Effects of Concrete Deterioration on Bridge Response

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Two reinforced-concrete T-beam bridges, constructed in 1931 with similar cross-section dimensions and properties, now show major differences in the appearance of their concrete. As a result, inspectors have rated one as having "serious deterioration and not functioning as originally designed" and the other as being in "new condition". Bridge engineers are responsible for translating these subjective evaluations into predictions of the structures' load-carrying capacity, but little information is available to permit quantification of the consequences of concrete deterioration on bridge response. The two bridges were instrumented to determine tension reinforcement stress under an equivalent HS-20 truck positioned on the bridge deck. The results showed no difference in behavior attributable to concrete deterioration. Theoretical justification for this result was established by showing the variation in reinforcement stress expected as a function of compressive concrete loss and reduction in concrete strength. In both cases, the magnitude of the resulting stress change was shown to be small and generally less than the measurement accuracy of the electrical resistance strain gages used on the actual bridges. Differences in concrete properties determined from cores taken from the structural decks corresponded with those in the appearance of concrete in the two bridges. Nevertheless, it was concluded that inspectors did not identify those aspects of structural condition of these bridges that have a significant impact on their safe load-carrying capacity. It is suggested that this result is typical of inspections of a majority of reinforced-concrete T-beam bridges.

Federal Highway Administration (FHWA) national bridge inspection standards (1) require that highway bridges be inspected and rated for load-carrying capacity to ensure the public safety. Procedures and guidelines to assist state and local agencies in this task are given in the American Association of State Highway and Transportation Officials Manual for Maintenance Inspection of Bridges (2). In addition, the load-rating engineer will have available a detailed inspection report on the structure, which generally is prepared in accordance with specific standards (3).

For concrete structures, the qualitative information in the inspection report cannot be used directly to establish a load capacity, and it is the structural engineer's job to quantify the consequences of reported deterioration on the capacity of the bridge. Little information is available to guide this quantification, and, as a result, load ratings of reinforced concrete bridges are believed to be unduly conservative.

This paper analytically examines the probable significance of certain types of deterioration and presents experimental data on the response of two reinforced-concrete T-beam bridges to service loads. Specifically examined are the effects on bridge capacity of reduction in concrete strength and loss of section in the compression zone that result from freeze-thaw damage of non-air-entrained concrete and corrosion of steel reinforcement.

BRIDGES EXAMINED

The bridges discussed in this paper are two of five tested to evaluate the practicality of an inexpensive physical test to establish the safe load-carrying capacity of deteriorated structures (4). The conclusion of that work was that this objective was partially unobtainable, for reasons to be discussed.

The structures considered here were selected because of their similar design dimensions and ages and the difference in the inspection ratings of their primary structural members. Both are T-beam structures constructed in 1931. Their cross sections (see Figure 1) differ only in stem depth (bridge 2 is 2 in deeper) and the presence of parapets on bridge 1. In addition, bridge 2 has a 39.5-ft span compared with 37.5 ft for bridge 1, and both support lines are skewed at 22°. Because of these structural differences, measured bridge responses to applied loads were adjusted (as described in a later section of this paper) to permit comparison.

BRIDGE CONDITION

Both bridges were rated visually by state forces as part of a routine biennial inspection (3). On a scale of 1 (potentially hazardous) to 7 (new condition), the primary members of bridge 1 were rated 3 (serious deterioration or not functioning as originally designed) because of severe concrete deterioration (see Figures 2 and 3), particularly in T-beam stems and in the lower extremity of the fascia beams. By contrast, the primary members of bridge 2 were assigned a condition rating of 7 because of their generally sound appearance (see Figures 4 and 5).

The New York rating scale is similar to a 10-point scale suggested by FHWA (5), which is widely used throughout much of the United States. On the FHWA scale, the primary members of bridge 1 would

Figure 1. Bridge cross sections.

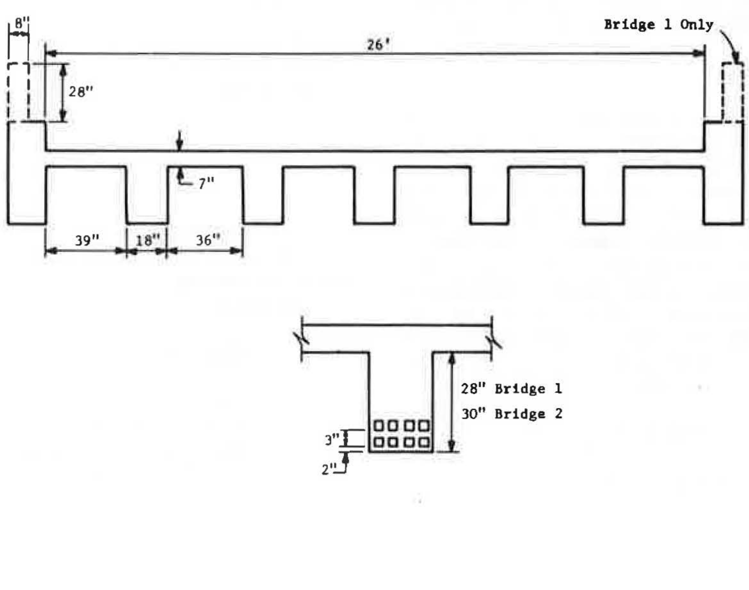


Figure 2. Bridge 1.

Figure 4. Bridge 2.

Figure 3. Deterioration of T-beams in bridge 1.

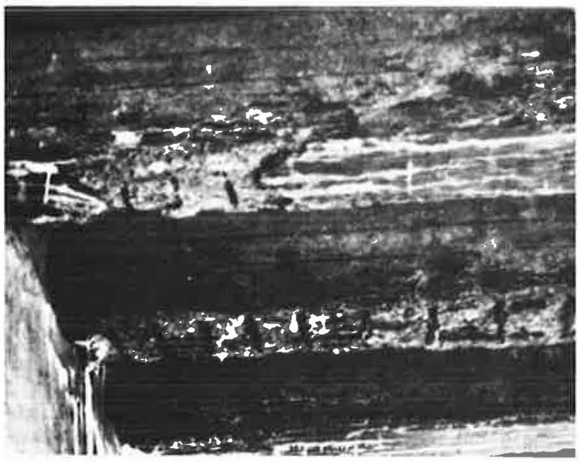


Figure 5. Condition of T-beams in bridge 2.

have been rated 4 (minimum adequacy to tolerate present traffic, immediate rehabilitation necessary to keep open).

Because subsequent load tests (described later) resulted in a nearly identical structural response in the two bridges for the same level and configuration of load, in spite of the gross difference in the apparent condition of their primary structural members, a limited study was undertaken to further define the relative condition of their concrete.

Bridge 1

The overall appearance of the concrete in bridge 1 was one of advanced deterioration. Curbs and parapets were badly eroded due to paste damage (Figure 2), and large areas of concrete had spalled from the underside of T-beam stems and fascia beams (Figure 3), exposing rusted steel reinforcement. The vertical faces of all beams exhibited extensive cracking, which generally paralleled their axes. Efflorescence was common. Measurements of sonic pulse velocity through the stems of three girders yielded values in the 1700- to 4400-ft/s range, which indicates concrete with generally low strength and elastic properties.

The deck was covered with a bituminous wearing course and thus was visible only from the underside. No scaling or spalling was evident, but severe cracking and efflorescence were present in many areas. Four pairs of 4-in-diameter cores drilled through the deck (at about one-quarter span) indicated that the 7-in structural deck was highly fractured throughout and that the cement paste was severely deteriorated locally. Otherwise, the concrete was well compacted but not air entrained. The results of the tests on the deck cores were as follows:

Item	Test Result
Structural deck	
Condition	Highly fractured, severe paste deterioration
Air entrainment	Non-air-entrained
Absorption (%)	5.6 (mean of seven values)
Sonic velocity (ft/s)	Not measurable
Compressive strength (psi)	5195 (single value)
Wearing course	
Condition	Severe paste deterioration
Absorption (%)	6.7 (single value)

The only core segment long enough to test in compression (ASTM C39-72) yielded a strength of 5195 psi. Seven core segments tested for absorption (ASTM C642-75, immersion method) yielded a mean value of 5.6 percent, which is greater than about 80 percent of values measured in cores from other New York bridge decks (6). An earlier concrete wearing surface of undeterminable thickness, probably dating from the bridge's initial construction, was totally disintegrated in all but one of the eight cores. A segment from this single core had an absorption of 6.7 percent.

Bridge 2

By contrast, the concrete in bridge 2 was in excellent condition (Figures 4 and 5). No scaling or spalling was evident on structural elements or on the underside of the deck, and both were free of cracking. Pulse-velocity measurements through the stems of two girders were in the 12 400- to 14 200-ft/s range, which indicates concrete of at least moderate quality.

A set of cores were drilled through the deck, similar to those from bridge 1. Results of tests on these cores were as follows:

Item	Test Result
Structural deck	
Condition	Generally sound, slight paste deterioration
Air entrainment	Non-air-entrained
Absorption (%)	4.1 (mean of eight values)
Sonic velocity (ft/s)	12 020 (mean of four values)
Compressive strength (psi)	5160 (mean of five values)
Wearing course	
Condition	Absent
Absorption (%)	--

The results indicated that the concrete was generally sound and unfractured and there was only slight paste deterioration immediately beneath the bituminous wearing surface. No evidence of a concrete wearing surface remained. Membrane waterproofing was present beneath the wearing course in two of the cores. The concrete from bridge 2 was also well compacted and not air entrained. Five core segments sufficiently long to test in compression yielded a mean strength of 5160 psi. Sonic pulse velocity measured through four of these core segments was in the 10 400- to 15 000-ft/s range. All eight cores were tested for absorption and yielded a mean value of 4.1 percent, which is greater than only 12.5 percent of values measured in cores from other New York bridge decks.

Comparison of Bridges 1 and 2

Both experimental bridges were built in 1931 from the same basic design, with well-compacted, non-air-entrained concrete of comparable strength. Yet their relative condition is markedly different. Although it was beyond the scope of this investigation to determine the causes for this difference, several contributing factors are apparent.

The absorption of bridge 1 concrete appears to be higher than that in bridge 2, possibly by as much as 1.5 percent when measured after 48 years of service. This suggests a higher initial water-cement ratio in the bridge 1 concrete that could have resulted in a higher permeability to both water and dissolved chlorides. Failure of concrete in the structural deck is judged to have resulted from freezing of water in the pores of saturated cement paste, aggravated by the presence of chlorides in solution. Failure of concrete in the lower portion of the T-beam stems and fascia girders is judged to have resulted from the same causes plus corrosion of steel reinforcement near the concrete surface. Both mechanisms are facilitated by increased concrete permeability. Differences in deterioration of concrete in other bridge decks in New York have been shown to be measurably related to differences in concrete absorption within the range encountered here (7).

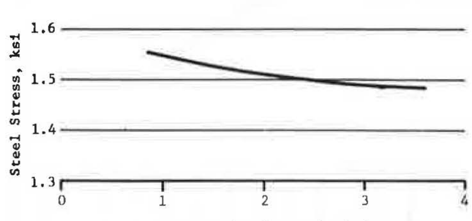
A second factor that may have contributed to the superior performance of bridge 2 is the possible presence of a protective waterproofing membrane between the structural concrete deck and the wearing surface at some time during the bridge's life. A membrane of the bituminous-epoxy type was found in two of the eight cores removed from this bridge. No evidence of a coating was found on the cores from bridge 1. Though such membranes have generally been found to be imperfect barriers to the passage of water and dissolved chlorides (8), they certainly inhibit such passage significantly, even in the least-effective applications.

THEORETICAL CONSIDERATIONS

Despite the emphasis on concrete strength in design calculations and construction inspection, this property has only a modest influence on structural capacity. In working stress design, which was in use in the era when the concrete bridges that are now causing load-rating difficulty were constructed, variations in concrete strength are reflected in the concrete modulus of elasticity and allowable stress. For sections of constant dimension under equal magnitude of bending moment, loss of concrete strength (reduction in modulus) results in an increase in the neutral axis depth, since a greater concrete area is needed to balance the steel. This increase is reflected in a decrease in the moment arm of internal forces and, consequently, an increase in steel stresses (see Figure 6). Because modulus varies as the square root of concrete strength, the change in steel stress is small over the probable range of concrete strengths. In contrast, concrete stress decreases under these conditions, since the decrease in internal moment arm is more than compensated by the increase in compression area. This decrease is also small, however.

Despite these modest effects on stress levels, the consequences of reduced concrete strength can be significant from the point of view of working stress. Allowable stress levels are a constant percentage of concrete strength (2), and thus, in certain cases, theoretical member capacity will vary directly with concrete strength. These cases occur when the amount of reinforcement is large and limiting stress levels are attained first in the concrete. Although each structure should be checked on an individual basis, this compression control situation is unusual and reductions in concrete strength will be of no consequence, except in the case of very weak concrete.

Figure 6. Variation in steel stress with concrete compressive strength.



If more recent methods of strength design are used, concrete strength also has little influence on capacity as long as this strength is above some limiting value. This limit is determined by calculating the concrete strength at which the failure mode changes from yielding of the reinforcement to compression failure. The latter is unacceptable because of the lack of visible signs of impending collapse. A specific section can be easily analyzed to determine this limit.

Figure 7. Variation in steel stress with slab thickness.

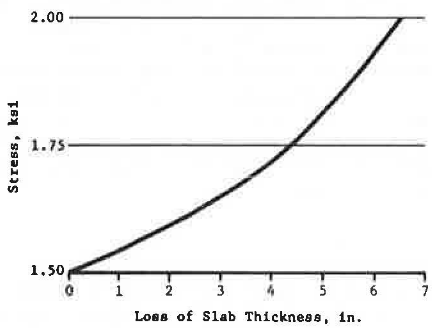


Figure 8. Effect of support skew on transverse influence line.

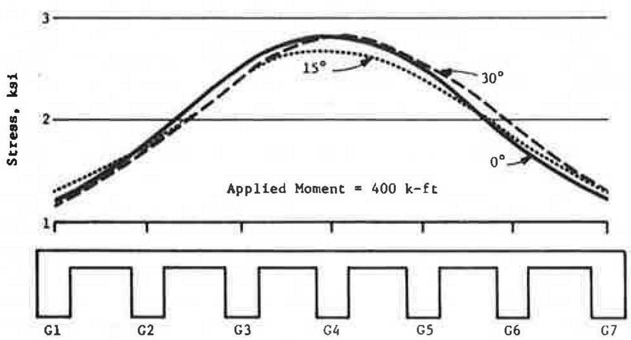
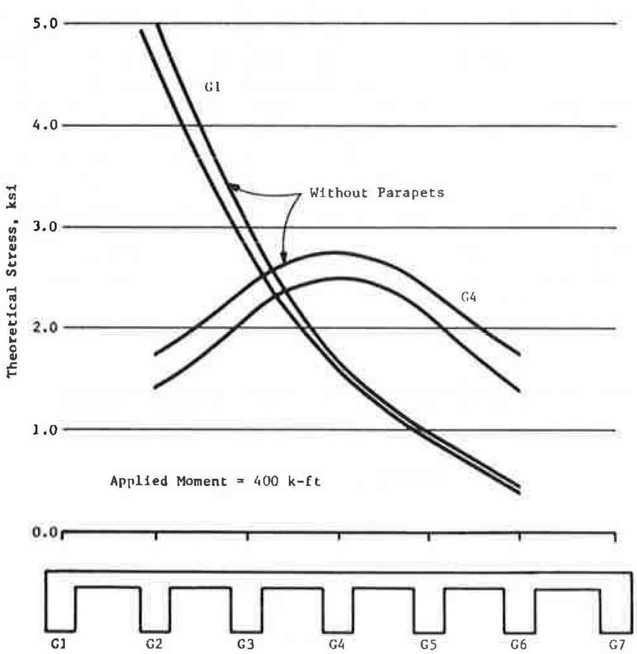


Figure 9. Effect of parapets on transverse influence line.



The consequences of loss of compression area can be evaluated by considering a specific T-beam section representative of the bridges tested. The section is subjected to a constant moment, and the variation in steel stress with decrease in slab thickness is evaluated. It can be seen in Figure 7 that steel stresses increase at an increasing rate. Although the variation in stress over the range of thicknesses shown is great, it should be realized that losses greater than about 3 in would weaken the ability of the slab to perform its primary function of resisting wheel loads.

From these two examples, it should be apparent that the consequences of loss of concrete strength or compression concrete area are small at working stress levels. Because of this result, observations of concrete deterioration in a conventional inspection report are of limited use to the structural engineer in making a rational estimate of safe load-carrying capacity.

FIELD TEST

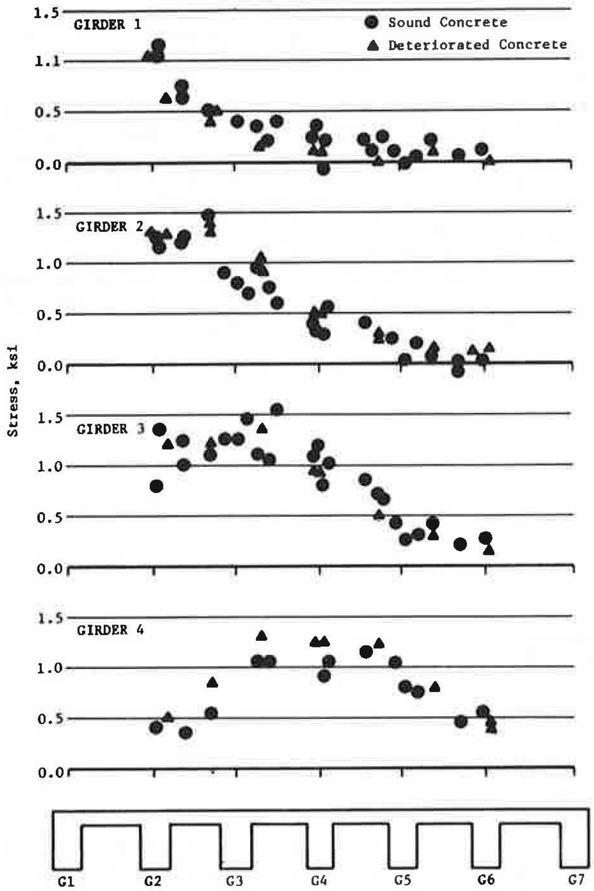
Despite the analytical findings regarding the influence of concrete deterioration on structure response, it was still felt that there should be a measurable difference between the behavior of sound and deteriorated structures. The two structures were instrumented with electrical-resistance strain gages bonded to the tension steel reinforcement at midspan. The test load was a dump truck weighing about 40 000 lb, which, when positioned at bridge midspan, produced a maximum bending moment close to the current HS-20 design load. The truck was positioned at several transverse locations across the widths of the structures, and strain in the reinforcement was measured. With these data, a transverse influence line was produced for each girder that gave the variation in reinforcement stress with transverse position.

Because of structural differences in the bridges, direct comparison of measured values is not possible. However, differences in response due to span length and support skew angle can be eliminated by normalizing the result to a constant level of midspan bending moment. Although support skew causes a marked change in the bending moment resisted by individual girders, there is an accompanying change in the total bending moment. Because of these changes, the variation in transverse distribution of bending moment is small for skew angles less than 30° (see Figure 8).

An analytical adjustment has also been made for the different stem depths. Conventional theory for reinforced concrete at working stress levels predicts that the stress induced in the reinforcing steel is proportional to the resisted bending moment. This factor of proportionality--the reciprocal of the section modulus--can be calculated from properties of the section. With knowledge of section modulus, measured strains can be adjusted to a common basis. The justification for this adjustment is the assumption that, although the effective section is poorly defined, if both structures were in new condition the only difference in the relation between moment and steel stress would be caused by the difference in stem depth.

An analytical adjustment for the effect of the parapets on the measured values cannot be made. Figure 9 shows the effect of the increased fascia stiffness that results from the parapets. Because of this increased stiffness, a greater proportion of total moment is carried by the fascia, which causes a reduction in stress in the interior girders. Because the section modulus of the fascia is increased, stress levels at this location are also

Figure 10. Experimentally determined influence lines.



reduced despite the larger moment. This effect cannot be removed from the data because the stiffness of the heavily deteriorated parapets cannot be reliably estimated.

The adjusted data are plotted in Figure 10 as influence lines showing the variation in stress in a particular girder as a function of load position. Data from symmetrical girders have been superimposed (i.e., the plot for girder 1 contains data from girders 1 and 7) since the raw data showed no evidence of unsymmetrical behavior. Except for the centerline member (girder 4), there are no differences in either the magnitude or trend of the influence lines for the two bridges. For girder 4, the response of the deteriorated structure (bridge 1) is slightly greater for loads near the centerline. Since this is the structure with parapets, and since analysis shows that stress levels would be decreased by the action of these elements, it is concluded that the parapets are inactive. In addition, the higher stresses imply weaker concrete or a reduced section resisting load.

The significance of the observed difference in response should be assessed in terms of the reliability of electrical strain gage measurements and the consequences to the load capacity of the structure. Data for each of the four girders scatter substantially. This scatter is the result of the joint variability of the load placement and the strain gages (and associated data-acquisition equipment). In addition, variation due to a lack of complete structural symmetry is a contributing factor. In view of the variability shown here, and experience with this type of measurement on other structures (9), claiming an accuracy better than ±200 psi would be unrealistic.

The maximum stress induced in girder 4 is 1300 psi in the deteriorated structure and 1150 psi in the sound structure. These stresses are the result of a 400-kip•ft moment resisted by the cross section. Based on these values and accounting for the dead-load stresses, these bridges could support 5.6 and 6.2 HS-20 trucks, respectively, with 30 percent impact at the steel working stress of 18 000 psi. Less conservatively, if one considers the effects of trucks in two lanes (the design load), the most heavily loaded girder is girder 2. For this design loading condition, the structures can support 3.0 and 3.2 times the design load, respectively. Thus, based on the evidence from these tests, neither structure is in any danger of flexural failure under highway loads.

OTHER STRENGTH CONSIDERATIONS

Clearly, the testing and analyses performed for these structures do not comprehensively evaluate their strength. Other forms of failure, such as fatigue, loss of bond between reinforcement and concrete, and punching shear in the slab, have not been evaluated. In general, these modes give no signs of impending failure, and testing short of destruction reveals no information on capacity. Analysis also falls short, particularly when it is necessary to factor in the existing condition of the concrete.

CONCLUSIONS

Despite an inspection report indicating that bridge 1 has severe deterioration, some testing that confirmed the presence of low-strength concrete, and rating by accepted procedures that suggested only minimal adequacy to tolerate present traffic, a comparison of physical test results from this and a geometrically similar but sound structure revealed only minor differences in flexural behavior. Estimates of flexural capacity, based on the field test results, show that both structures can support at least three times their design load without exceeding allowable stress in the steel reinforcement.

The results presented indicate that structural inspection and rating procedures used in New York and believed to be typical of national practice are deficient for evaluating deteriorated structural concrete elements. Apparently, bridge inspectors are not identifying those aspects of structural condition, which have a significant impact on safe load-carrying capacity. Although it is not an objective of this paper to identify appropriate inspection items, it is suggested that such things as length of exposed rebar (as an indication of bond loss) and loss of rebar cross section may be of greater significance to load-rating engineers than apparent overall quality of the concrete. Additional work is needed to identify types of bridge conditions that are detrimental to capacity and to develop procedures for converting these observed conditions into accurate load ratings that do not penalize the aesthetically deficient but structurally safe bridge.

ACKNOWLEDGMENT

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Abridgment

Accuracy of the Chace Air Indicator

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Results of a study undertaken to quantify and improve the relation between air contents in concrete determined by using the Chace air indicator (CAI) and those determined by using the pressure method are reported. The study revealed very poor agreement between air contents determined by the two methods. The pressure method gave values typically 30 percent higher than anticipated based on the CAI readings. The poor agreement was found to involve relations between the volume of the stems, the volume of the bowls, and the mortar correction factors supplied by the manufacturers of the CAI. Consequently, it was recommended that AASHTO T199-72 be modified to account for these relations so that the CAI can be used to obtain a reasonably accurate indication of the air content of fresh concrete.

Inspection personnel like the Chace air indicator (CAI) (AASHTO T199-72) because of the relative ease with which it can be used. Unfortunately, very poor agreement has been noted between the air contents determined by the CAI and those determined by the pressure method (ASTM C231-75 and AASHTO T152-76). As shown in Figure 1 (1), concrete accepted with the CAI and noted as having an air content of 8 percent, which would be acceptable, could actually have an air content of 12 percent or more, which could cause the concrete to fail the strength test. Frequently, when concrete cylinders have failed the 28-day strength test, subsequent petrographic examinations of the hardened concrete have revealed that the air content was much too high.

The study reported here, which comprised the preparation and testing of 99 batches of pavement and bridge-deck concretes (2, p. 145), was conducted to quantify and improve the relation between air contents determined by the CAI and those determined by the pressure method. The air content of each batch was determined once by using the pressure method, twice by using the CAI to measure the air content of each of two mortar samples obtained by passing a portion of the concrete through a no. 10 sieve (screened samples), and twice by using the CAI to measure the air content of each of two samples obtained by removing mortar from the concrete with a putty knife (unscreened samples).

RESULTS

Figure 1 shows the plots of the average of the CAI determinations on the two unscreened samples as a function of the air content determined by the pressure method, after the data for the CAI have been corrected for the mortar content of the concrete based on the manufacturer's recommended mortar correction factors (MCFs). A relation similar to that shown in Figure 1 was obtained by plotting the average of the CAI determinations on the two screened samples as a function of the air content determined by the pressure method. The data obtained for the screened mortar samples were slightly more variable than those for the unscreened samples: The standard deviation was 0.81 percent as compared with 0.71 percent (1).

Chace Factor

To determine why the CAI was indicating air contents that were much too low, measurements were made of the volumes of the bowls and stems of 36 randomly selected indicators from three manufacturers. The important consideration was the Chace factor, which is defined here as the volume of one graduation on the stem, which represents 1 percent air, expressed as a percentage of the volume of the bowl, which contains the sample of mortar. The measurements revealed that, for CAIs supplied by manufacturers H and C, the average Chace factor was 2.30 and the uniformity was good, exhibiting standard deviations of 0.05 and 0.03, respectively. The CAIs from manufacturer L had an average Chace factor of 1.87, but the variation among the instruments was broad: The standard deviation was 0.46. In fact, one CAI from manufacturer L had a Chace factor of 1.43 (the inside diameter of the stem was relatively small and would produce high stem readings) and another had a Chace factor of 2.51 (the inside diameter was relatively large and would produce low readings). For