, the ages of the bridges should ideally be about the same. Unfortunately, for reasons that will be discussed, it was necessary to compromise some of these initial selection criteria.

Selection of Inspection Sites

Although no problem existed in finding sufficient numbers of bridges having steel bridge forms, the extent of the use of membrane waterproofing systems in conjunction with steel bridge forms was not known. Also, although membranes applied as liquids (polymeric resins and epoxy modified bitumens) have been in use for some time, the preformed materials for sheet membranes are fairly recent newcomers to the bridge-deck construction scene. The latter class of membrane materials is currently recommended by the Federal Highway Administration for waterproofing bridge decks. Therefore, as a starting point, the three leading manufacturers of preformed membrane materials were contacted to ascertain which highway agencies were using the membranes. The contractor on NCHRP project 12-11, Waterproof Membranes for Protection of Concrete Bridge Decks, was also contacted to obtain information on potential bridge inspection sites. A consulting engineering firm that pioneered in work in this field was also contacted. As a result of these initial inquiries, five state highway agencies (Massachusetts, New York, Pennsylvania, Vermont, and Virginia) and the New Jersey Turnpike Authority were contacted, and two facts became immediately evident. First, bridges that were sufficiently advanced in age (at least 5 to 10 years) to provide meaningful results had membranes that were not of the preformed type but that had been applied as liquids. Second, most highway agencies that have extensively used steel bridge forms for long periods of time have not used membrane sealing systems (e.g., Pennsylvania), and most highway agencies that have used the membranes have not extensively used steel bridge forms (e.g., Vermont). In the final analysis, sufficient numbers of bridge decks having both the steel forms and waterproof membranes to warrant further consideration were found to exist only in New York State and on the New Jersey Turnpike.

New York State Bridges

The New York State Department of Transportation was

Table 1. Dilations during freezing.

Slab			Dilation (μm) by Cycle												
No.	Form	Treatment	17	21	22	23	24	25	26	29	38	45	51	53	68
2	Wood	None	—[a]	2.5	1.5	—[a]	3.6	3.8	5.8	—[a]	5.1	—[b]	—[b]	—[b]	2.8
3	Wood	Membrane and asphaltic concrete	—[a]	7.6	4.1	5.6	4.1	9.1	5.6	9.1	7.4	—[b]	—[b]	—[b]	5.1
5	Steel	None	8.1	—[a]	—[a]	6.1	1.5	3.3	4.1	—[a]	—[a]	—[b]	—[b]	—[b]	3.0
6	Wood	Linseed oil	—[a]	2.0	1.5	1.5	1.5		—[a]	—[a]	4.3	4.1	9.1	8.6	9.4
7	Steel	Linseed oil	11.2	2.0	4.1	—[a]	—[a]	—[a]	4.6	6.1	—[a]	—[b]	—[b]	—[b]	—[a]
9	Steel	Membrane and asphaltic concrete	12.2	4.1	—[a]	—[a]	1.8	—[a]	—[a]	5.3	—[a]	—[b]	—[b]	—[b]	—[a]

Note: 1 μm = 0.039 μin.

[a]Transducer frozen. [b]Not tested.

Figure 1. Effect of freeze-thaw cycles on pulse velocity for test slabs.

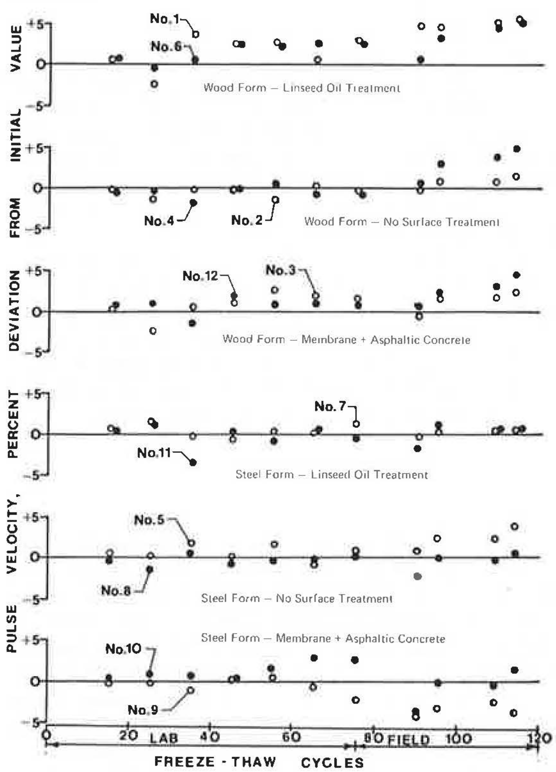
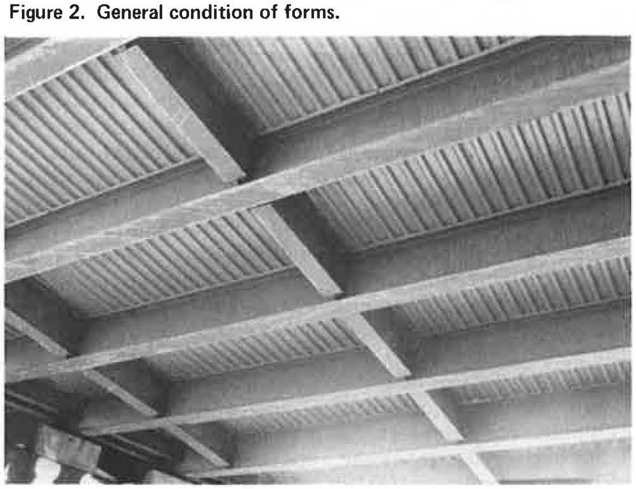


Figure 2. General condition of forms.



able to provide a listing of 34 bridges that had been constructed with the use of steel deck forms and waterproof membranes. The bridges were built during the 1961 to 1963 period and had either polyester resin or epoxy modified coal tar for membranes and were topped with an asphaltic concrete wearing course. Twelve of these bridges, widely distributed across the state, were selected for inspection. Also, one bridge built during the same period and having the membrane system, but constructed with wood forms, was chosen for purposes of comparison.

New Jersey Turnpike

The selection of bridges on the New Jersey Turnpike was more complicated than the selection of bridges in New York State. Three distinct categories of structures existed on the New Jersey Turnpike:

1. Type A—replacements of deteriorated sections of existing decks using steel forms, liquid membranes, and asphaltic concrete wearing courses (this replacement started in 1963);
2. Type B—new construction of decks by using steel forms, liquid membranes, and asphaltic concrete wearing courses (this construction was from 1963 to 1972); and
3. Type C—new construction of decks by using steel forms, preformed membranes, and asphaltic concrete wearing courses (this construction started in 1972).

The type A and type B systems are basically the same as those used in New York State. The difference between A and B is simply that A involved replacement sections of existing bridges and B was new construction. To account for the age factor, the two oldest bridges of type B (built in 1963 and 1966) and five oldest replacement sections of type A (done in 1965) were selected.

Since all of the bridges selected to this point had liquid membrane systems (polyester resin or epoxy modified coal tar), it was felt that a representative sample of steel-formed decks having preformed membrane systems should be selected for inspection in spite of the fact that they would have to be fairly new bridges. Accordingly, six type C bridges were selected. All but two of the bridges selected are simply supported, steel I-beam structures. The other two, both in New Jersey, are continuous plate girders. The bridges ranged in age from 1 to 13 years, and all but six were 8 or more years old.

Inspection Procedures

The rationale behind the inspection procedures adopted for this study was based on the premise that general deck conditions could be reliably assessed by means of a few simple observations. These inspections were not intended to be detailed condition surveys like those that might be conducted by a highway agency before making repairs. Rather, the purposes were

1. To determine if the presence of steel bridge forms in combination with waterproof membranes has any effect on deck durability, especially freeze-thaw susceptibility; and
2. To assess the long-term durability of steel bridge forms.

Except for a few locations on the New Jersey Turnpike, all close inspections of the undersides of the bridges were limited to those areas that could be conveniently reached, i.e., near the abutments. The areas that could not be reached for close inspection were carefully scanned with the use of 7 by 50 binoculars. To the extent that it was practical to do so, the following inspection procedures were carried out on each bridge:

1. Visual examination of the deck riding surface by noting general conditions and presence of cracking, potholing (including incipient failures), and recent patching or repairs;
2. Visual examination of the underside of the deck by noting the general condition of the steel forms and the location and severity of corrosion and its relationship to details of the deck design;
3. Hammer sounding of accessible regions on the underside of the deck to determine the percentage of area that indicated separation (debonding) of the forms from the concrete and insufficient filling or consolidation of the concrete in the valley (flute) portions of the forms; and
4. Pulse-velocity determinations by using the James V-scope to assess the soundness of the decks.

It was not possible to perform procedures 1 and 4 on many of the New Jersey Turnpike bridges because of the unavailability of traffic control that was a prerequisite for topside deck inspection. In addition to the four procedures described above, form panels were removed from five of the New York State bridges and seven of the New Jersey Turnpike bridges. The insides of the forms were examined for evidence of corrosion, and the concrete areas exposed by removal of the forms were closely studied for evidence of freeze-thaw deterioration, insufficient consolidation or segregation, and the presence of foreign materials.

Inspection Results

Deck Surface

In general, the decks were in excellent condition, especially in view of the ages involved. However, because of the inability to obtain maintenance histories of the deck surfaces, observation of the current condition of individual decks cannot, by itself, be considered a valid assessment of the condition of the concrete structural deck. It is known that some of the decks were overlaid with asphaltic concrete subsequent to initial construction. Nonetheless, the general overall lack of potholes, recently repaired areas, or other signs of distress tends to indicate satisfactory performance.

Form Condition

Overall, the forms were in excellent condition, even on the oldest (13-year-old) bridges. Figure 2 illustrates this fact and shows a general view of one of the bridges that exhibited the most severe and extensive corrosion problems of the bridges inspected. Where corrosion was found to exist, it could almost invariably be traced to the drainage of salt-laden waters from the deck surface. Consequently, most of the corrosion observed was found to exist along the facia girders and at the span ends, as shown in Figure 3. A particularly striking example of the effect of deck drainage on form corrosion was found on the four bridges inspected in the Binghamton area in which grating type of drainage features permitted ready access to the forms by water running off the decks, as shown in Figure 4.

Corrosion on the inside of removed sound-form panels was virtually nonexistent. Two of the form panels that were removed displayed a few spots of light rusting, and a third showed evidence of slight corrosion on an area in which sawdust and wood chips had been inadvertently left when the concrete was placed.

Hammer Soundings

A positive correlation, significant at the 95 percent level, was found to exist between percentage of hollow soundings and deck age. This would tend to indicate that the development of hollow areas is progressive with time and, therefore, results from separation of the form from the concrete rather than from the incomplete consolidation of the concrete.

Observation of the concrete that was exposed by the removal of selected form panels revealed 1 area in 12 (about 8 percent) that showed the existence of incompletely consolidated concrete. Since this value exceeds the 6 percent overall average of hollow-sounding areas, it fails to substantiate the debonding theory. However, because the sample size is so small it also does not refute it.

Pulse Velocity

In general, the pulse-velocity values for the outdoor test slabs are lower than those observed for the laboratory test slabs. The four test slabs that had membranes and asphaltic concrete wearing courses had initial pulse-velocity values that averaged 4026 m/s (13 209 ft/s) in comparison with the pulse-velocity values with an overall average of 3529 m/s (11 578 ft/s) for the bridge decks. One reason for this disparity is that the thickness values assumed for the bridge decks and used for the pulse-velocity calculations were quite likely on the low side. The thicknesses used were design values. Also, it is quite likely that overlays were applied to the decks subsequent to construction, and this would add to the deck thickness in some cases. However, this factor was taken into account when known. Another factor considered was that most of the overlay thicknesses used on the bridge decks were much greater than those used on the test slabs, and the overlay layer does modify the pulse velocity. For example, the average initial pulse velocity for the test slabs that had the overlay was over 305 m/s (1000 ft/s) lower than the average for the test slabs that had no overlay. Finally, lower pulse-velocity readings may not necessarily indicate concrete damage—they could also result from form or membrane debonding.

In view of the factors cited above and the fact that the pulse velocity will vary because of differences in the water to cement ratio, the aggregate type, entrained air content, and degree of water saturation of the concrete, it is not feasible to establish a threshold value below which concrete deterioration may be positively assumed to have occurred. However, concrete that is of sound quality should, theoretically, exhibit a pulse-velocity value of 3292 m/s (10 800 ft/s) or greater. Furthermore,

Figure 3. Corrosion of forms near span end and along facia girder.

Figure 4. Corrosion of forms in vicinity of grating type of drainage feature.

Figure 5. Condition of concrete in form removal area.

Figure 6. Sand embedded in concrete surface in form removal area.

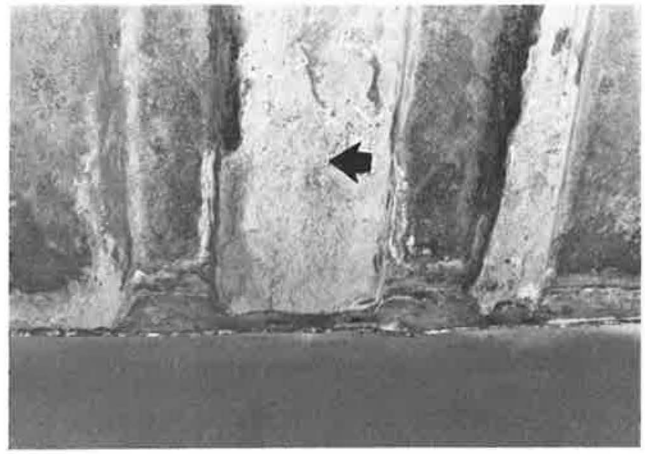


Figure 7. Corroded form and deteriorated concrete at location of foreign material in deck.

on a given bridge deck, large differences in pulse velocities taken at different points should be viewed as an indication that some areas of concrete deterioration may exist. From the use of these guidelines, it appears that the two bridge decks in New York State may contain deteriorated concrete. One of the decks does not have steel bridge forms. It appears that the south end of this deck may be experiencing deterioration of the concrete. However, the north end of this deck gives highly satisfactory readings; therefore, one would judge that the cause of the problem might be corrosion of the reinforcing bar and subsequent delamination rather than freeze-thaw damage, which would tend to be more general. The other suspect deck displays consistently low readings that might indicate freeze-thaw damage. The only way to be certain in either of these cases would be to extract cores for examination. Although it cannot be stated positively that the remaining bridge decks did not suffer freeze-thaw damage, the pulse-velocity values for these bridge decks did not indicate freeze-thaw damage.

Condition of Concrete Exposed by Form Removal

The concrete exposed by form removal was found to be in sound and excellent shape, as shown in Figure 5. Only one instance of incomplete consolidation of the concrete in the valley portion of the form was observed. In two cases, foreign materials left on the forms at the time of concrete placement were found embedded in the concrete, as shown in Figure 6. An unusual instance of form corrosion and concrete deterioration due to a block of wood left on the form is shown in Figure 7.

CONCLUSIONS

Based on the research described in this paper, the following conclusions appear to be warranted:

1. Steel-formed bridge decks, with or without surface sealing, up to 13 years of age are no more prone to freeze-thaw deterioration than wood-formed decks or steel-formed decks without membranes;
2. In carefully controlled laboratory freeze-thaw tests, steel-formed decks and wood-formed decks behaved no differently;
3. Although separation of the steel forms from the concrete occurs in some cases, it apparently has no effect on the durability of either the deck or the forms; and
4. Form corrosion is generally not a problem if the deck design provides proper drainage from the deck surface to prevent contact with the forms.

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