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Flexural fatigue strength of hybrid layered concrete

by

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INTRODUCTION

Fatigue of Concrete

Fatigue is the term associated with the failure of a material due to repeated applications of loads each smaller than the ultimate static strength of the material. This process of progressive, permanent structural change resulting in cracks or complete failure occurs in most engineering materials. Considerable time and effort has gone into the study of the fatigue behavior of metals. The first structural components to be recognized as exhibiting fatigue behavior included mine-hoist chains, railway axles and steam engine parts. Thus, the first fatigue tests were performed on materials used in these components known to fail under repeated applications of a load less than that required for failure under a single application.

Although the design of most structural concrete members utilizes data obtained almost exclusively from static tests, many concrete elements, plain and reinforced, are subjected to repetitive loading. Among the more common such elements are highway pavements and bridge decks, and airport runways and taxiways.

Research on the fatigue behavior of concrete has been conducted since the beginning of this century. Many of these early studies were of limited scope, investigating the influence of only one or two variables. In the last quarter century, more extensive studies have been conducted dealing with the relative effect of coarse aggregates, fine aggregates, air content, water-cement ratio, etc., as well as the effect of various types of loading conditions. In the study of fatigue behavior, specimens are generally subjected to a sinusoidal wave form of loading. To evaluate the effect of one variable, specimens are subjected to cyclic loading at different stress levels. The result of these tests are usually presented in the form of an S-N diagram. The S-N diagram graphically represents stress (S) on the vertical axis versus the number of cycles to failure (N) on the horizontal axis. If for a certain material, this curve becomes asymptotic to a horizontal line, the stress at which this occurs is called the endurance or fatigue limit. The S-N diagram for most metals will have this characteristic which indicates that if the metal is subjected to repetitive stresses that do not exceed this level, no fatigue failure will occur.

As was previously stated, fatigue studies dealing with concrete began at the start of this century. Among those early studies was one by J. L. Van Ornum in 1903 in which cement cubes were tested in compression under repeated loads. The study indicated a fatigue limit of approximately 55 percent of the strength of the cement. In a later study involving concrete prisms tested in compression and reinforced concrete beams tested in flexure, Van Ornum determined a fatigue limit of 50 percent of the strength of the concrete under progressive loading (27).

The next significant investigations involving flexural fatigue of concrete were carried out by the Illinois Division of Highways and Purdue University at about the same time. The tests by the Illinois Division of Highways as reported by Clemmer (13) and Older (31) indicated that an endurance limit for concrete did exist. An apparatus was designed in which seven beams could be loaded simultaneously at a rate of 40 applications of load per minute. The beams tested were 6 inches square by 36 inches in length. The following conclusions were drawn from these tests:

- Concrete beams will fail under repeated applications of a load corresponding to a stress smaller than that required to cause failure under a single application of load.
- The endurance limit of the concrete specimens was between 51 and 54 percent of the modulus of rupture strength.
- Loads which produce stress less than the endurance limit do not cause failures but actually increase the strength of the specimen if the load is near this critical percentage of modulus of rupture.
- Stresses below the endurance limit do not cause permanent deformation.
- For stresses above the endurance limit, the number of applications of load required to produce failure decreases with increasing percentage of stress.

The studies conducted at Purdue University by Hatt and Crepps provided for a complete reversal of stress in flexural fatigue tests (20, 27). Various mixes were used in the fabrication of the 4 in. x 4 in. x 30 in. beams.

Included in the conclusions were the following: (30)

- For 28-day specimens, an endurance limit could not be established between 40 and 60 percent modulus of rupture.
- Rest periods have only a temporary effect on the fatigue strength.

- Conditioning a specimen by stressing it below the fatigue limit raises the endurance limit.

In a later investigation involving the effect of speed of testing on fatigue strength of concrete, Kesler tested 100 concrete beams of dimensions 6 in. x 6 in. x 64 in. (23). Flexural tests were conducted for two concrete strengths, 3600 psi and 4600 psi, at speeds of 70, 230 and 440 cpm. Tests were carried out to a maximum of ten million cycles of stress; no specimens failed when stressed less than 55 percent of modulus of rupture. However, there was also no indication that a fatigue limit exists. Kesler concluded that for the speeds at which the tests were conducted, the fatigue strength, expressed as a percentage of the static strength was the same for both concretes indicating the independence of fatigue strength and conpressive strengths. He also concluded that the speed of testing in the range investigated has little or no effect on the fatigue life and strength of concrete. Fatigue life is the number of cycles of a certain stress level which can be endured before failure occurs, while fatigue strength of concrete is the stress that it can withstand repeatedly for a given number of cycles.

In a later investigation carried out by Murdock and Kesler (28), the range of the repeated loads was varied throughout the study. A constant ratio between the minimum and maximum applied stresses was maintained for each series. No fatigue limit was found for plain concrete, at least through ten million cycles of stress. It was also concluded that the repeated loads which plain concrete can withstand for a finite number of repetitions without failure is a critical percentage of the static ultimate flexural strength and is a function of the range of stress to which the concrete is subjected.

In a later study, Hilsdorf and Kesler (22) investigated the behavior of concrete under repeated loads when the minimum and maximum loads vary during the tests or when the load cycles are interrupted by rest periods. Phase 1 of the study dealt with the rest periods; specimens were subjected to a sequence of loading periods and rest periods. The loading periods consisted of 4500 load cycles and the rest periods were of 1, 5, 10, 20, or 27 minute duration during which the minimum load was maintained.

It was concluded that the fatigue strength increases with increasing length of rest periods up to 5 minutes. If the length of the rest periods was increased to 27 minutes, no further increase in fatigue strength occurred.

The effect of variation in load was studied in Phase 2 of the above-mentioned investigation, with the maximum load being altered during the test. Conclusions drawn from Phase 2 include the following:

- When the maximum stress level is changed once, the fatigue life of a specimen is larger if the highest stress level is applied first compared to the fatigue life of a specimen in which the lower stress level was applied first.
- When the maximum stress level is varied continuously, the fatigue life decreases with increasing magnitude of the higher stress level and with increasing number of cycles under the higher stress.
- The Miner hypothesis used for the design of concrete structures gives unsafe values of the fatigue strength of concrete at high

stresses but is too conservative at low stresses.

- A procedure is proposed to adjust the Miner hypothesis so it can be safely used in the design of concrete.

Ballinger conducted a similar study in 1972 (6) and made the following conclusions:

- The general fatigue behavior of plain concrete can be represented by a straight line on a semilog S-N diagram.
- The damage occurring in the low cycle region, up to 70 cycles is not linear; it is apparently affected by other fracture phenomenon besides fatigue.
- Concrete does not appear to have a fatigue limit.
- Above a certain level of stress, 70 percent of static ultimate strength, concrete is very sensitive to fatigue.
- The accuracy of the S-N line is dependent on the accuracy of the predicted static strength of the specimen.
- The Miner hypothesis appears to reasonably represent cumulative damage effects from variations in fatigue loading.
- The order in which variations of maximum stress are applied appears to have no effect on the fatigue life of concrete.

These last two conclusions differ from conclusions drawn by Hilsdorf and Kesler earlier.

Concrete does not have a fatigue limit but the fatigue strength at 10 million cycles of load is approximately 55 percent of the static ultimate strength (2, 6, 7). Since concrete does not exhibit a fatigue limit, it is important that the fatigue strength be quoted for a specific number of cycles. The fatique strength of concrete is influenced by rate of loading, range of loading, eccentricity of loading, load history, material properties and environmental conditions(2, 22).

Fatigue of concrete is associated with the formation of microscopic cracks at the cement matrix - aggregate interface and in the cement matrix itself (7, 21, 37). Bennett (7) found that the length of cracks visible on the surface of a specimen subjected to 100,000 cycles of stress equal to 75 percent of the static strength was 35 percent greater than the length measured after a single application of a load 95 percent of the static strength.

The importance of the fatigue behavior of concrete is recognized in the design of concrete pavements. The most widely used fatigue curves for plain concrete pavement design are those of the Portland Cement Association (36). Essentially all modern rigid design methods consider the anticipated number of axle loads as well as the weights of these heavy axle loads which will be applied during the pavement design life. The American Association of State Highway and Transportation Officials, AASHTO, Interim Design Procedure (1) is used by most highway departments in this country and the Road Note 29 design procedure (4) is used by U.K.

Fatigue behavior of highway bridge decks can be related to that of highway pavements even though decks are reinforced while pavements typically are not. As reported by Kesler (23), reinforced concrete pavements exhibit approximately the same type of behavior in flexure as if they were plain concrete.

Highway concrete slabs are subjected to many repetitions of traffic loads during their service lives which hastens the premature cracking and spalling on bridge decks. This cracking and spalling is believed to be caused by deicing salt which when dissolved in water penetrates the pores and cracks in the concrete to the level of the top layer of reinforcing steel in the bridge deck. The salt solution hastens corrosion of the reinforcing bars which causes an increase in volume. Because of this increase in volume, high stresses develop in the concrete; cracks widen and finally develop into spalls (9, 11). This whole process is accelerated by the continuous loading from heavy traffic and limited concrete cover over the reinforcing steel.

When spalling, scaling and cracking first appear in bridge decks, maintenance repairs range from cold mix asphalt through conventional concrete and epoxy mortars to high-early strength and/or expanding concretes (9). Wearing surfaces are frequently added over these repairs, but generally deterioration of the underlying bridge deck continues. Conventional portland cement is sometimes used in patching spalls on deteriorated bridge decks. Preparation of the area to be patched is important and even if this is accomplished, the patch may fail from continued erosion of rebars and additional freeze thaw cycles. Other types of patches include high-early strength cements, expanding concretes and polymer concretes. Various epoxy-based bonding agents have also been used. Epoxy injection has been used experimentally on bridge decks where delaminations exist but break-up and spalling is not advanced. This process does not arrest the corrosion process but reduces the susceptibility of the bridge deck to deterioration from freeze-thaw and traffic loads.

Asphalt overlays have been used to provide a smooth riding surface but are permeable and trap water containing chloride on the surface of the deck. Concrete overlays provide a greater concrete cover over the rebars and impermeability can be increased by using a denser concrete or a sealing agent in the mix.

In 1963, Gillette (18) did a study on the bonded overlays which had been used in bridge deck repairs since 1954. The major emphasis of this investigation was the determination of the bond strength of various portland cement concrete overlays. Gillette found that overlays will perform adequately if proper surface preparation and construction procedures are followed.

In a study by Sinno and Furr (32), direct shear tests were performed on bonded concrete overlays to evaluate the parameters involved in bonding freshly overlaid concrete to deteriorated concrete surfaces. Cement grout and epoxy were found to be satisfactory in this capacity. Laboratory flexural fatigue tests were also performed; the thin bonded overlays increased the flexural rigidity and the load carrying capacity of the tested beams even though there was some decrease caused by the added dead weight of the overlay.

Furr and Ingram (17), in a later investigation of bonded concrete overlays made the following conclusions:

- If the base material is properly prepared, portland cement concrete overlays bonded with either portland cement grout or epoxy resin are capable of developing higher bond strengths than normally required in service.
- One- or two-inch thick concrete overlays, which are properly bonded will undergo at least 2 million load cycles without bond failure.
- Thin concrete overlays can be placed and cured on a vibrating base similar to bridge decks under controlled traffic loads.
- Epoxy and portland cement grout bonded concrete overlays will undergo ASTM C 290 durability test without failure.
- Air entrainment provides good resistance to freeze-thaw scaling.

The Iowa low-slump and latex modified concretes are the leading contenders for the overlay type of deck repair. Both are fairly common today and result in a relatively impervious concrete - one due to its high density and the other due to a latex additive.

Iowa Method Low-Slump Concrete

Iowa, like other states utilizing deicing salts, became concerned over the increasing number of bridge decks with unsound concrete surfaces. This subject was considered at Iowa Department of Transportation, DOT, District Engineers Meetings in the fall of 1962. Personnel from the Maintenance, Materials and Bridge Design Departments then began a survey of the bridge deck deterioration throughout the state to determine the extent of damage and to come up with a recommended method of repair (8).

In 1964, some partial depth patching was done using maintenance mix concrete. A contract was awarded in August, 1964 for an experimental research project to resurface a bridge on Iowa 196 in Sac County. This project consisted of removing the unsound concrete and placing a one-inch thick portland cement concrete overlay on the entire bridge deck. The Iowa DOT followed recommendations and procedures of PCA bulletin No. D44, "Repair of Concrete Pavement," by Earl J. Felt (8, 11). Many bridge decks were repaired by using a portland cement concrete overlay until 1966 when the Maintenance Department decided to use low-slump concrete for the overlay.

This resurfacing process includes removing all unsound concrete, scarifying the surface and sandblasting all exposed concrete and reinforcing steel before addition of the overlay (11). The major areas of unsound concrete are determined by dragging sounding equipment with a hammerhead device over the concrete surface. Areas containing badly corroded steel are isolated using a corrosion detection device. In the scarification process, 1/4 inch or more of the wearing surface is removed to rid the surface of vehicle drippings, linseed oil and possibly other contaminants which could weaken the bond. Rust and potentially weak concrete are removed in the sandblasting process.

A minimum of two inches of concrete is required for cover over the reinforcing steel. The "Iowa method" concrete used for the overlay is a high-cement-content, high-density, low-slump, air-entrained mix. The ingredients of this low-slump or dense concrete are used in the following proportions by weight: water: cement: coarse aggregate: fine aggregate =

1: 3: 5.2: 5.2; it also contains 6 percent entrained air and a water-reducing admixture to improve workability (8, 9, 11, 12, 30). This mix has a low water-cement ratio, 0.328, low slump requirements, 3/4 inch ± 1/4 inch, and high density, 98 percent of rodded unit weight. A compressive strength as high as 9000 psi may be obtained from dense concrete which is properly mixed, consolidated and cured (11, 12). The water reducing admixture contained in dense concrete is thought to be absorbed onto the cement particles causing them to become negatively charged and mutually repulsive as a result of the anionic nature of the water reducer (35). Thus the attraction between cement particles is reduced, greater mobility of the particles is possible and water is available to lubricate the mix and provide a wetter consistency. As a result, a low water-cement ratio can be obtained for a given workability or consistency.

A bonding grout consisting of equal parts by weight of portland cement and sand mixed with enough water to form a stiff slurry, is used between the old and the new concrete. The deck surface is thoroughly cleaned with an air blast and the grout is applied immediately in front of the paver to prevent the grout from becoming dry before the new concrete is placed. This low-slump concrete can not be placed when the ambient temperature is below 40 deg. F and a longer curing period may be required when the temperature is below 55 deg. F (33).

Dense concrete satisfactorily prevents further penetration from later applications of deicer salts. Also, it allows the interface between the overlay and the base concrete to breathe, preventing a buildup of moisture which could initiate further corrosion of the steel.

Dense concrete has the following desirable properties: (35)

- reduction in permeability
- higher compressive and flexural strengths at all ages
- increased abrasion resistance
- decreased shrinkage upon set
- decreased long-term shrinkage
- increased durability (8).

"Iowa method" low-slump or dense concrete has been used successfully in Iowa for more than ten years. Some minor localized failures have occurred - such as loss of bond in a spot where an air compressor leaked oil and the saturated area was not removed (11, 12). In 1973, the Department of Transportation initiated a program of analyzing the concrete on several resurfaced bridge decks as reported by Bergren and Brown (8). Fifteen bridges which had been resurfaced with dense concrete were selected for chloride penetration sampling. Only bridges which had been repaired to a depth greater than one-half inch into the old deck were used for the sampling. This was to insure that a meaningful salt penetration profile could be plotted. The resurfacing of these bridges took place between 1965 and 1972, and none of them exhibited deterioration of the resurfacing concrete. In spite of high chloride levels in the base concrete, corrosion of rebars was arrested and a good bond achieved. No deterioration was found and electrical measurements indicated that no active corrosion was taking place (8, 12). The majority of the more than 400 bridge decks resurfaced using "Iowa method" low-slump concrete have shown no disintegration or other problems.

Latex Modified Concrete

Latex modified concretes are the oldest type of concrete composites containing polymer and currently represent the majority of commercial applications of polymer modified concrete in the United States (3).

A latex contains very small spherical particles of polymers held in suspension by surface-active agents. The polymer latex is usually formed directly by emulsion polymerization of the monomer and usually contains about 50 percent solids by weight (3).

The first work in the area of modifying concrete with latices began in the thirties and involved the use of natural rubber latices to modify cement mortars. Synthetic latices were first used in the forties and were the result of development of polyvinylacetate latices (15). Mortar modified with these materials had good bond strength to concrete and improved compressive and tensile properties but was water sensitive, limiting its use to interior applications.

In the fifties, the use of styrene-butadiene latices in portland cement systems was introduced. These concretes had the same characteristics as those modified with polyvinylacetate latices except for the water sensitivity. Thus these concretes could be used in exterior applications.

Acrylic latices were brought into the industry in the late fifties and early sixties and produced mortars with similar characteristics to those of styrene-butadiene latex modified mortars. Also introduced in the early sixties were the saran latices which exhibited the highest strength properties of all the latices mentioned above.

These latex modified mortars were used because of two significant characteristics: their ability to bond well to a substrate, usually concrete, and their excellent durability. These qualities were important in the first application of latex mortars as floor underlayments over concrete. The mortar was used to level a concrete floor before tile or carpeting was laid. The use of latex modified mortars expanded into the areas of upgrading portland cement stucco, resurfacing deteriorated bridge decks, brick masonry applications, concrete block construction, sandwich panels and thin shell applications (15).

As was previously stated, latex modified mortars have been used in bridge deck resurfacing. Some of these overlays of 1/2 in. - 3/4 in. thickness have been in place and exposed to deicing salts for as long as 12 to 14 years and show no evidence of scaling due to freeze-thawing or destruction caused by rebar corrosion (25). This may be explained by considering two basic factors of cement hydration: excess water in the concrete mortar, and microcracking in the mortar (25). Considering the first factor, for deck concrete to be properly placed in the forms and consolidated around the reinforcing steel, it contains twice the amount of water needed for hydration of the cement. This excess water bleeds to the surface and evaporates, leaving behind capillary tubes in the hardened mortar portion of the concrete. The second factor, microcracking, is known to occur in portland cement compositions. These cracks are formed during the setting and hardening of the mortar portion and are believed to be caused by internal stresses built up by the heat of the chemical reaction of cement and water. Shrinkage and volume changes are also causes of microcracking.

In latex modification of these same portland cement mixtures, 1/3 less water is required for proper placement since the polymer particles fluidize the mix. Thus, fewer capillary tubes are formed in the mortar. Microcracking still occurs but as the mortar cures, i.e., excess water departs and the cement gel hardens, the microcracks are plugged with hardened ploymer (3, 5, 15). One might expect that the capillary tubes are also plugged when water borne polymer is left behind as the water evaporates. Laboratory investigations have confirmed the high degree of impermeability provided by latex modification of portland cement systems (25). Latex modified concrete followed latex modified mortars with many of the same properties. The mix used for latex modified concrete varies but the basic ingredients are the same as for normal portland cement concrete with two exceptions; the addition of latex and the exclusion of an air entraining agent since latex entrains air. Latices are generally more effective in richer mixes.

The effects of latex modification on concrete are as follows: (3, 25)

- reduction in permeability
- higher tensile and flexural strength
- improved ductility of cement paste as reflected in the higher tensile and flexural strength and increased strain at failure
- increased durability - existing microcracks tend to be held together by microfibers of polymer
- improved workability
- improved resistance to acids and alkalis

- improved frost resistance
- improved wear resistance
- improved adhesive properties
- decreased shrinkage (with styrene-butadiene)
- improved compressive strength

Because of these improved qualities, latex modified concrete was introduced in the bridge deck resurfacing industry. Latex modified mortars had proved satisfactory in this capacity so it was reasonable to expect that a layer of latex modified concrete would also prove satisfactory.

Kentucky Department of Highways began using latex modified mortars in bridge deck repair in 1961 (34). They have since approved four latex modifiers for use in concrete utilized in bridge deck repairs. Of the 500 bridges in Kentucky which have been overlaid with latex, only about one percent have been classified as unsuccessful.

Iowa began using latex modified concrete in the repair of its deteriorated bridge decks in 1972, following the same procedures used for resurfacing decks utilizing dense concrete. The mix used by the Iowa DOT consists of 7 sacks cement, 17.5 gallons water, a dry weight ratio of cement: sand: coarse aggregate of 1: 2.5: 2, and 24.5 gallons of latex for one cubic yard (8, 33). Specifications used by the Iowa DOT require a water-cement ratio within the limits of 0.34 - 0.40, a slump of 3-5 inches, and a plastic air content of 3-6 percent (33).

For optimum properties, latex modified concrete must be moist cured for 1 to 3 days and then dry cured at ambient conditions (3, 5). Until excess moisture leaves the specimen, the polymer will not harden and adequately seal microcracks and micropores; thus extended dry cure enhances strength and durability properties of the concrete. The more massive the specimens, the longer the curing period required. Latex modified concrete is also weather sensitive and can be placed only when the ambient temperature is between 45 and 85 degrees F (9, 33). If the temperature is below 55 degrees F a longer curing period may be required. At temperatures above 85 degrees F, the latex modifier tends to form a plastic film on the surface, thus the concrete must be placed at temperatures below 85 degrees F.

In its evaluation of resurfaced bridge decks (8), the Iowa DOT studied one bridge deck which had been resurfaced using latex modified concrete. This deck had only been exposed to one winter of salt application, thus data was limited. No deterioration of the resurfacing concrete was found.

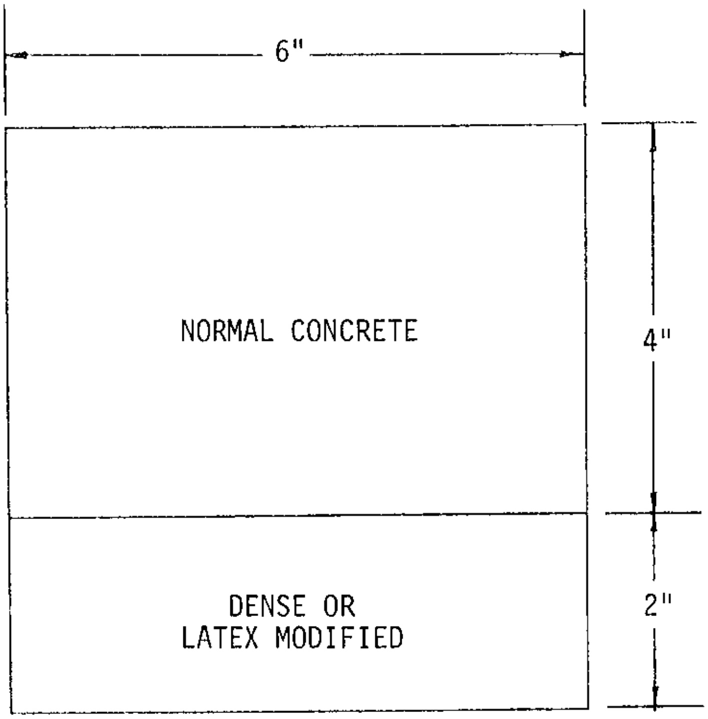
Bergren and Brown (8) also compared "Iowa method" dense concrete with latex modified concrete. They concluded that the physical properties of the two concretes are similar. Bridges resurfaced with latex modified concrete resulted in a smoother ride than those resurfaced with dense concrete. The Federal Highway Administration gave approval in early 1974 for the use of either latex modified concrete or "Iowa method" concrete in resurfacing bridge decks at the contractor's option. Currently the "Iowa method" concrete is used more often since it is less expensive than latex modified concrete.

Since these two concretes are used in the resurfacing of bridge decks and for a period of time, in the construction of new bridge decks, research which determines the effect of the addition of a layer of latex modified concrete or dense concrete on the fatigue strength of normal concrete would be of great value. To the author's knowledge, no research has been done on the fatigue strength of these concretes or on what effect they have on the fatigue life of the bridge deck. Thus, in this investigation, a determination of the fatigue strength of dense concrete and latex modified concrete both as a homogeneous single layer beam and as a component in a layered beam will be made.

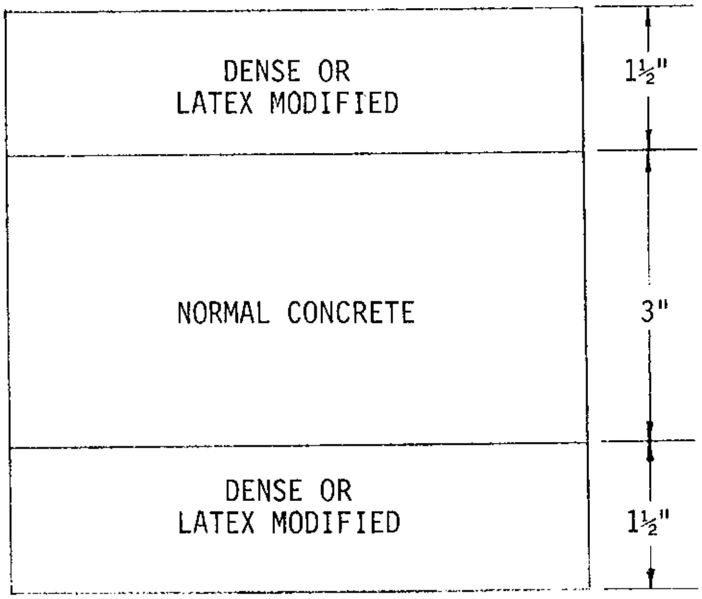
PURPOSE AND SCOPE

The major cause of bridge pavement deterioration is spalling due to penetration of deicing salt into the concrete. These deicing salts contain chloride which deteriorates the reinforcing steel after penetrating the concrete surface. Inadequate concrete cover over the reinforcing steel further complicates the problem. These bridge pavements periodically need to be resurfaced. In recent years many bridge decks have been resurfaced using either "Iowa method" low-slump concrete or latex modified concrete as previously described. Henceforth, "Iowa method" low-slump concrete will be referred to as simply dense concrete. Both of these concretes help prevent penetration of deicing salts and thereby retard surface deterioration (3, 8, 9, 11, 12, 15, 25). Studies have shown that both latex modified concrete and dense concrete have improved compressive and flexural strengths (3, 8, 11, 12, 15). Since the maximum stress occurs near the surface of a beam or slab under flexural loading, it is hypothesized that a superior concrete at the surface might increase the fatigue life of the specimen. Surface treatment has been known to improve the fatigue strength of metals, thus, one might hypothesize the same would be true for concrete. The purpose of this study is to determine how layers of either dense concrete or latex modified concrete affect the fatigue life of plain normal concrete. The term normal concrete will refer to concrete containing no additives other than an air entraining agent, and having a water-cement ratio of about .34, air content of 6% and slump of about 2 1/2 inches.

The scope of this work includes flexural fatigue testing of layered beams using both dense concrete and latex modified concrete with normal concrete. The normal concrete used with both the latex modified concretes and dense concrete was to have the same W/C ratio and air content. Two-layered and three-layered beams as shown in Figure 1 were tested along with control beams consisting of only normal concrete. The results of these tests were to be plotted on an S-N diagram to compare the curves of the normal concrete control beams with the curves of the layered beams utilizing dense concrete and latex modified concrete.



Two-layer beam



Three-layer beam

Figure 1. Cross-section of two-layer and three-layer beams

MATERIALS AND PROCEDURES

Testing Program

The objective of this investigation was to compare the fatigue strength of plain normal concrete with the fatigue strength of hybrid layered beams containing normal concrete and either dense concrete or latex modified concrete. To optimize the research effort in this direction, the test program was designed such that the only variables were the type of concrete, dense or latex modified, and the quantity of that concrete used in the beam. The normal concrete was to be the same for both the layered dense concrete beams and the layered latex modified concrete beams. However, after the first batch utilizing dense concrete and normal concrete was poured and compression tests run, it was decided that the compressive strengths of the two concretes did not differ by what was felt to be a significant amount. Therefore a different mix was designed and used as the normal concrete of the second series with latex modified concrete. Another difference between the two normal concretes was curing conditions. The series with dense concrete was water cured, whereas the series with latex modified concrete was air cured since latex modified concrete must be dry cured for optimum properties. All beams were tested at approximately the same age thereby essentially eliminating the age variable. Six in. by 6 in. by 36 in. beams were used for fatigue testing. The first 18 inches of the beam were used for the modulus of rupture test and the remaining unstressed portion was tested in fatigue; this procedure provided a companion modulus of rupture test for each individual fatigue test. Between the modulus of rupture test and the fatigue test, the beams which were wet cured were sealed in a plastic bag to maintain saturated moist conditions. Previous studies (29) have shown that the scatter of data is increased if the beams are allowed to air dry during testing. Differential strains generated by moisture gradients within the beam are believed to be the cause of this increase in scatter.

A percentage of the modulus of rupture stress was used to determine the stress levels for fatigue testing. This stress level was converted to an equivalent maximum load which was repeatedly applied to the beam until failure occurred. The minimum load applied to the beam was 100 pounds corresponding to a stress of less than 10 psi; thus no stress reversal took place. Fatigue tests on the series of beams utilizing dense concrete were made at four stress levels corresponding to 90, 82, 73, and 60% of the modulus of rupture. Fatigue tests on the series of beams utilizing latex modified concrete, including the control beams, were made at three stress levels corresponding to 90, 73, and 60% of the modulus of rupture since fewer beams of each of these series were poured.

In addition to the main fatigue test program three other investigations were carried out. These included the following subjects:

1. Modulus of rupture tests
2. Compressive strength tests
3. Modulus of elasticity determination

Materials

Four different concretes were used in this investigation, normal(D), dense, normal(L) and latex modified concrete. The normal(D) concrete is the normal concrete utilized in the series of beams containing dense concrete. Likewise the normal(L) concrete is the normal concrete utilized in the series of beams containing latex modified concrete. These normal concretes were purchased from ready-mix companies so many properties of the aggregates, cement and air entraining agent are unavailable. Properties of the coarse and fine aggregates used in these concretes are summarized in Table 1. All aggregates utilized came from Iowa DOT approved stockpiles. The gradation of fine and coarse aggregates used in dense concrete and latex modified concrete are given in Tables A-1 and A-2 of Appendix A. Type I Portland Cement used in these concretes was obtained from Marquette Cement Corporation in Des Moines, Iowa. Chemical and physical properties are shown in Table A-3 of Appendix A. The air entraining agent used in the dense concrete was Ad-Aire, a vinsol resin produced by the Carter Waters Company of Kansas City, Missouri. No air-entraining agent was used in the latex modified concrete.

The dense concrete mix was designed according to the Iowa Department of Transportation specifications for this type of concrete (33). The water-cement ratio was approximately 0.32. A water reducer was used to aid in the workability of the concrete. The water reducer utilized was Plastocrete 161 which is a polymer-type admixture manufactured by Sika Corporation, Lyndhurst, New Jersey. Further information about this water reducer could not be obtained.

Normal(D) concrete consisted of an Iowa DOT C-4 mix (33). In this mix a standard iron oxide mortar dye was used so the interface between the normal(D) concrete and the dense concrete could be investigated. A search for literature dealing with the effect of mortar dye on the strength of concrete was futile.

The latex modified concrete was designed according to Iowa DOT specifications for latex modified concrete (33). The water cement ratio was 0.35. The latex used was DOW Concrete Modifier A, a styrene-butadiene polymeric emulsion manufactured by DOW Chemical, USA, Midland, Michigan. Specifications of this latex are found in Appendix A.

Normal(L) concrete was also designed according to Iowa DOT specifications (33).

Batch quantities of these concretes are given in Table A-4 in Appendix A.

Preliminary Testing

To determine the optimum mixes, trial batches of the four concretes were mixed in the laboratory. These trial batches were used to determine the proportions of aggregates, cement, water and additives required for the proper water-cement ratio, slump and air content. After satisfactory trial batch mixtures were obtained, the quantities were increased for larger batches required for pouring the flexural specimens and control cylinders.

Mixing Procedures

Hybrid beams of normal concrete and dense concrete as well as hybrid beams of normal concrete and latex modified concrete were poured for the investigation.

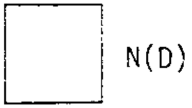
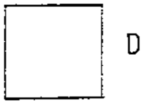
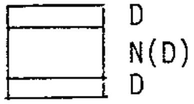
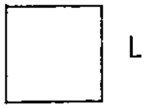
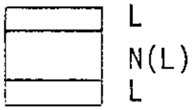
The four series of beams utilizing dense concrete were designated D-0, D-1, D-2 and D-3 representing respectively, the normal(D) concrete beams, the dense concrete beams, the two-layer beams of dense concrete and normal(D) concrete and the three-layer beams of dense concrete and normal(D) concrete. The series D-0, D-1, D-2 and D-3 were poured at the same time and contained 5 beams, 4 beams, 20 beams and 18 beams respectively. Also poured at this time were 6 cylinders of normal(D) concrete and 6 cylinders of dense concrete. As shown in Figure 1, the two-layer beams consisted of 2 in. of dense concrete and 4 in. of normal concrete. The three-layer beams had 1 1/2 in. of dense concrete on the top and bottom , with 3 in. of normal concrete in the middle. All hybrid beams were 6 in. x 6 in. x 36 in., the normal beams (D-0) and the dense beams (D-1), poured for modulus of rupture comparison, were 6 in. x 6 in. x 30 in. All cylinders utilized were 6 in. in diameter and 12 in. high.

Four series of beams utilizing latex modified concrete were poured at one time. For the purpose of identification, the following designations were assigned: normal(L) concrete beams, L-0, latex modified concrete beams, L-1, two-layer beams consisting of normal(L) concrete and latex modified concrete, L-2, and three-layer beams containing these two concretes, L-3. The series L-0, L-1, L-2, and L-3 contained 17 beams, 3 beams, 14 beams, and 14 beams respectively. Ten cylinders of normal (L) concrete and 6 cylinders of latex modified concrete were also poured. The beams were layered and sized the same as those utilizing dense concrete.

The previous information concerning beam configuration, etc., has been summarized in Table 2.

The dense concrete and the latex modified concrete were mixed in the laboratory using materials stored in the lab to insure uniformity of the mixes. Since a one cubic yard lab mixer could not be obtained, a ready-mix transit mixer was rented and brought into the laboratory for mixing the concrete. Prior to charging the materials into the mixer, it was inspected for residue concrete and excess water. Batch quantities were weighed, corrected for moisture content, and charged into the mixer. The normal concretes, normal(D) and normal(L), were obtained from ready-mix companies utilizing the mixes previously designed. Slump and plastic air content tests were performed and recorded following a mixing time of three minutes for dense concrete. These tests were performed on the latex modified concrete following a mixing period of four minutes and a resting period of four minutes. Figure 2 shows these tests being performed. Standard air meters of the pressure type were borrowed from the Iowa Department of Transportation and used for the air tests. The fresh concrete was transferred to the beam molds using wheelbarrows. Screeds were made to level the concrete at the proper depths as shown in Figure 3. The flexural specimens were vibrated in accordance with ASTM C 192 (14) with a small laboratory pencil type vibrator with a one inch head operating at 10,500 vibrations per minute. The bottom layer of the two-layer beams was vibrated followed by addition of the second layer which was also vibrated. The vibrator was then allowed to penetrate both layers six times per beam; this was considered adequate to achieve

Table 2. Characteristics of each series of beams

Series	Concrete used	Number of beams	Layer configuration
D-0	Normal(D)	5	
D-1	Dense	4	
D-2	Dense normal(D)	20	
D-3	Dense normal(D)	18	
L-0	Normal(L)	17	
L-1	Latex modified	3	
L-2	Latex modified normal(L)	14	
L-3	Latex modified normal(L)	14	

N(D) = Normal(D) concrete

D = Dense concrete

N(L) = Normal(L) concrete

L = Latex modified concrete

Dimensions as shown in Figure 1.

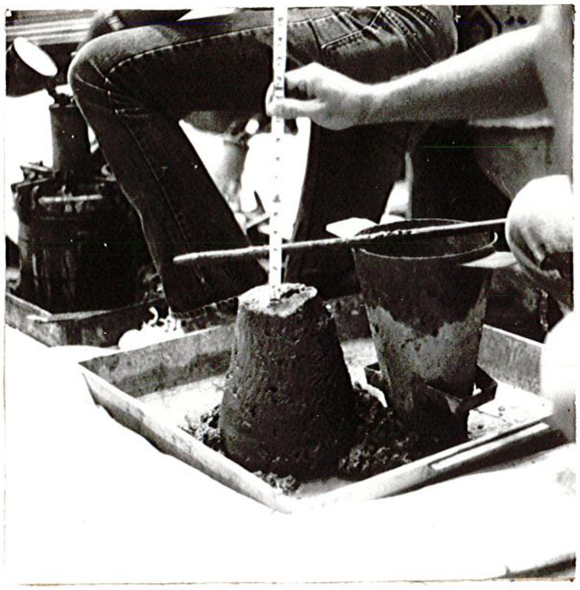


Figure 2. Slump and air tests

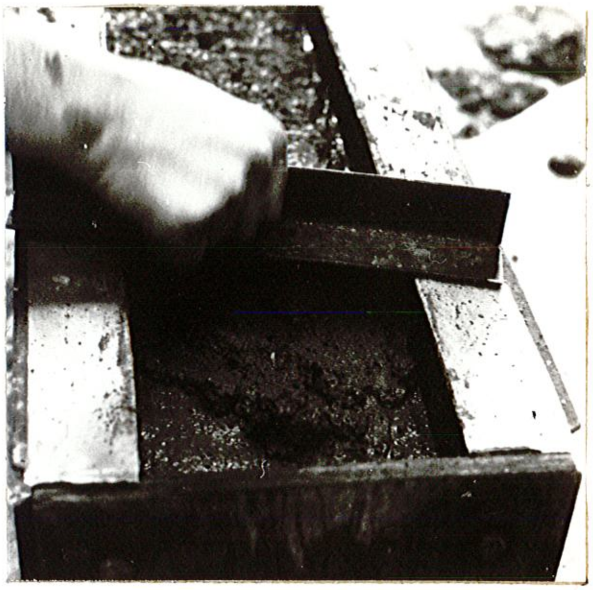


Figure 3. Screed used to level concrete

Figure 4. Finished beams and cylinders

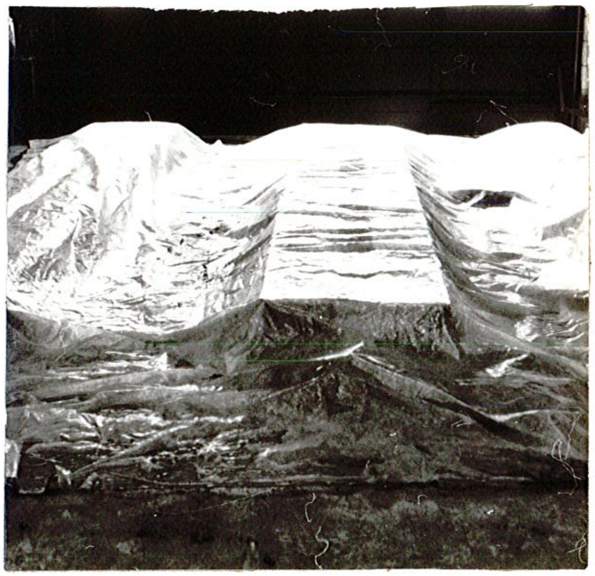


Figure 5. Beams covered with wet burlap and polyethylene sheet

a bond between the layers. The same procedure was followed for the bottom two layers of the three-layer beams and after placement of the third layer, the beams were vibrated through the top two layers six times per beam. All mixing procedures utilized were in accordance with ASTM C 192 (14). All cylinders were cast in 6 in. x 12 in. waxed cardboard cylinder molds with each of the four concretes used in the flexural specimens. Figure 4 shows the beam forms and cylinder molds filled with concrete.

The beams and cylinders were covered with wet burlap and polyethylene sheets to assure correct moisture conditions for proper curing (Figure 5). After 48 to 72 hours, the forms were stripped and the beams removed. The dense series of beams, (D-0, D-1, D-2, D-3), were then cured in water in large metal tanks until time of testing. Since latex modified concrete must be dry cured following removal of forms, the latex series of beams, (L-0, L-1, L-2, L-3), were air cured until time of testing.

Equipment

A 400^k Baldwin-Satec Universal Testing Machine was used for modulus of elasticity and compression tests following ASTM standards C 469 and C 38 (14). A Tinius Olsen concrete cylinder compressometer was used for determination of the modulus of elasticity. The modulus of rupture tests were performed according to ASTM C78-75 (14) in an American Beam Tester Company Model S6 Concrete Beam Tester using third point loading as shown in Figure 6. The overhanging portion of the test specimen caused an insignificant amount of stress of opposite sense at the critical test section and thus its effect was negligible. After the

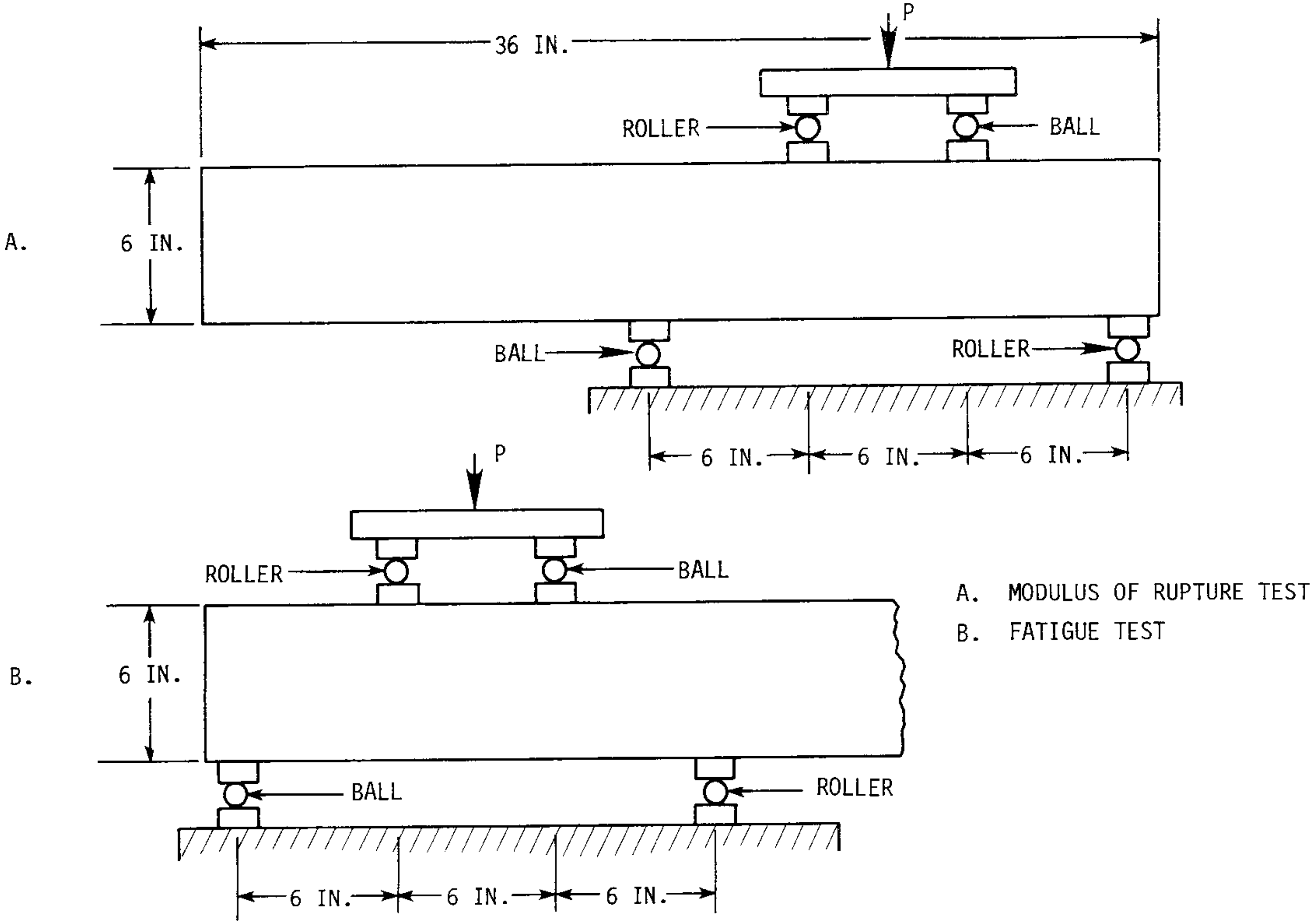
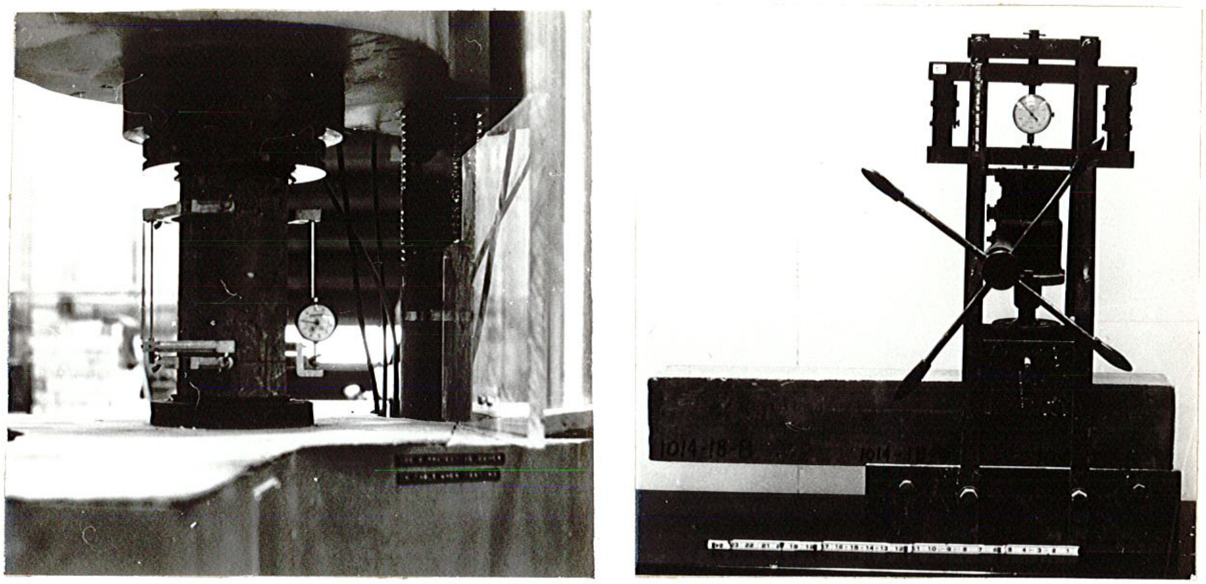


Figure 6. Schematic diagram of loading arrangements

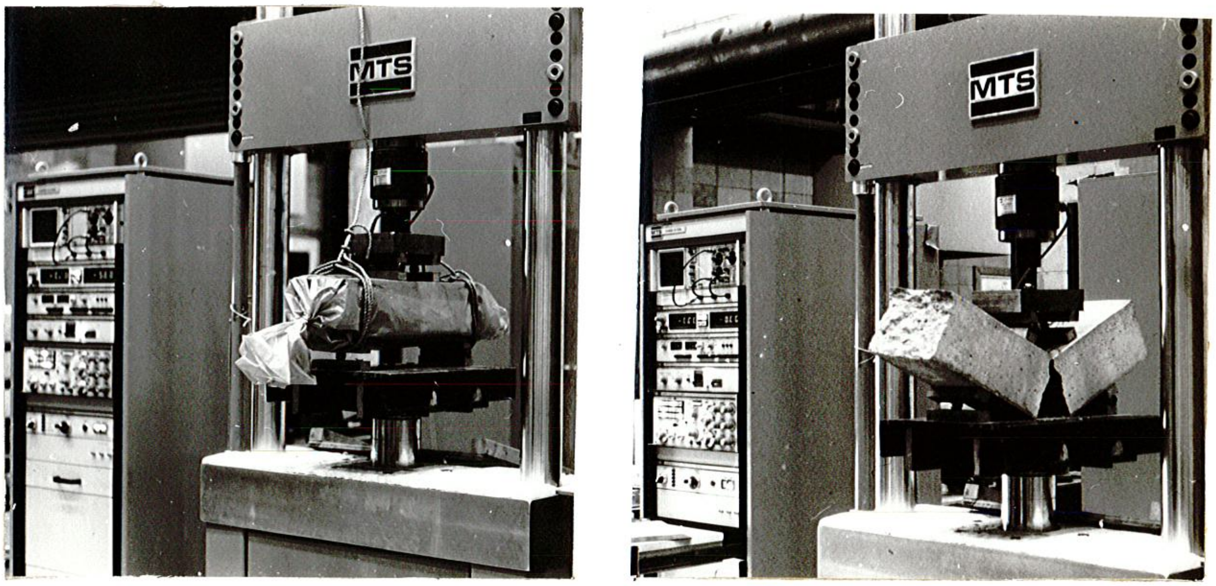
modulus of rupture test, the remaining portion of the test specimen, approximately 25 inches long, was placed in a Materials Test System, MTS, Model 810 testing system dynamic cycler for fatigue testing. Since the portion used in the fatigue test was the overhang portion of the beam, it was stress free in the modulus of rupture test. Flexural one-third point loading was applied using the same load configuration as in the modulus of rupture test. All hybrid beams were tested with the layer interfaces perpendicular to the load. The two layer beams were tested with the two-inch layer on the bottom, thus simulating the loading on a bridge deck over the support. The capacity of the MTS fatigue machine is ±110,000 pounds and a load can be applied at a frequency of .00001 to 990 hertz. Figure 7 shows the test machines utilized in the project.

High pressure air content was determined by the Materials Laboratory of the Iowa Department of Transportation. This process involves an oven-drying period of 72 hours at 300°F and a cooling period of 3 hours; the four-inch diameter cores used in this process were drilled from sections of tested fatigue beams. After these cores were weighed, they were soaked in water for 48 hours and then weighed in water. Upon removal of the cores from water, they were patted dry with a cloth and weighed again in air to determine absorption. The cores were then placed in the high pressure air meter and a pressure of approximately 5000 psi was applied. Air content was then determined using dial readings based on Boyle's Law. More details on this procedure can be found in Test Method No. Iowa 407-A, April 1971, Iowa Department of Transportation, Materials Department.



(a) 400^k machine with compressometer

(b) Modulus of rupture machine with beam



(c) MTS fatigue machine with beam of dense series

(d) MTS fatigue machine with beam of latex modified series

Figure 7. Test machines utilized

RESULTS AND DISCUSSION

Physical Properties

The experimentally determined physical properties of the four concretes are presented in the following sections. Table 3 summarizes these concrete properties. Two values of air content are presented for each concrete; the first value is the plastic air content and the second value is the hardened high pressure air content. As can be seen, there is some variation in the consistency of these two air tests. Two compressive strengths are also presented for each concrete; the first value is the 28-day compressive strength and the second value is the compressive strength obtained at the conclusion of the fatigue testing period. Thus this value is the compressive strength at 103 days for D-0, and D-1 and at 78 days for L-0 and L-1. The compressive strengths presented are the average of 3 compression tests, the modulus of elasticity value is the average of 2 tests and the modulus of rupture value is the average of all the beams containing only that concrete in question.

Compressive strength

Results of the 28-day compression tests as well as results of compression tests performed at the completion of the fatigue testing are shown in Figure 8. The dashed lines indicate the compressive strengths at 103 days for D-0 and D-1 and at 78 days for L-0 and L-1. This bar graph shows that the dense concrete had a higher compressive strength than the normal(D) concrete at 28 days. However, the compressive strength of the dense concrete was not nearly as high as expected. When properly

Table 3. Physical characteristics of concretes

Type of concrete	Air content, %	Slump, in.	Unit weight, pcf	Compressive strength, psi	Modulus of elasticity, psi x 10^6	Modulus of rupture, psi
Dense	5.9	2.5	137.2	6380	4.36	775.4
(D-1)	5.7			6700		
Normal(D)	4.8	4.0	140.6	5840	4.32	797.3
(D-0)	3.6			7060		
Latex	3.8	4.5	145.0	5490	4.12	599.4
(L-1)	8.8			6570		
Normal(L)	7.6	3.0	139.2	4020	3.98	551.6
(L-0)	9.7			4460		

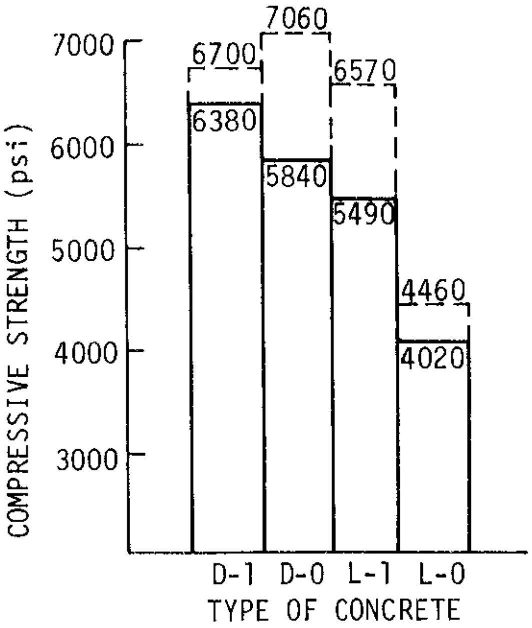


Figure 8. Results of compression tests

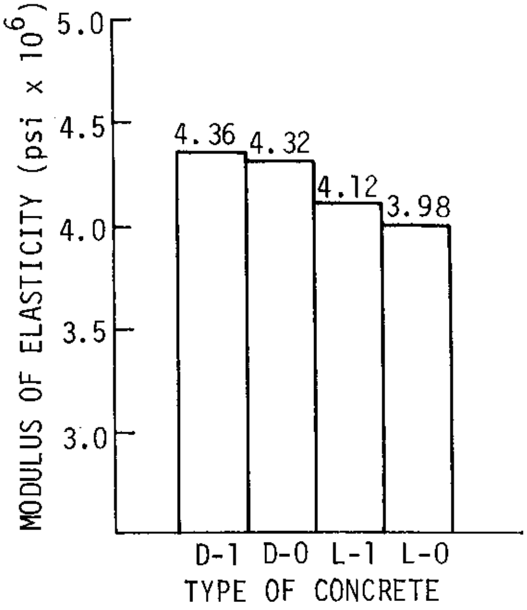


Figure 9. Results of modulus of elasticity tests

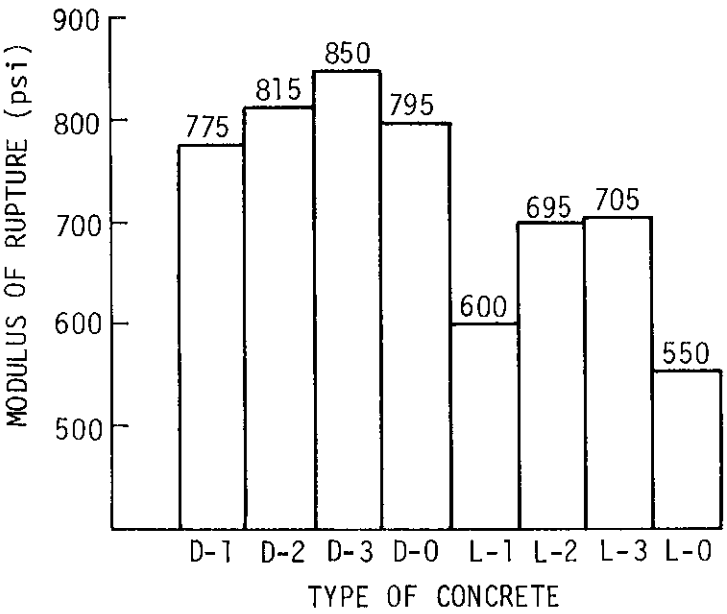


Figure 10. Results of modulus of rupture tests

mixed, consolidated and cured, the compressive strength of dense concrete should be approximately 9000 psi (11, 12), considerably higher than the 6400 psi compressive strength obtained. This lower compressive strength may be attributed to the fact that the transit mixer probably had excessive water in it upon arrival at the laboratory. The high slump obtained for the dense concrete is also an indication of the excessive water. The 2 1/2 inch slump obtained was considerably higher than the 3/4 in. ± 1/2 in. slump requirement for this concrete. Bergen and Brown, in their evaluation of bridge deck resurfacing (8) also did not obtain 9000 psi 28-day compressive strength for dense concrete. The compressive strength of normal(D) concrete was higher than expected, thus decreasing the difference between it and the strength of dense concrete. As has been experienced numerous times in the past when dealing with ready-mix concrete, the strength obtained was higher than that for which the concrete was designed. At the completion of fatigue testing, the compressive strength of normal(D) concrete was higher than that of dense concrete. Thus these concretes did not have the compressive strengths expected and desired.

As indicated in Figure 8, the compressive strength of latex modified concrete was higher than that of normal(L) concrete. The normal(L) concrete had the low compressive strength desired, giving a 1500 psi difference in the two strengths. This concrete was obtained from a ready-mix company utilizing a continuous mix truck. The mix lacked uniformity; an indication of this is the variation in compressive strengths found in Table B-1 in Appendix B.

Modulus of elasticity

As would be expected, the modulus of elasticity values follow the same trend as the compressive strength. Results of the modulus of elasticity tests are presented in Figure 9. The difference between the modulus of elasticity values of D-1 and D-0 is insignificant, likewise the differences in values of L-1 and L-0 is insignificant. These values closely follow the ACI standard 318-77 (10) relationship between modulus of elasticity and compressive strength, $E = w_c^{1.5}\, 33 \sqrt{f'_c}$ where w_c is the unit weight and f'_c is the compressive strength.

Modulus of rupture

Modulus of rupture tests were performed on each beam which was fatigue tested. This method of supplying a companion static test for each fatigue test is considered to be the most accurate when dealing with a nonhomogeneous material such as concrete. The variation in strength due to age is reduced since the modulus of rupture strength of each beam is determined at the time it is to be tested in fatigue. The results of the modulus of rupture tests are represented in Figure 10 as a bar graph. In the series of beams utilizing dense concrete and in the series of beams utilizing latex modified concrete, the three-layer beams had the highest modulus of rupture strength with the two-layer beams having the next highest strength. The dense concrete had a lower modulus of rupture value than the normal(D) concrete contrary to what would be expected.

Results of Fatigue Tests

Originally the flexural test specimens were to be cast at two different times. The first pour was to include the beams utilizing dense concrete. Twenty two-layer beams and twenty three-layer beams were to be poured along with five beams of dense concrete and five beams of only normal concrete. Fatigue results obtained from testing the two-layer and three-layer dense concrete specimens were to be compared. These results were also to be compared to the results of fatigue tests performed on normal concrete control beams to be cast at the time of the second pour. The beams containing only dense concrete and only normal concrete were to be tested for modulus of rupture comparison only. The normal concrete poured at this time was to be the same as the normal concrete to be placed in the second pour. This second pour was to include beams utilizing latex modified concrete. Due to lack of beam forms, three-layer latex modified concrete beams were not to be constructed. Two-layer latex modified concrete beams and normal concrete beams were to be tested in fatigue, thus twenty beams of each were to be poured. The normal concrete beams were to serve as the control for the entire project. Five latex modified concrete beams were to be poured so that the modulus of rupture of these beams could be compared with that of the layered beams.

After determining the compressive strengths of the normal concrete and dense concrete of the first pour, it was decided that the difference between these two strengths was not great enough for proper comparison, due in part to the high strength of the normal concrete. Therefore, a different normal concrete mix was designed for lower strength for use in the second pour. At this time it was also decided to test three-layer beams containing latex modified concrete along with the two-layer beams and the normal concrete beams. Due to lack of forms, only fourteen beams of each of the series to be fatigue tested were poured along with three latex modified concrete beams. Since the normal concrete of the first pour was not the same as that poured as a control in the second pour, it was decided to test the first normal concrete beams, normal(D), in fatigue also. Variation in the fatigue life of the dense concrete beams and the latex modified concrete beams with the two-and three-layer beams was of interest, thus the beams containing only dense concrete and only latex modified concrete were also fatigue tested. Even though very few data points would be obtained from testing the normal(D) concrete beams, the dense concrete beams and the latex modified concrete beams, it was felt that pertinent information could be obtained from these tests.

Consequently, eight series of beams were tested in fatigue; these included dense, (D-1), normal(D), (D-0), two- and three-layer dense, (D-2 and D-3), latex modified, (L-1), normal(L), (L-0) and a two- and three-layer latex modified, (L-2 and L-3).

The series of beams containing dense concrete and normal(D) concrete were tested at four stress levels (60%, 73%, 82% and 90%). Three stress levels (60%, 73% and 90%) were used for the series of beams utilizing latex modified concrete and normal(L) concrete since fewer beams were poured. Where the number of beams was extremely limited, Series D-0, D-1 and L-1, even fewer stress levels were used. Modulus of rupture strength, maximum load applied as a percentage of modulus of rupture, and fatigue life for each specimen are given in Tables B-2 through B-8 in Appendix B. Each specimen listed in these tables has a three digit designation, for example D-0-3. The first digit indicates the type of concrete used with normal concrete (dense or lated modified). The second number represents the number of layers in the beam; the zero indicates a normal concrete beam. The third number is the specimen number and ranges from 1 to the number of beams in that series. Specimens were loaded a minimum of one million cycles; the number of cycles recorded for those specimens which did not fail is the total number of cycles of load which had been applied to the beam at the time the test was stopped. The specimens were loaded such that the bottom fiber stress varied from essentially zero to a maximum value of 60, 73, 82 or 90 percent of the static modulus of rupture strength. Figure 11 shows the failure surfaces of two typical fatigue specimens. Since there is no visible difference between the modulus of rupture failure surfaces and the fatigue failure surface (26), only the later is shown. In all cases failure occurred through the coarse aggregate. The two-layer beam in Figure 11 exhibits a very even interface between the two concretes in the beam as was typical for the two-layer beams. The bottom interface of the three-layer beam in this figure was also even; the top layer had some variation in depth - a larger depth at the center of the beam than at the edges. This depth variation was fairly typical for the three-layer beams and was probably due to the fact that the concrete of the middle layer had a wetter

Figure 11. Failure surfaces of Series
L-2 and L-3

consistency than the concrete of the top layer. No evidence of failure of bond between layers was observed in any of the two-and three-layer beams.

Shown in Figures 12 to 21 are the S-N curves which have been plotted for each of the series of beams. These curves are the result of a log-log regression analysis plotted on a semilog scale. The computer program used for these plots, CENSOR, took into account the points representing beams which did not fail. A discussion of the regression analysis of the data and the equations of the curves appear in Appendix C.

The curves presented in Figures 12 to 15, representing the series of beams utilizing dense concrete and normal(D) concrete are combined on a semilog plot in Figure 6. Since the D-0 and D-1 curves are based on four data points, very little confidence can be put in them. The curves representing the two- and three-layer dense beams, (D-2 and D-3) were based on a minimum of four beams at each of the stress levels used, therefore, more confidence can be put in these curves.

As can be seen from the plot in Figure 16, the normal(D) concrete beams exhibit a much higher fatigue life at the lower stress levels than the beams utilizing dense concrete. The dense concrete beams have the lowest fatigue life at all but the lower stress levels. The D-2 and D-3 curves lie close together and cross the D-0 curve and each other at stress levels between 75 and 85% of modulus of rupture; thus the layered beams have a higher fatigue life than the normal concrete at high stress levels. A possible hypothesis for this behavior would be that the dense concrete very likely contains fewer voids than the normal concrete, thus,

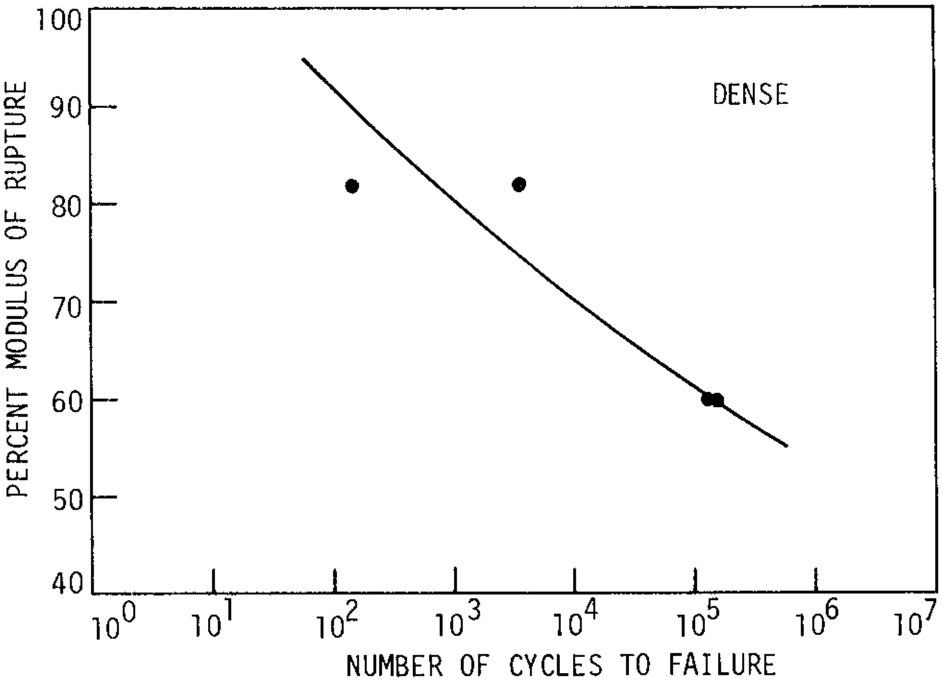


Figure 12. S-N curve for Series D-1

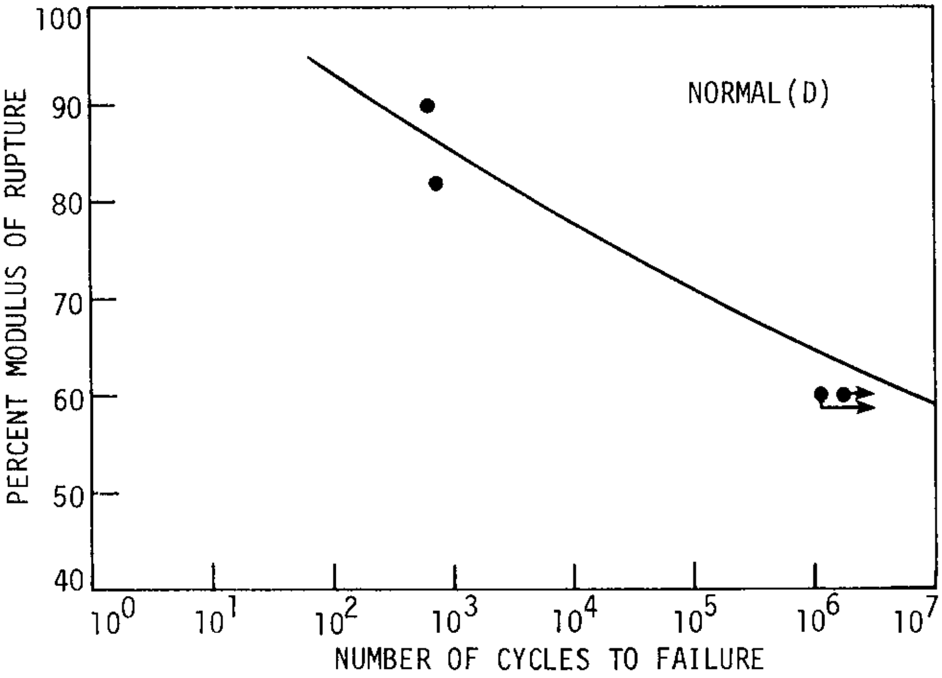


Figure 13. S-N curve for Series D-0

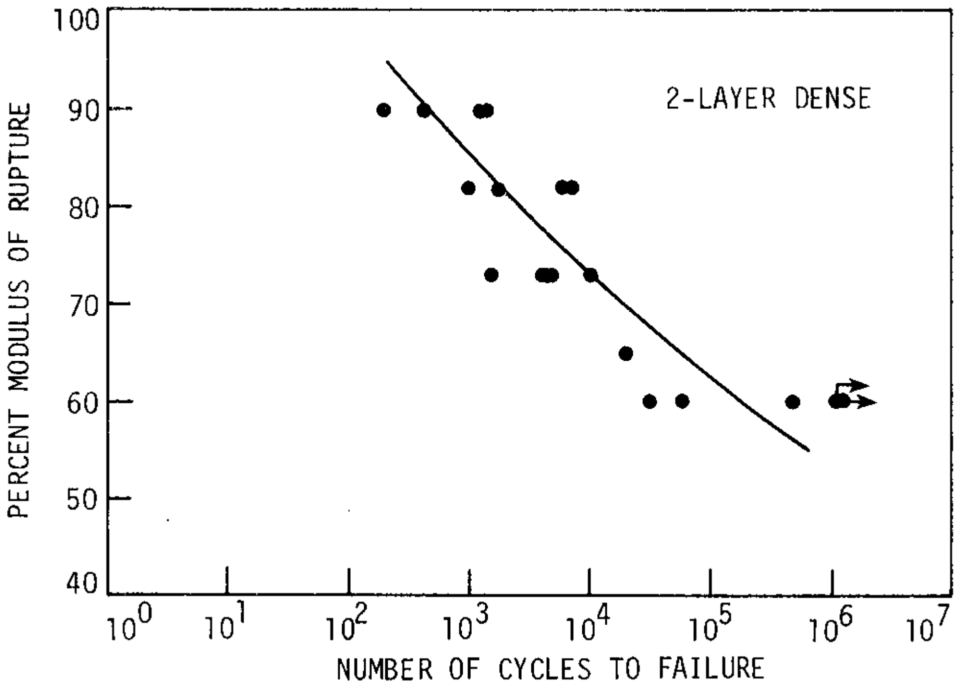


Figure 14. S-N curve for Series D-2

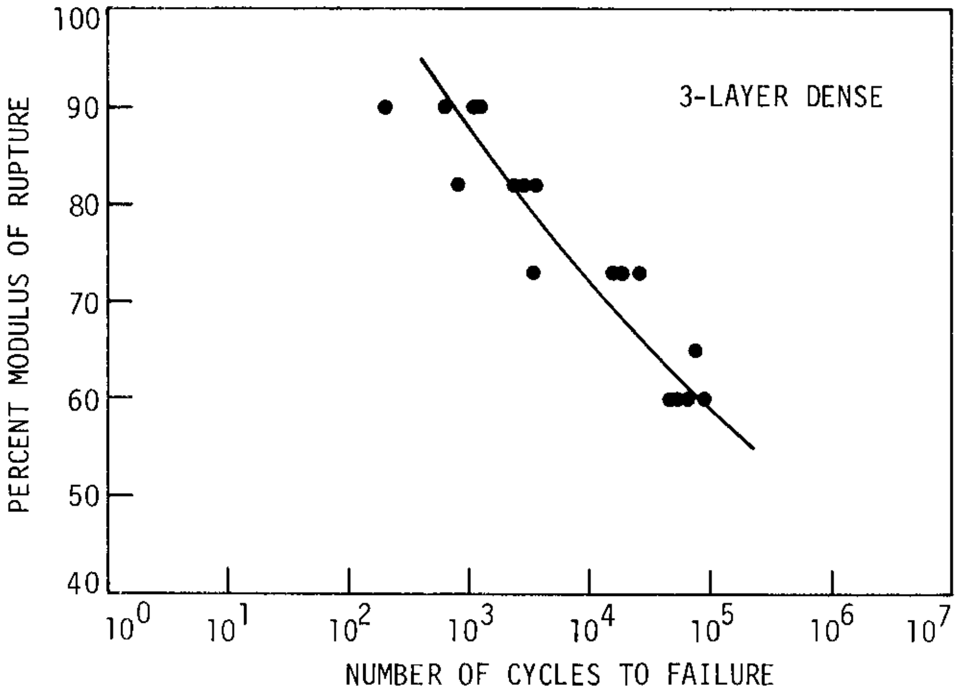


Figure 15. S-N curve for Series D-3

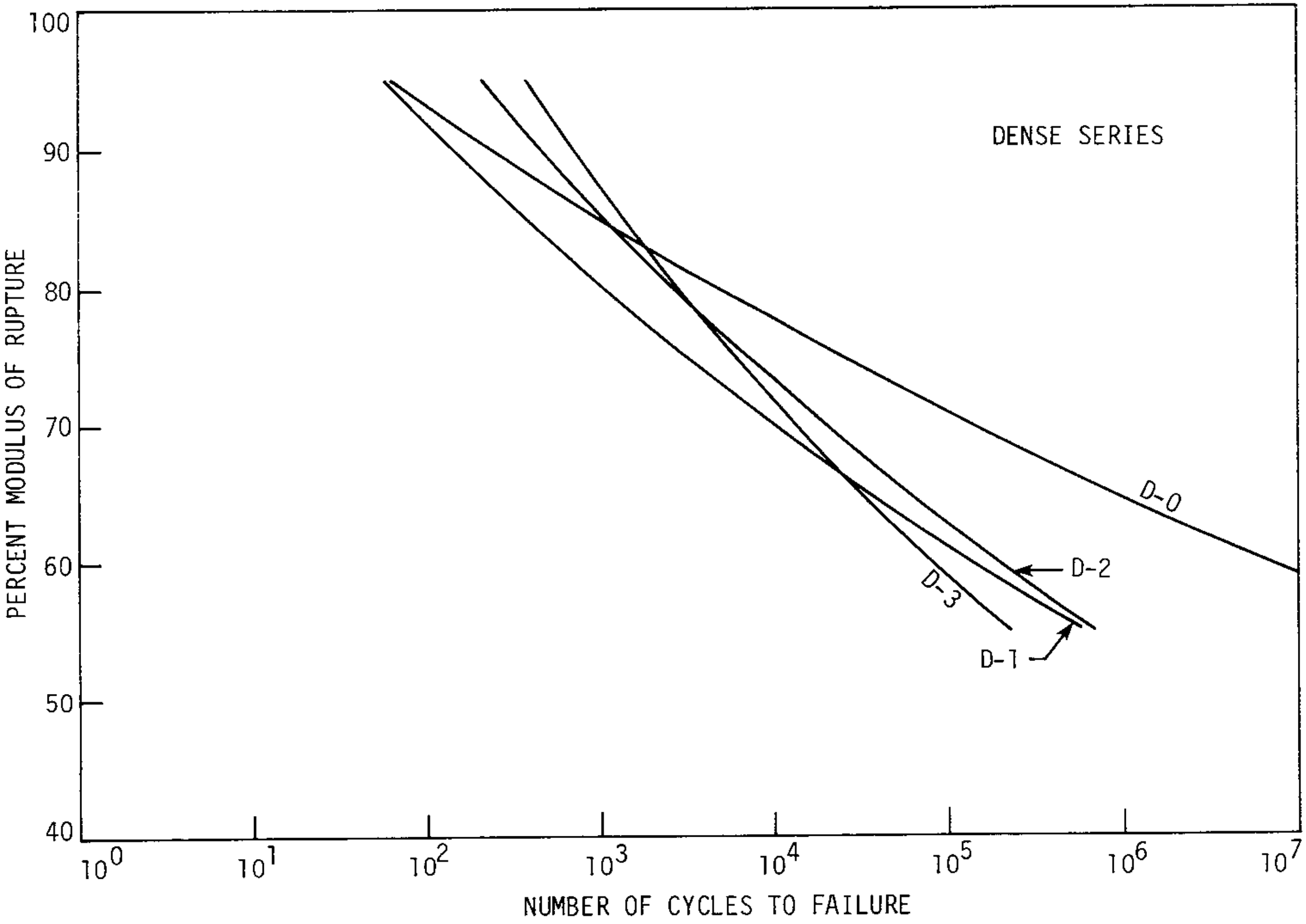


Figure 16. Composite S-N curves for dense series

at lower stress levels the normal concrete will have a higher fatigue life since the voids would act to arrest propagating cracks. At higher stress levels, the voids could conceivably cause stress concentrations, thus the normal concrete has a lower fatigue life at these stress levels. One might expect the D-2 and D-3 curves to lie to the right of the D-0 and D-1 curves since the D-2 and D-3 series exhibited higher modulus of rupture strengths than the D-0 and D-1 series, however this was not the case. The three curves representing beams containing dense concrete all lie close together. This might be expected since they all contain dense concrete at the surface where microcracks would begin propagating. Due to the limited number of specimens tested in fatigue, confidence limits were not plotted for these curves. Six specimens must be tested at each stress level for 95% confidence limits to be used (22). If confidence limits were plotted for the curves representing beams containing dense concrete, they would very likely overlap. Thus essentially the same fatigue strength was obtained using either an entire beam of dense concrete or a beam containing only a thin layer of dense concrete at the surface.

The effect of compressive strength on the flexural fatigue strength is difficult to determine since the compressive strength of the dense concrete was higher than that of the normal(D) concrete at the beginning of the fatigue testing period and lower than that of normal(D) concrete at the end of the fatigue testing period. Kesler (23) concluded from his studies of concrete with compressive strengths of 3600 psi and 4600 psi that the fatigue lives of these two concretes were the same when tested at speeds of 70, 230, and 440 cycles per minute. To the author's knowledge, no other study comparing the flexural fatigue life of concretes with different static compressive strengths have been conducted. Gray, McLaughlin and Antrim (19) compared the compressive fatigue properties of high strength lightweight aggregate concrete with low strength lightweight aggregate concrete. They concluded that the compressive fatigue properties of the two concretes were the same regardless of differences in static properties. This would agree with Kesler's conclusions on flexural fatigue of concretes with different compressive strengths. This was essentially found to be true in this investigation also.

The S-N curves for the series of beams utilizing latex modified concrete and normal(L) concrete are presented individually in Figures 17 to 20 and are combined in Figure 21. The point designated as L-1 in Figure 21 represents the average of three beams tested at 73% of modulus of rupture. As may be observed, this point lies between the L-2 and L-3 curves and a considerable distance to the left of the L-0 curve. However, the individual data points for both the L-0 and L-1 curves lie in the same range of values. The curves for two- and three-layer beams again lie close together and cross the curve for the normal(L) concrete at the same stress range that the D-2 and D-3 curves cross the D-0 curve. If the latex modified concrete exhibits the same trends that the dense concrete exhibits one would expect the L-1 curve to lie close to the L-2 and L-3 curves and have a shape very similar to them. A possible explanation for the lower fatigue strength of L-1, L-2, and L-3 at the lower stress levels may again be found in the void structure of the concrete. The latex

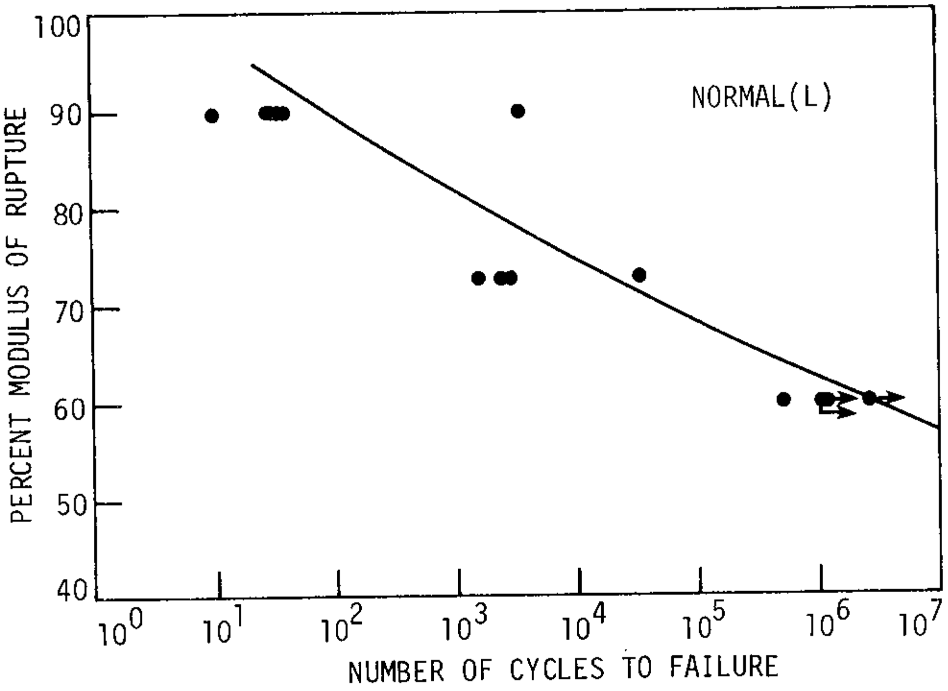


Figure 17. S-N curve for Series L-0

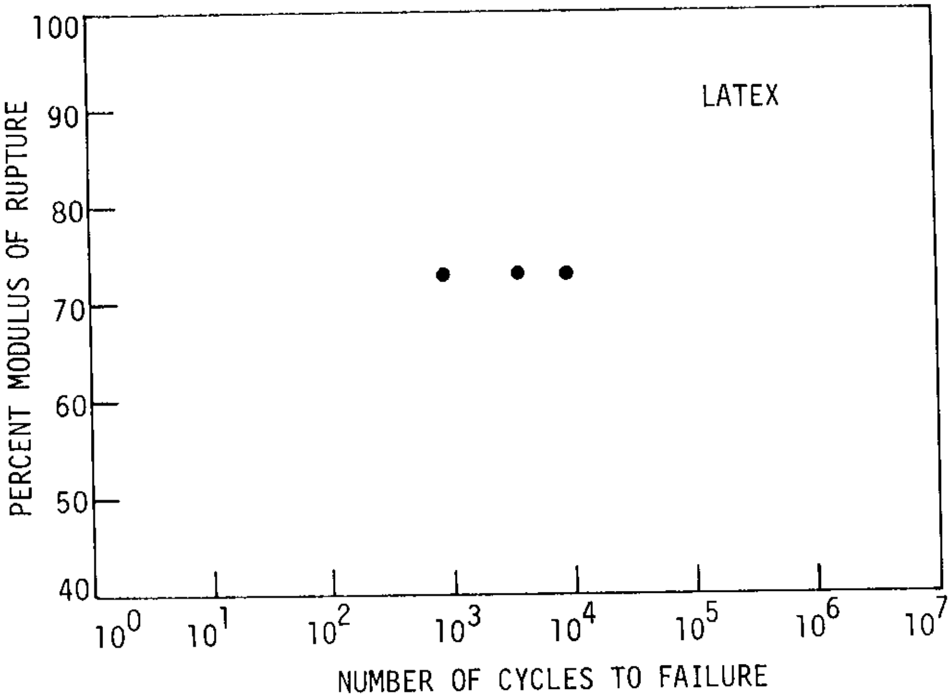


Figure 18. S-N curve for Series L-1

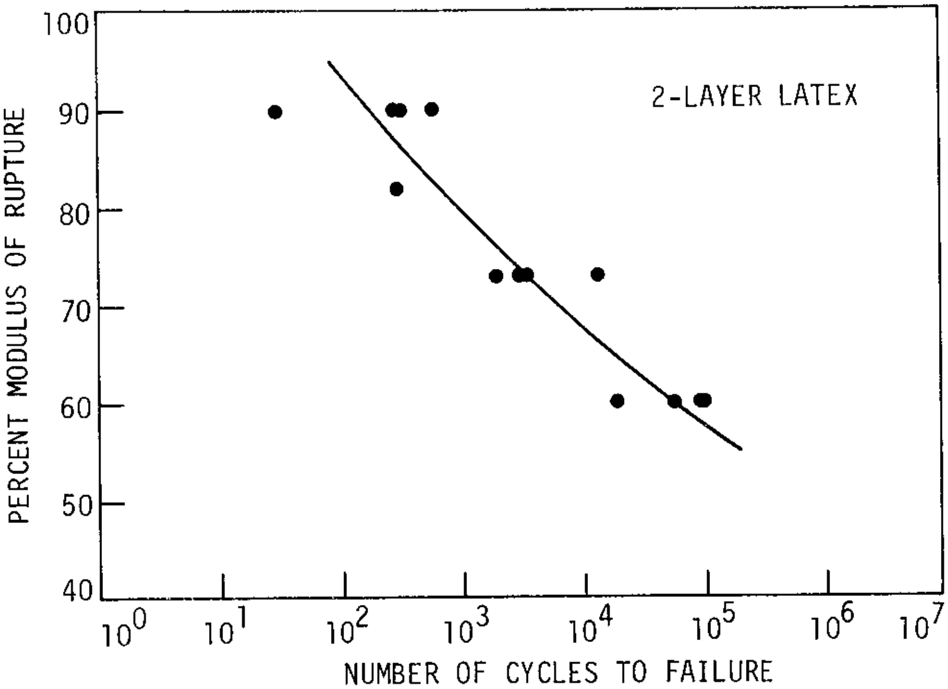


Figure 19. S-N curve for Series L-2

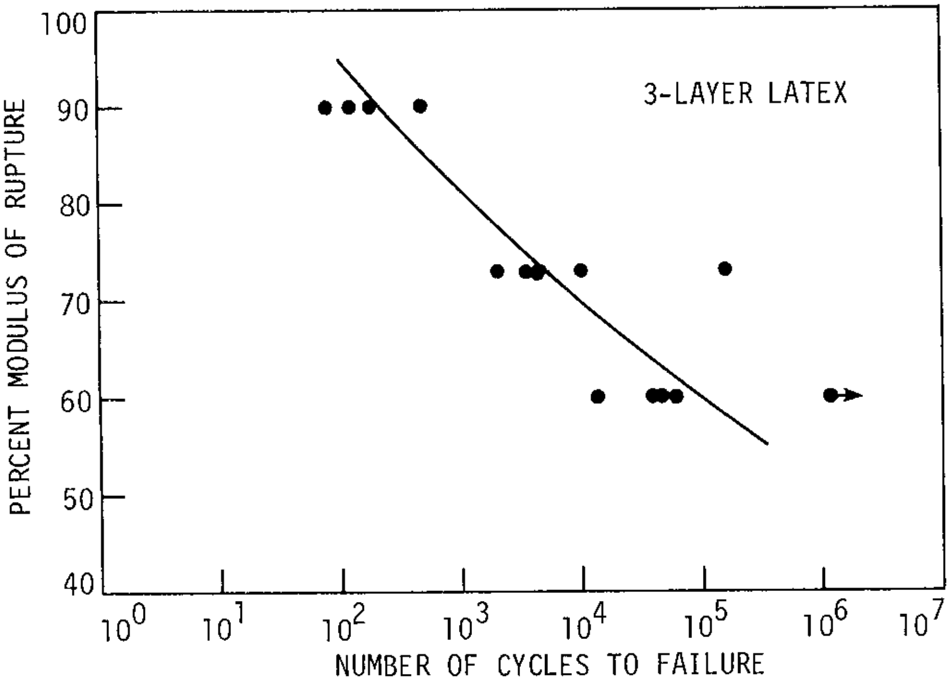


Figure 20. S-N curve for Series L-3

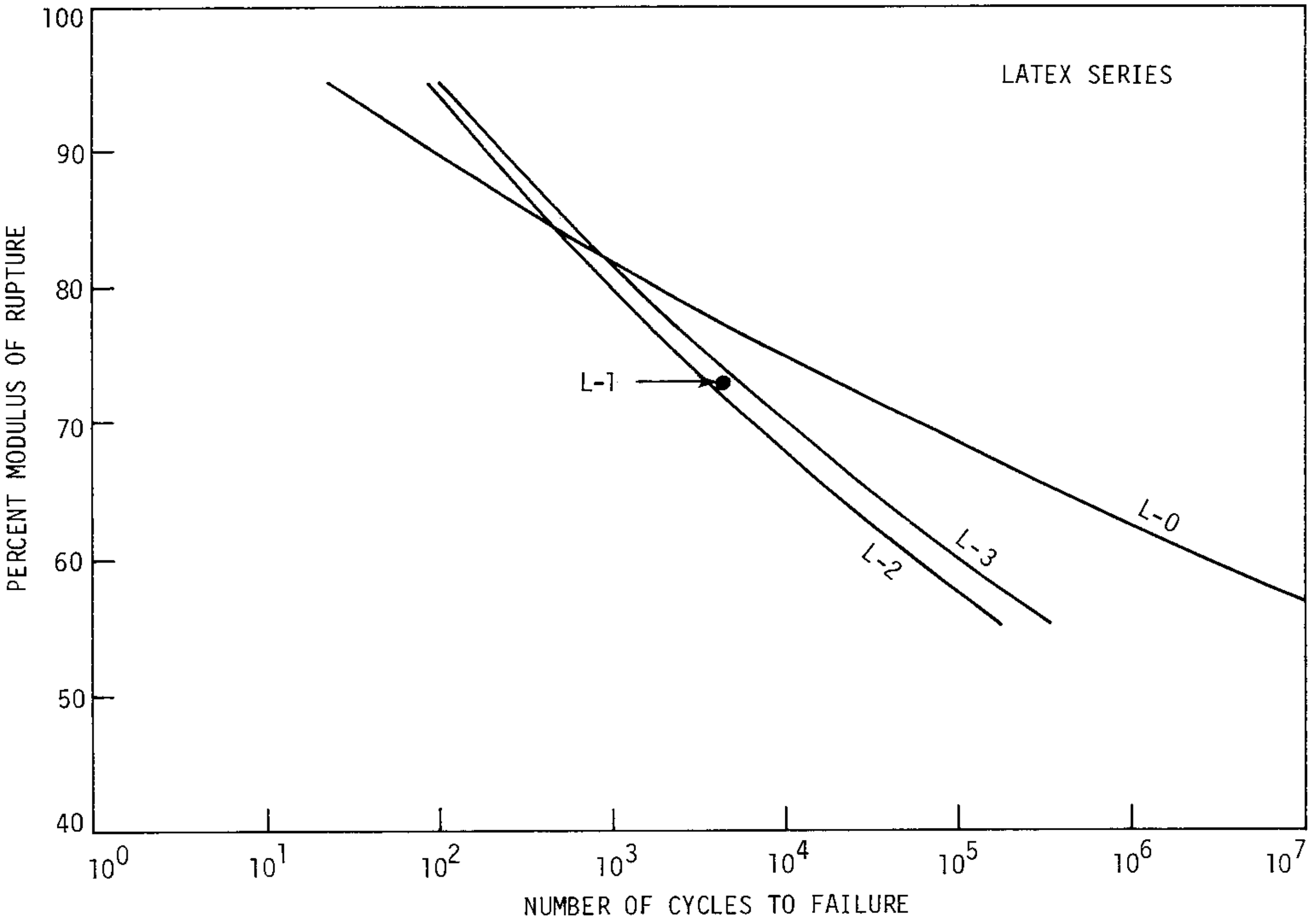


Figure 21. Composite S-N curves for latex series

modified concrete contained a smaller percentage of air, thus it may have fewer voids than the normal(L) concrete to arrest propagating cracks.

Again the effect of compressive strength on the flexural fatigue strength is difficult to determine. The difference between the L-0 curve and the L-2 and L-3 curves at the lower stress levels appears to be a significant amount. As in the dense series, the modulus of rupture strengths of the two- and three-layer series are much higher than the modulus of rupture strengths of the normal(L) and latex modified series and one would then expect the fatigue lives for L-2 and L-3 to be higher than those for L-0 and L-1; this is not the case. Looking at both the dense series and latex modified series, the normal concrete exhibits a higher fatigue life at the lower stress levels in both cases. However, in the first case, the compressive strength of the normal concrete was higher than the compressive strength of the dense concrete. In the second case the compressive strength of the normal concrete is lower than that of the latex modified concrete but the fatigue curves still exhibit the same trend.

As evidenced in Figures 16 and 21 the fatigue life of the normal concrete beams is decreased by the addition of layers of dense concrete and latex modified concrete at stress levels lower than 75% modulus of rupture, but is increased by these additions at higher stress levels.

SUMMARY AND CONCLUSIONS

Summary

Deterioration of highway bridge decks caused by deicing salts has led to the use of the "Iowa method" low-slump or dense concrete and latex modified concrete in resurfacing these decks. Both of these concretes exhibit improved impermeability, thus decreasing the speed of deterioration due to deicing salts. The fatigue properties of these concretes are virtually unknown; this study investigated the fatigue properties of layered beams using these concretes.

The results of the study on the fatigue behavior of layered hybrid beams using dense concrete and latex modified concrete is presented herein. Tests were conducted on eight series of beams, four utilizing dense concrete and four utilizing latex modified concrete. The four series included two-layer and three-layer beams, plain normal concrete beams and either dense concrete beams or latex modified concrete beams. Fatigue specimens consisted of 6 in x 6 in. x 36 in. beams subjected to flexural loading under a zero to maximum load cycle. The maximum bottom fiber stress for a specific test was usually either 60%, 73%, 82% or 90% of the static modulus of rupture strength.

Dense concrete and latex modified concrete were mixed and poured in the laboratory; the normal concretes were purchased from a ready-mix company and poured in the laboratory. The series of beams utilizing dense concrete were stored submerged in water while the series of beams utilizing latex modified concrete were air cured.

Fatigue tests were conducted using an MTS Model 810 dynamic cycler utilizing third point loading identical to that used in the modulus of rupture tests.

Results of these tests indicate that the fatigue life of normal concrete beams is larger than the fatigue life of the two- and three-layer beams and the beams of dense concrete and latex-modified concrete at stress levels lower than 75% modulus of rupture but smaller at higher stress levels.

Investigations determining the compressive strength, modulus of rupture strength, and modulus of elasticity were also conducted. These results are presented in tabular and graphical form. Hardened air contents were determined by use of the high pressure air meter method.

Conclusions

Although based on a very limited number of data points, the following conlcusions can be reached as a result of the tests performed in this investigation:

1) The fatigue life of normal concrete beams is considerably higher than the fatigue life of layered concrete beams utilizing dense concrete or latex modified concrete at the lower stress levels but lower at the higher stress levels.
2) Essentially the same fatigue strength was obtained for entire beams of either dense concrete or latex modified concrete as for beams containing only a small amount of these concretes at the surface.

3) No failure of bond between layers occurred in the layered beams during either static or fatigue testing.
4) The modulus of elasticity values determined experimentally for the four concretes correspond to those values determined using ACI standard 318-77 (10).
5) Although dense concrete and latex modified concrete have been used successfully in resurfacing bridge decks for a number of years, a literature search indicates that little is known about the fatigue properties of these concretes. Thus, much research still needs to be done to better understand the fatigue behavior of bridge decks overlaid with dense concrete and latex modified concrete.

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APPENDIX A: MATERIAL CHARACTERISTICS AND PROPORTIONS

Table A-1. Gradation of fine aggregate

	Percent Passing	
Sieve size	Hallett sand	Iowa D.O.T. specifications
3/8 in.	100	100
No. 4	99.0	90-100
No. 8	85.2	70-100
No. 16	65.0	-
No. 30	40.2	-
No. 50	8.2	-
No. 100	0.4	-
No. 200	0.1	0-1.5

Table A-2. Gradation of coarse aggregate

	Percent passing		
Sieve size	Alden limestone	Ferguson limestone	Iowa D.O.T. specifications
1 1/2 in.	100	100	100
1 in.	93.3	100	95-100
3/4 in.	70.4	66.9	-
1/2 in.	36.7	37.5	25-60
3/8 in.	15.1	20.1	-
No. 4	1.14	3.8	0-10
No. 8	0.26	1.3	0-5
No. 200	0.12	0.14	0-1.5

Table A-3. Cement properties

Property		Test value	Specification (ASTM C150 and Fed. SS-C-1960/3)
Chemical data			
SiO_2		22.0	-
AlO_3		5.4	7.5 max.
FeO_3		2.3	6.0 max.
CaO		64.5	-
MgO		2.3	5.0 max.
SO_3		3.0	3.5 max.
Loss on ignition		1.0	3.0 max.
Insoluable residue		.4	.75 max.
Physical data			
Fineness, Blaine - cm^2/gm		3700	2800 min.
Soundness, Autoclave - %		.4	.80 max.
Time of set, Vicat - minutes	Initial	80	45 min.
	Final	190	480 max.
Air content - %		10	12.0 max./min.
Compressive Strength - psi:	3 day	2900	1800 min.
	7 day	3990	2800 min.

DOW MODIFIER A SPECIFICATIONS

The latex shall be a styrene-butadiene polymeric emulsion in which the polymer comprises 46.5% to 48.0% of the total emulsion. The polymer shall contain 66 ± 1 1/2% styrene and 34 ± 1 1/2% butadiene. The polymeric emulsion shall be stabilized with an anionic, non-ionic and polydimethylsiloxane fluid surfactent in which the anionic surfactent is a sodium alkyl sulfate. The polymer particles shall have a average size of 1900 to 2500 angstroms. The latex shall have a pH of 9.5 to 11.0 and a weight at 25°C of 8.40 to 8.47 pounds per gallon.

Table A-4. Laboratory batch quantities

Series	W/C	Cement, lb	Water[a] lb	Sand, lb	Coarse Aggregate, lb	Latex, gal	Ad Aire, ml	Water Reducer, ml	Yield yd^3
D-0	.43	626	208	1547	1514	0	237	0	1.0
D-1	.32	619	226	1029	974	0	154	733	.75
L-0	b	336	163	1273	1273	0	177	0	.75
L-1	.35	428	70	1058	798	15.93	0	0	.65

[a] Includes water required to bring aggregate to saturated surface dry condition.

[b] The water-cement ratio varied during the pour since a continuous mix truck was used.

APPENDIX B: TEST DATA

Table B-5. Fatigue test data for Series D-2

Specimen	Modulus of rupture, psi	Percent modulus of rupture	Fatigue life, number of load applications for failure
D-2-11	805	90	1,390
D-2-19	780	90	420
D-2-1	792	90	206
D-2-6	784	90	1,260
D-2-4	790	82	1,750
D-2-9	769	82	6,300
D-2-2	830	82	6,740
D-2-14	841	82	990
D-2-12	800	73	4,590
D-2-20	810	73	10,040
D-2-3	865	73	1,560
D-2-17	840	73	4,410
D-2-16	861	73	4,360
D-2-8	807	65	19,590
D-2-10	837	60	58,630
D-2-5	832	60	498,160
D-2-13	817	60	31,860
D-2-18	812	60	1,144,040[a]
D-2-7	775	60	1,217,250[a]

[a] Failure did not occur.

Table B-6. Fatigue test data for Series D-3

Specimens	Modulus of rupture, psi	Percent modulus of rupture	Fatigue life, number of load applications for failure
D-3-5	827	90	1,130
D-3-7	830	90	1,160
D-3-9	867	90	200
D-3-15	820	90	620
D-3-3	790	82	3,500
D-3-18	876	82	2,810
D-3-10	890	82	800
D-3-14	808	82	2,320
D-3-8	841	73	15,050
D-3-2	893	73	24,450
D-3-11	803	73	17,920
D-3-17	849	73	3,260
D-3-4	850	65	74,010
D-3-1	923	60	88,600
D-3-6	850	60	64,240
D-3-16	829	60	51,710
D-3-12	864	60	46,480

Table B-7. Fatigue test data for Series L-0

Specimen	Modulus of rupture, psi	Percent modulus of rupture	Fatigue life, number of load applications for failure
L-0-9	570	90	30
L-0-5	620	90	10
L-0-3	574	90	3,471
L-0-17	670	90	30
L-0-15	519	90	20,964
L-0-11	560	90	31
L-0-14	510	90	37
L-0-4	525	73	1,510
L-0-10	450	73	2,810
L-0-1	521	73	33,440
L-0-7	550	73	2,310
L-0-16	635	60	1,049,440[a]
L-0-8	585	60	524,300
L-0-6	475	60	1,158,100[a]
L-0-2	489	60	872,350[b]
L-0-12	588	60	588,740[c]
L-0-13	523	60	2,614,900[a]

[a] Failure did not occur.

[b] Specimen was stopped at 6970 cycles due to machine malfunction but did not break.

[c] Specimen was stopped at 588,740 cycles - did not fail.

Table B-8. Fatigue test data for Series L-2

Specimen	Modulus of rupture, psi	Percent modulus of rupture	Fatigue life, number of load applications for failure
L-2-13	700	90	320
L-2-2	750	90	580
L-2-11	730	90	29
L-2-9	626	90	275
L-2-5	715	82	290
L-2-6	710	73	3,330
L-2-4	730	73	12,960
L-2-1	673	73	2,890
L-2-7	685	73	1,870
L-2-3	715	60	89,000
L-2-12	710	60	52,320
L-2-10	660	60	93,350
L-2-8	655	60	18,850

Table B-9. Fatigue test data for Series L-3

Specimens	Modulus of rupture, psi	Percent modulus of rupture	Fatigue life, number of load applications for failure
L-3-14	740	90	180
L-3-5	622	90	120
L-3-2	767	90	470
L-3-9	817	90	75
L-3-6	649	73	3,390
L-3-4	657	73	4,320
L-3-11	653	73	9,860
L-3-8	700	73	159,309
L-3-13	645	73	1,960
L-3-3	627	60	1,116,080[a]
L-3-1	818	60	38,500
L-3-12	750	60	59,960
L-3-10	730	60	19,630
L-3-7	673	60	45,110

[a] Failure did not occur.

APPENDIX C: STATISTICAL REGRESSION ANALYSIS

In some investigations, the experiment is stopped or "censored" before all of the observations are made; nothing is known about these observations except that they will occur at a later time. Fatigue research in which specimens at low stress levels do not fail is one such investigation.

In fitting a regression equation to a set of observations in which the experiment was censored, statisticians use the theory that a weighted average can be made and a value can be assigned to the set of data considering all observations. This theory has recently been implemented into a computer program. This program, CENSOR, was developed by Dr. William Meeker, Associate Professor of Statistics at Iowa State University and was used in this investigation.

The equations of the curves shown in Figures 14 through 23 of this report are of the form:

$$\mathrm{Log}(N) = b_0 + b_1 \mathrm{Log}(S)$$

where

N = number of cycles to failure

b_0 = y-intercept of the curve

b_1 = slope of the curve

S = percent modulus of rupture (i.e., 90, 80, ...).

The slopes and y-intercepts of the curves are given in Table C-1. All specimens in Series D-1, D-3, and L-2 failed; thus a standard log-log regression program was used for the analysis of these data. Since Series L-1 contained only three data points a regression analysis was not performed on this data.

Table C-1. Constants for fatigue equations

Series	y-intercept, b_0	Slope, b_1	Figure in which curve appears
D-1	35.1500	-16.8800	14
D-0	51.5062	-25.1341	15
D-2	31.5163	-14.7559	16
D-3	25.6700	-11.6700	17
L-0	51.5714	-25.3917	19
L-2	29.8400	-14.1200	21
L-3	31.8273	-15.0853	22