Effects of Overloads on Deterioration of Concrete Bridges

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An overview of the problem of predicting the effects of overloads on highway bridges is presented, with emphasis on understanding the mechanisms responsible for progressive overload-induced damage to concrete decks. Interaction between physical damage directly attributable to wheel loads and other damage mechanisms such as corrosion is discussed. Existing methods of linear and nonlinear analysis of bridges and rating methods are reviewed. The rapidly developing field of fracture mechanics of concrete and its application to concrete bridge decks is discussed.

The most important economic issue facing the transportation community today is the deterioration of the roadways and structures comprising the nation's highway system. The significance of this problem is emphasized by Turner *(1)*, who also proposes a solution to the perceived rapid deterioration. To reduce the rate of traffic-induced deterioration, primarily of pavements, Turner proposes a redesign of the nation's truck fleet over a 10-year period to reduce the equivalent single-axle loading (ESAL) of the various vehicles by reducing the legal single- and tandem-axle limits to 15,000 and 25,000 lb, respectively. To maintain transportation productivity, Turner proposes an increase in gross vehicle weights to approximately 112,000 lb, with a corresponding increase in the length and number of axles to accomplish the reduced ESAL. This proposal has considerable merit; however, rational assessment of the economic impact of changes in the nation's truck fleet is not a simple matter. The effects of changes in ESAL on flexible and rigid pavements can be evaluated by techniques and data originating in the 1960 Illinois Road Test; however, the effects of heavy vehicles on highway bridges, particularly on bridge decks, are not so well understood. There is considerable economic pressure to increase transportation sector productivity by increasing legal vehicle weights. A recent study *(2)* has resulted in a recommended change in the formula regulating vehicle and axle weights, the so-called bridge formula, to allow longer and heavier vehicles, but to reduce the legal weights of some shorter vehicles, independent of changes in single- and tandem-axle weight limits.

Methods and data to quantify the effects of heavy-truck traffic on highway bridges have not been available in sufficient detail to provide meaningful analyses. Brown et al. *(3)* were forced to omit consideration of the increased cost of maintenance associated with a hypothetical scenario of increased weight limits, citing, ". . . the lack of technology regarding the effects of heavy loading and frequency on bridge deterioration." Some facts are clear, for example, that bridge deterioration is to some extent accelerated by increased truck traffic, both by numbers of trucks and by higher gross vehicle weights. Higher speeds also increase the rate of damage; Turner *(1)* speculates that the actual stress increase due to dynamic effects on bridge decks may be as much as twice the value anticipated by current design methods. The most significant manifestation of bridge damage due to truck traffic is damage to the deck, with damage rates to other structural components being less significant.

Deck damage may take numerous forms; however, the most important damage mechanisms are transverse cracking and longitudinal cracking. Wheel-load-related deck cracking is worse on structures having lower ratios of dead load to total load. Reinforced-concrete decks on steel I–beams are more susceptible to damage of this type than are decks on prestressed girders, probably because of the inherently greater flexibility of the steel stringer bridges. Overweight vehicle damage to bridges, especially deck cracking, is interrelated with other progressive damage mechanisms (such as corrosion) in a complex manner.

Corrosion of reinforcement is intensified by the increased cracking caused by overloaded vehicles, and spalling of concrete cover resulting from reinforcing steel corrosion is certainly accelerated by traffic. Cady and Weyers *(4)* present a method for estimating the deterioration rate of concrete bridge decks due to corrosive attack, but the method does not involve parameters that depend on traffic density or presence of overloaded vehicles. Damage mechanisms affecting steel floor system members are much better understood than are damage mechanisms affecting concrete beams and girders, and the influence of overloads on steel bridge members is not considered here.

REVIEW OF RECENT RESEARCH

Field Studies of Progressive Damage

In a Texas SDHPT-sponsored study in progress, a preliminary survey of approximately 25 structures was accomplished to identify candidate structures for more detailed study. Structures along routes carrying high levels of heavy truck traffic were surveyed in the preliminary study to identify those structures in which the effects of differential truck traffic could be observed by comparison to control structures. The results of the preliminary study indicated that an observable correlation of some

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forms of progressive damage induced by heavy trucks could be seen on some of the 25 structures surveyed. In particular, qualitatively higher levels of deck cracking were observed in concrete decks on steel stringer bridges in the structures carrying heavier truck traffic. Other forms of progressive damage were observed, but not so clearly attributable to increased levels of truck traffic.

From the results of the preliminary survey, two candidate structures were selected for further study, along with two control structures. All structures were simple-span bridges, having steel stringers and reinforced-concrete decks. Each candidate structure was of essentially the identical design, construction, and age as the corresponding control structure, and being located on the same route of a divided highway, each candidate structure and corresponding control structure carried approximately the same levels of traffic, with the notable exception of the differential truck traffic from several aggregate quarries located north of the sites. Loaded trucks from these quarries travel southbound to the cities of Ft. Worth and Dallas, and return northbound to the quarries. The candidate structures carrying the southbound lanes experience a significantly higher level of heavy-truck traffic than do the control structures that carry the northbound traffic. This differential truck traffic is heavy, being estimated at approximately 180 veh/hr. This differential traffic is thought to have been experienced by these structures for approximately 26 years. Some of the results of this field study are summarized here.

Procedure

A 12-ft-long section across the width of the span of the US–287 bridge over FM–730 was inspected to investigate the variation in cracking across the width of a span. This 12-ft section represented slightly more than 20 percent of the span length. This area was divided into nine regions, consisting of three regions across the width of the span by three along the length. The regions or panels referred to in the figures are 4-ft lengths across the width of the span. Each region represents a 4-ft-long area between two of the four bridge stringers. The designations left, center, and right refer to the regions between two of the stringers in relation to the flow of traffic.

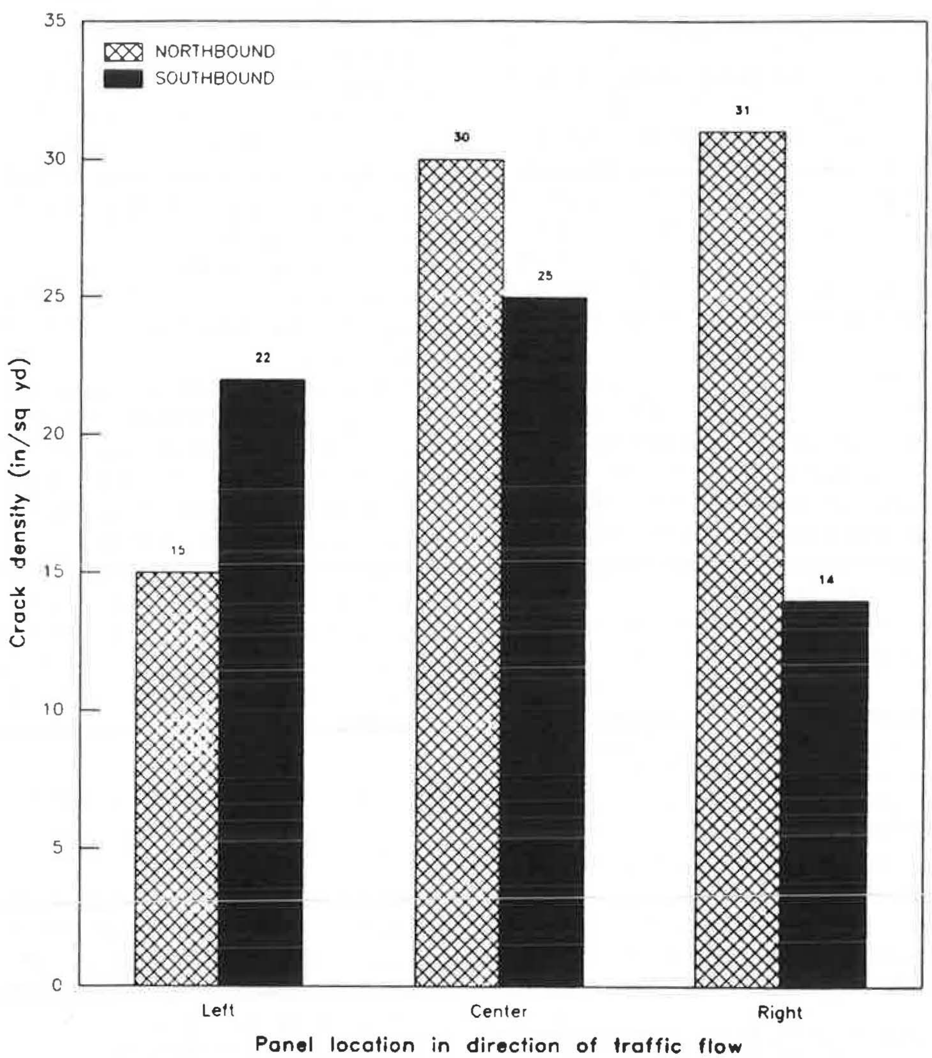

FIGURE 1 Average crack density in reinforced concrete decks subjected to different levels of traffic.

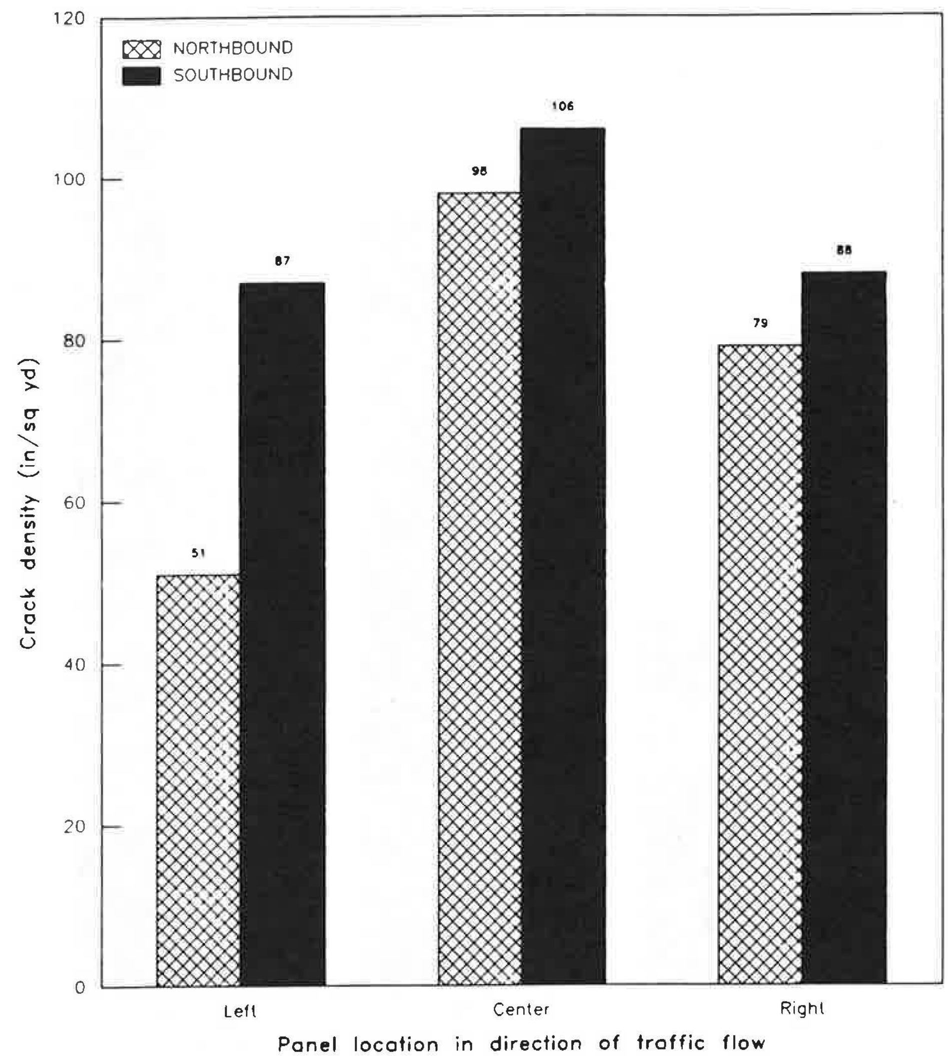

FIGURE 2 Longitudinal crack density in reinforced concrete decks subjected to different levels of traffic.

Cracking on the lower surface of the deck was observed visually and marked by hand. The regions were photographed, allowing office study of the crack patterns. In the analysis of the cracks, cracks oriented within 30 degrees of a line perpendicular to the flow of traffic were denoted transverse cracks, cracks oriented within 30 degrees of a line parallel to the flow of traffic were denoted longitudinal cracks, and all other cracks were denoted diagonal cracks. The crack density reported is the sum of the length of the observed cracks divided by the area of the region of study.

Although both bridges were subject to loading, the southbound bridge, carrying the heavier traffic loads is referred to as the loaded structure, and the northbound bridge is referred to as the unloaded structure. Some of the preliminary observations of crack densities and orientations are summarized in Figures 1–4.

Observations of Total Crack Densities

The observed regions of the loaded bridge almost always exhibited more cracking than the corresponding regions of the unloaded bridge deck. In the loaded bridge, the left regions were always cracked more, by an average of 65 percent, than the left regions of the unloaded bridge deck. The center and the right regions of the loaded bridge usually exhibited greater crack densities, by approximately 6 percent more than the corresponding regions of the unloaded structure.

For both the loaded and unloaded bridges, the center regions always exhibited more cracking than the left regions. On average in the center regions there was 124 percent more cracking for the unloaded bridge and 43 percent more cracking for the loaded bridge. The center regions also usually exhibited more cracking than the right regions in both bridges, averaging 8 percent more for the unloaded bridge and 9 percent more for the loaded bridge. The right regions always exhibited more cracking than the left regions in the unloaded bridge, averaging 107 percent more cracking. In the loaded bridge, the right regions usually exhibited more cracking than the left, averaging 31 percent more cracking.

The difference in crack density between the loaded and unloaded bridges is far greater in the left regions of the bridge deck than in the center or right regions. The more heavily cracked center and right regions have smaller differences between the loaded and unloaded spans.

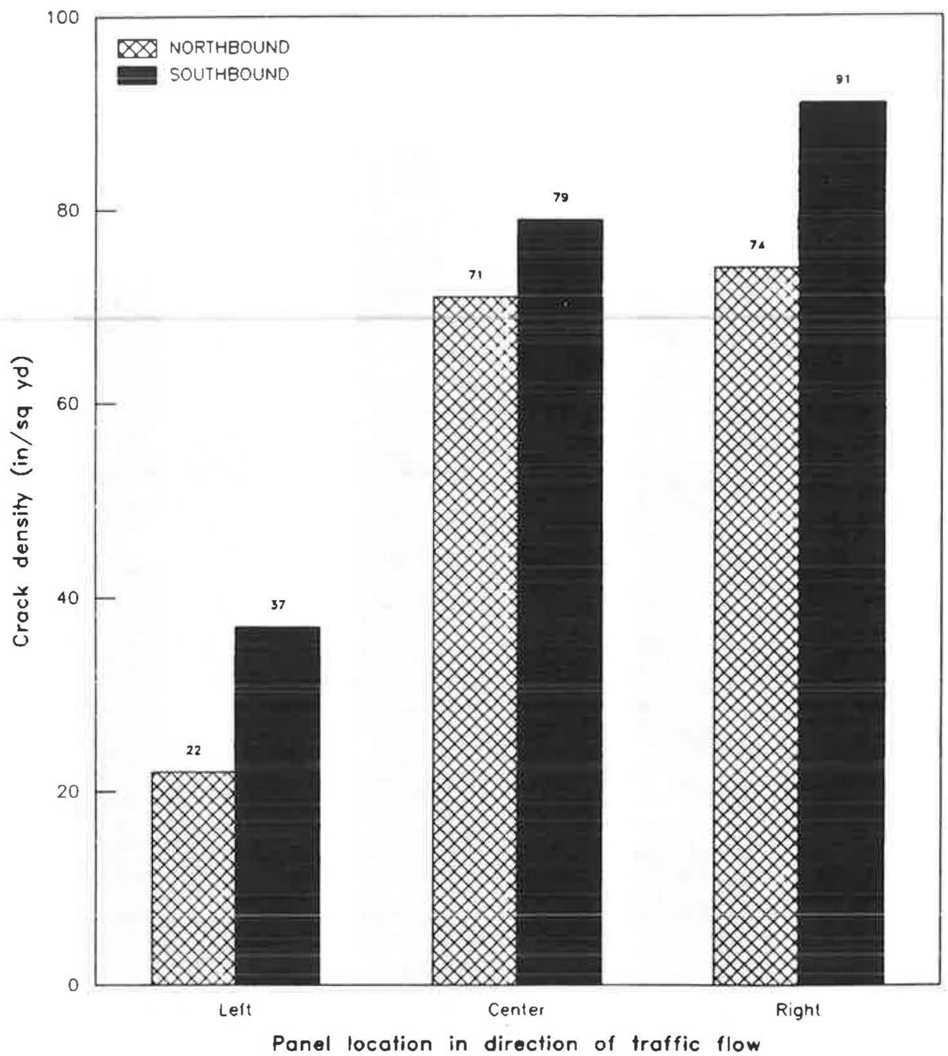

FIGURE 3 Transverse crack density in reinforced concrete decks subjected to different levels of traffic.

Observations of Transverse Cracking

The observed regions of the loaded bridge uniformly exhibited more transverse deck cracking than those of the unloaded bridge by an average of 71 percent in the left regions, 8 percent in the center regions, and 11 percent in the right regions.

For both the loaded and unloaded bridges, the center regions always exhibited more transverse cracking than the left regions. On average, there was 92 percent more transverse cracking in the center regions of the unloaded bridge, and 22 percent more transverse cracking in the center regions of the loaded bridge. The center regions also usually exhibited more transverse cracking than the right regions in both bridges, averaging 24 percent more for the unloaded bridge and 20 percent more for the loaded bridge. For the unloaded bridge, the right regions always exhibited more transverse cracking than the left regions, averaging 55 percent more cracking. For the loaded bridge, there was little difference between the right and left regions.

Observations of Longitudinal Cracking

The loaded bridge always exhibited more longitudinal cracking than the unloaded bridge in the left and right regions. The left regions exhibited an average of 68 percent more longitudinal cracking, and the right regions exhibited an average of 23 percent more longitudinal cracking. The center regions usually exhibited more longitudinal cracking in the loaded bridge, by an average of 11 percent.

For both the loaded and unloaded bridges, the center regions always exhibited more longitudinal cracking than the left regions. On average, there was 223 percent more longitudinal cracking for the unloaded bridge, and 114 percent more longitudinal cracking for the loaded bridge. The right regions always exhibited more longitudinal cracking than the left regions in the loaded bridge, averaging 13 percent more longitudinal cracking. For the unloaded bridge, the right regions usually were cracked more than the center regions by an average of 4 percent. The right regions always exhibited more longitudinal cracking than the left regions in both bridges, averaging 146 percent more cracking in the loaded bridge, and averaging 236 percent more cracking in the unloaded bridge.

Observations of Diagonal Cracking

The loaded bridge usually exhibited more diagonal cracking than the unloaded bridge in the left regions by an average of 47

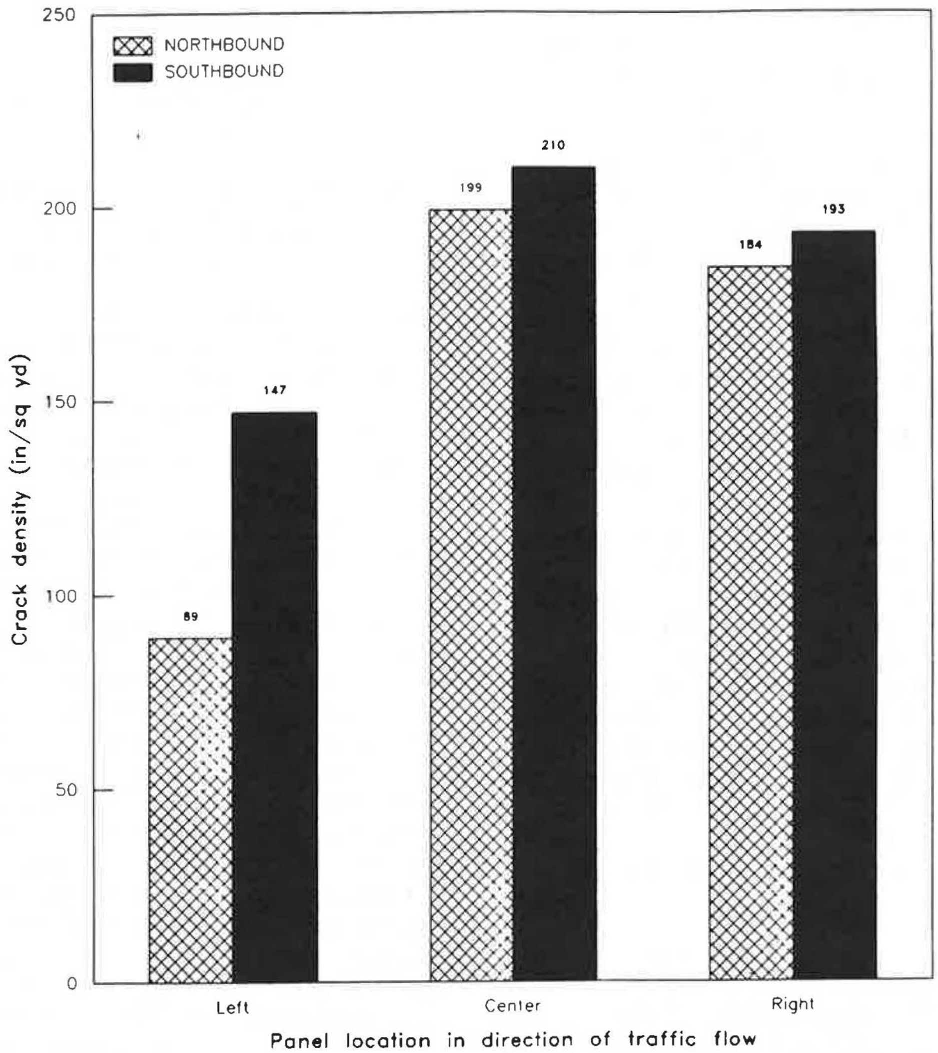

FIGURE 4 Diagonal crack density in reinforced concrete decks subjected to different levels of traffic.

percent. In the more heavily cracked center and right regions, however, an unexpected reversal of this trend was observed. The center regions were usually cracked less for the loaded bridge by an average of 17 percent, and the right regions always exhibited less diagonal cracking for the loaded bridge by an average of 55 percent.

For the unloaded bridge, the center regions always exhibited more cracking than the left regions, averaging 100 percent more cracking. For the loaded bridge, there was an average of 14 percent more diagonal cracking in the center regions than in the left regions. The center regions usually exhibited less diagonal cracking than the right regions in the unloaded bridge, by an average of 3 percent. For the loaded bridge, the center regions always exhibited more diagonal cracking than the right regions, by an average of 79 percent. The right regions always exhibited more diagonal cracking than the left regions in the unloaded bridge, averaging 107 percent more cracking. For the loaded bridge, the right regions usually exhibited less diagonal cracking than the left regions, averaging 36 percent less cracking.

Mechanisms of Overload-Related Damage to Bridge Decks

A finite element model of a steel-concrete composite bridge was applied by Wegmuller *(5)*. The proposed model was an orthotropic layered deck composite with a layered steel I–beam. The concrete, reinforcing steel, and I–beams were modeled as nonlinear materials. Model results were compared to experimental data to demonstrate the successful modeling of such structures in the postelastic regime. The model was limited to short-term, monotonically increasing loads. Tensile cracking of the concrete was treated simply as complete loss of stiffness after a cracking stress is reached. Gradual strain softening, the gradual reduction of normal stress with increasing strains in the postcracking regime as discussed by Bazant *(6)*, was not modeled.

Wegmuller's work *(5)* indicates that successful nonlinear analyses of bridges subject to overloads are possible. Further, he observes that up to about 20 percent overload, the response of the structure can be approximated by modeling the slab as an elastic, perfectly plastic material.

Kostem *(7)* discusses the initiation of deck damage resulting from overloading, drawing conclusions from a parametric investigation using the computer program BOVA *(8)*. Kostem's study is limited to simple-span reinforced or prestressed concrete girders and concrete deck bridges. Kostem considers several vehicles, calculating the gross vehicular weight for each that would induce cracks through the deck of each of several representative bridge designs. For instance, a 3S2 vehicle is predicted to cause deck cracking in the structures analyzed at gross vehicle weights from 91 to 132 kip, approximately 132 to 193 percent of the current legal gross vehicle weight for the 3S2 vehicle studied. The critical structures for the 3S2 vehicle were 100 ft simple spans using 7 and 8 AASHTO girders and a 7.5-in. deck. The reported results are limited to concrete decks on prestressed girders; however, the method should be applicable to structures of other types.

The earliest indication of overload damage to bridges is expected to be cracking in the slab. Batchelor et al. *(9)* present the results of a series of fatigue tests on 37 panels of five model slabs. Although not conclusive, the results indicate that the fatigue endurance limit punching loads for isotropically reinforced slabs having a reinforcement ratio of 0.2 percent and for slabs conventionally reinforced according to AASHTO specifications are 40 and 50 percent of the static punching failure loads, respectively. The endurance limit for the isotropic slab represents stresses caused by wheel loads approximately four times the design wheel load. Observed fatigue failure modes were predominantly punching shear, with flexural failures observed in some of the slabs having low reinforcement ratios. During the first few cycles of repeated loading, cracking was initiated. The cracks then widened and spread with subsequent repeated loading until a relatively stable period was reached during which little crack growth was observed. Empirical equations are presented that best fit the data. For conventional orthotropic reinforcement, the ratio of the fatigue strength P_f to the punching resistance P_s' is given in terms of the number of cycles to failure N by

$$P_f / P_s' = 1.0 - 0.102 \log N + 0.006(\log N)^2 \qquad (1)$$

The observation that fatigue failure of decks is unlikely neglects any interaction of fatigue cracking, which was observed to occur at much lower loads, and corrosion or other environmental effects. Certainly the presence of working cracks in the deck is detrimental to durability, and in many instances the serviceability of the deck is limited by corrosion of reinforcing steel. Cracking of reinforced concrete decks is expected under normal traffic levels.

Maeda et al. *(10)* suggest that slabs under normal service loadings are expected to develop longitudinal and transverse cracking spaced no closer than 1.3 ft, with crack widths of up to 0.008 in., at crack densities of up to 168 in. (linear) of crack per square yard, and exhibiting leaching of calcium hydroxide. More severe cracking, scaling, or spalling of concrete is indicative of abnormal distress, according to Maeda *(10)*. Figure 1 shows that total crack densities considerably greater than the quoted value of 168 in./yd^2 were recorded in the preliminary results of the field study reported here.

The problem of granting overload permit necessitates a more practical approach to the effect of overloads on bridges. White and Minor *(11)* discuss a computerized method of evaluation of bridge overloads that is typical of standard practice by the various state highway agencies. The method described is applied during the permitting process, and consists of a calculation of the maximum moment caused by the overweight vehicle under consideration on the bridge under consideration. An equivalent HS vehicle, some multiple of the HS–20 that causes the same moment, is identified. This vehicle is compared to the operating rating of the bridge, which has been previously catalogued according to AASHTO *(12)* and FHWA *(13)* procedures. The problem with this widely used method of analysis is identified by Kostem *(7)*.

> Any analysis scheme based on linear elastic behavior will . . . lead to conservative results since this approach will ignore the ever present material nonlinearities and the redistribution of stresses in the superstructure.

Other simplifying assumptions used in this and similar methods *(14, 15)* may be unconservative, however. The method reported by White does not check end shear because the only data available to the automated procedure is the rating based only on moment. Still this linear method of analysis, rating, and permitting offers the advantages of automation, economy, and practicality, and any bridges that compute as inadequate are identified for further individual analysis. More rigorous analytical methods can be applied to individual checks of these bridges.

Shanafelt and Horn *(16)* reported the results of a series of tests on a prestressed concrete girder. The purpose of the tests was to determine the effectiveness of various repair techniques applied to damaged girders. Damage representative of impact by overheight vehicles was considered in the series of 10 reported tests; however, Test 1 was a load test of an undamaged girder. Test 1 is helpful in understanding the progressive damage mechanisms that may be a factor in the durability of prestressed concrete girders. Further, this test helps quantify the damage to such girders due to overweight vehicles.

The prestressed concrete girders and composite reinforced-concrete deck design, as represented by this specimen, is among the most durable and overload-resistant designs available to designers; however, the potential for overload-induced progressive damage exists. The test girder was loaded in positive moment as a 60-ft, simply supported beam, by a midspan-concentrated load causing a bending moment equal to 75 percent of the calculated ultimate moment. This moment was equal to 195 percent of the HS–20 LL+I service load moment.

The resulting damage is described as being more severe than anticipated by the researchers. Eight major cracks were created extending from the bottom fiber of the girder to within a few inches of the bottom of the composite slab. In addition, minor cracks were observed between the major cracks. The eight major cracks were spaced 12 to 28 in. over a region approximately 130 in. long at midspan. Crack openings averaged 0.015 in. The stress at crack initiation was significantly less than the value $6\sqrt{f_c'}$. Effects of the conservatism afforded by the AASHTO lateral distribution factors would likely protect such a girder in service; however, if more realistic lateral distribution factors are used, cracking at this loading would be anticipated.

After the initial loading resulted in cracking of the girder, the test was repeated, and the behavior of the girder was different, the load-deflection curve having become nonlinear at a significantly lower load than that required to cause cracking in the initial test. This result could be interpreted as progressive cracking damage, except that a third load cycle resulted in a nearly identical load-deflection relationship. A more credible explanation than progressive damage is suggested by the following. During the first load cycle, the uncracked section behaves essentially linearly up to the point of crack initiation, at which point the moment of inertia begins to be reduced by the crack, which extends slowly, or at least in small increments, as the load is increased. The continual change in slope of the load-deflection curve indicates that the moment of inertia is continually changing, that is, the crack is continually extending, up to the maximum test load.

During unloading, there is a short elastic unloading portion, followed by a nonlinear unloading portion during which the cracks progressively close and a compression prestress redevelops. On completion of the first loading cycle, a residual strain is apparent. The second load cycle begins to exhibit a nonlinear relationship at a significantly lower load level, as a result of the cracks opening at zero normal stress rather than at the tensile strength of the concrete as in the first cycle. This means the cracks begin to open at a moment of only 75 percent, approximately, of that required to crack the girder in Load Cycle 1. On completion of the second load cycle, there is no additional crack elongation and no significant additional residual deformation because the second load cycle does not reach a greater moment than the first.

The response to the third load cycle is essentially identical to that of the second cycle. Ideally, this behavior might be expected to continue for repeated loadings to the same moment; however, other factors can alter this idealized behavior. The repeated opening and closing of the cracks in the structure can result in propping of the cracks by fines that are displaced from the fracture surfaces, resulting in further increases in residual deformation. Corrosion interacts with this process, accelerating and being accelerated by the repeated opening and closing of the cracks. If loads sufficiently large to initially crack the girder are experienced once and loads sufficiently large to open the cracks occur repeatedly, these mechanisms can serve to cause progressive damage to prestressed concrete bridges.

Prestressed concrete girder bridges are to some extent susceptible to progressive damage mechanisms dependent on the level of the live loads. Such structures are believed to be more resistant to such damage than other structures with a larger ratio of live load to dead load. In a field study in progress, observations of heavily traveled bridges are being compared with observations of less heavily traveled, but otherwise identical, bridges in Texas. Preliminary findings confirm that the decks of some heavily traveled structures clearly exhibit more cracking than the decks of similar but less heavily traveled structures. Also, there is qualitatively a higher level of deck damage in decks supported by steel stringers than in prestressed girder decks. Both of these findings are as expected, however quantified documentation of such progressive damage mechanisms is essential to economic analysis of the effects of changes in truck regulations. Efforts to quantify these observations are in progress.

Fracture Mechanics of Concrete

Although not yet in wide use by the bridge engineering community, numerical fracture mechanics methods are being applied to plain, reinforced, and prestressed concrete structures in other fields, and considerable interest by researchers in this area is evident in the literature. Two distinct methods of modeling fracture of concrete structures are developing. Modeling the crack in a continuum by the use of finite elements having free surfaces along element boundaries where the crack is open, the first approach to have been explored, is perhaps the more straightforward approach. An overview of this method is provided by Ingraffea *(17)*. This method suffers the inherent difficulties associated with the necessity of redefining the mesh as the crack propagates, resulting in significant computational problems, and with the inherent limitation that cracking may occur only on predefined element boundaries. The second, more widely accepted, method, originally proposed by Rashid *(18)*, models the crack by a continuous smeared crack band. This method eliminates the necessity of redefining the mesh as the crack propagates, although the stiffness of the elements involved in the fracture process zone obviously changes with crack propagation. The smeared crack numerical model is in fact believed to be a better physical model of fracture in concrete than the discrete crack model, because in concrete there is a zone of some finite thickness surrounding each crack where physical damage to some extent reduces the stiffness of the continuum.

The applicability of the two methods depends on the scale of the problem, because the fracture of massive structures such as dams and the fracture of smaller structures such as bridge decks in flexure, require significantly different modeling techniques. In the smeared crack band approach, fracture is modeled by a reduction in modulus at integration points where cracking criteria are satisfied. The fracture is therefore manifested in the model by a zone or band of anisotropic material with reduced modulus in the direction normal to the modeled fracture surface.

The constitutive models in use for the concrete vary. A failure of tensile strength and linear strain-softening rule is frequently applied. Its variations include the concepts of crack closing, shear capacity through aggregate interlock, intersecting of cracks, and creep behavior being added under some circumstances. Failure modes of the concrete in compression and biaxial loading are considered in some applications. Reinforcing steel is modeled with associated nonlinear behavior that includes application of yield, rupture, and debonding models.

CONCLUSIONS

1. Based on several observations, cracking of concrete members may occur at tensile stresses below the expected value of $6\sqrt{f_c'}$ or $7.5\sqrt{f_c'}$. Whether residual stresses due to creep or shrinkage are the explanation for this observation is not known. In addition to Shanafelt and Horn *(16)*, who reported cracking at stresses of $2.7\sqrt{f_c'}$ to $4.5\sqrt{f_c'}$, Bonilla et al. *(19)* observed significant cracking in a one-third scale reinforced-concrete deck subjected to negative moments at stresses as low as $3\sqrt{f_c'}$.

2. Concrete bridge decks respond to overloads by exhibiting increased densities of longitudinal and transverse cracking, although normal traffic levels are thought to result in cracking or reinforced concrete decks in some circumstances *(10)*. The increased cracking leads to accelerated corrosive attack and spalling or scaling.

3. Mechanisms exist for progressive overload-induced damage to reinforced-concrete decks and for interaction between mechanical and chemical effects. Models for studying

the rate of deterioration due to these effects and the related economic impact have not yet been developed.

4. Bridge decks supported by steel girders are more susceptible to progressive overload-induced damage mechanisms than are decks on prestressed concrete girders. Preliminary results of field studies in progress indicate that although there is observable damage to decks of both types attributable to heavy-truck traffic, there is a measurably greater extent of damage to decks supported by steel girders when subjected to heavy-truck traffic. A complete quantified measure of the effect of heavy-truck traffic is not yet available, however.

5. Mechanisms exist for progressive damage of cracked prestressed-concrete girders so long as the girder is subject to moments large enough to reopen the initial cracks. Test data *(16)* indicate that the moment necessary to reopen the cracks is significantly lower than the moment causing initial cracking, perhaps only 75 percent of the moment causing initial cracking.

RECOMMENDED FUTURE RESEARCH

Application of rapidly developing technology in the field of fracture of concrete to the problem of progressive mechanical deterioration of concrete bridge decks is a logical extension of the present state of the art in the field of bridge engineering. The establishment of a relationship between mechanical damage (such as that due to wheel loading) and corrosion-induced deterioration is a second problem that should be addressed. Although the solution to the first problem is achievable with current nonlinear finite element technology, the solution to the second problem is more difficult. Further research such as that reported by Cady and Weyers *(4)*, as well as field studies identifying and quantifying the relationships between progressive deterioration and mechanical and environmental loading, are essential for solution of the second problem.

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