Survey of Field and Laboratory Tests on Bridge Systems

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Field and laboratory tests were conducted on the behavior of several short-span bridge systems: orthotropic deck; composite box girder; composite U-beam superstructure; precast, prestressed concrete deck planks; and concrete box girder. The results are critically reviewed from the viewpoint of fabrication, erection, performance, and first cost. Because the test results consistently proved that the current AASHO transverse load distribution factors are very conservative, tentative distribution factors are suggested for the interior and exterior girders of bridges with two or more lanes.

Systems construction may be defined as a schematic technique for a well-conceived, mechanized operation that uses optimal sizes of mass-produced structural components and requires minimal field labor (i.e., makes efficient and maximum use of available human and material resources). It may be described as a concept that signifies the thinking and operation process of which the structure is a part. The process consists of an interrelation of function, design, production, erection, and economics in which flexibility of arrangement is one of the most important criteria. Terms such as prefabrication, industrialized building, and modular construction are also used to describe these aspects.

Because the systems construction of bridges is different from conventional construction methods, extensive research and development and large-scale field testing are required to (a) develop speedy, economical, foolproof methods of construction by assembling various components on a "streamline" basis; and (b) ensure that the response of the final assemblage should be at least as good as that of a monolithically built bridge structure. In addition, adequate constraints based on theoretical criteria and previous experience, which are contained in national building codes, are needed to ensure safety and satisfactory serviceability of a new bridge system.

The most important objective of this paper is to examine the behavior of several superstructural bridge systems tested in the field and the laboratory. In addition, basic limitations on and possible modifications of bridge-system construction are discussed in relation to fabrication, transportation, erection, performance, and first cost. The discussion is limited to short-span bridge superstructures [3 to 23 m (10 to 75 ft)] that lend themselves to a modular type of construction. Although many excellent sources exist on the general subject of bridge testing (1, 6, 7, 13), a literature review is not attempted here because most of these publications do not deal with systems concepts of bridges.

TESTS

Tests were performed on the following modular bridge units: (a) orthotropic deck systems; (b) composite box-girder systems; (c) composite U-beam superstructure; (d) precast, prestressed concrete deck planks; and (e) concrete box girders. The tests and their results are discussed below.

Orthotropic Deck Systems

In 1964 Bethlehem Steel Corporation built an experimental bridge at its Sparrows Point, Maryland, site (4, 24). The system used a basic modular unit with the deck assembly and main girder. Dimensions of these units varied from 6 to 24.4 m (20 to 80 ft), in increments of 3 m (10 ft). Figure 1 shows details of the system. A similar system with two continuous spans (Figure 2) was built by the Michigan Department of State Highways as a part of Crietz Road over I-496 (18).

In this research the static behavior of a single unit of the Bethlehem Steel bridge was studied by placing the unit in a test rig in the laboratory. The entire width of the deck assembly acted uniformly in resisting overall bending in the unit, and the field test results showed how the wheel-load distribution factor varied with the width of the bridge. For example, the distribution factor was reduced from 97 to 68 percent of the American Association of State Highway Officials (AASHO) code value (2) by doubling the width of a single deck unit. In addition, test results revealed that the number, the type, and the positioning of vehicles had an influence on the distribution factors. In the field tests critical stresses were observed at the bottom of a longitudinal rib that was passing over the floor beam. A comparison of computed and measured strains and deflections of various components such as main girders, ribs, floor beams, and deck of the Crietz Road bridge indicated that the design assumptions are very conservative and that all the measured values are well below the calculated ones. For example, the maximum calculated transverse strain and relative deflection in the deck, based on the uniform wheel-load distribution over a rectangular area, were about 700 μm/m (μin/in) and 1.52 mm (0.06 in) respectively, whereas in the test results they were of the order of 100 μm/m (μin/in) and 0.25 mm (0.01 in).

It is interesting to note, from tests on the Bethlehem Steel bridge, that dual-tire stresses are about 14 percent less than the theoretical values computed from the design manual of the American Institute of Steel Construction (3). However, single-tire stresses were about 8 percent higher than theory. A similar phenomenon was observed in the case of the Crietz Road bridge. The impact factor for the girders of the Bethlehem Steel system was found to be about 0.1, whereas the AASHO code (2) specifies a value of 0.3. However, the impact factor for the deck and the floor beams directly under the wheel agreed with the AASHO specifications. A similar trend was noted in the Crietz Road bridge: Dynamic runs at speeds of about 30.2 km/h (20 mph) gave peak strains and deflection values approximately 10 to 15 percent higher than the static readings.

Although the field performance of asphalt surfacing with epoxy membrane in the Bethlehem Steel system was reported to be satisfactory over a 3-year period, recent reports, including the Crietz Road bridge records (18), have proved that the wearing surfaces in general did not function satisfactorily. However, a comprehensive study by a Pennsylvania firm on the performance aspects of wearing surfaces revealed that rubberized asphalt surfacing with epoxy membrane performed well during field and laboratory testing. The product is supplied only by Adhesive Engineering Company of San Carlos, California, and the complete installation costs may vary from \$30 to \$33/m^2 (\$36 to \$40/yd^2). Additional information on paving practices for wearing surfaces on orthotropic steel bridge decks can be found elsewhere (18).

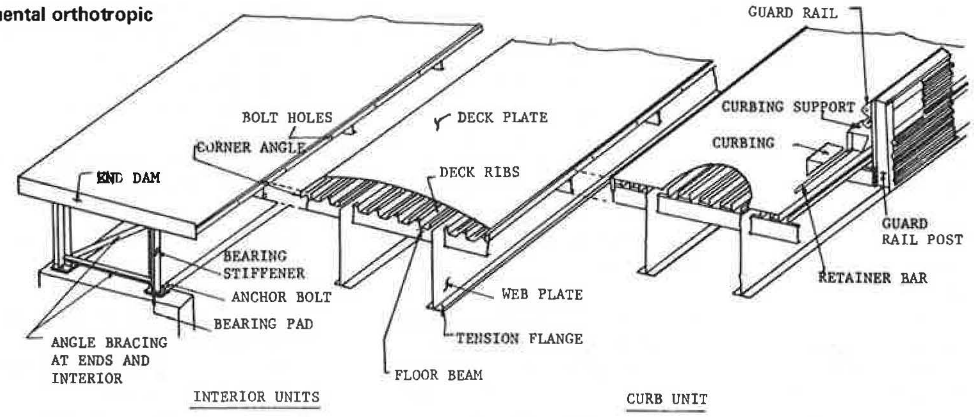


Figure 1. Bethlehem Steel experimental orthotropic system.

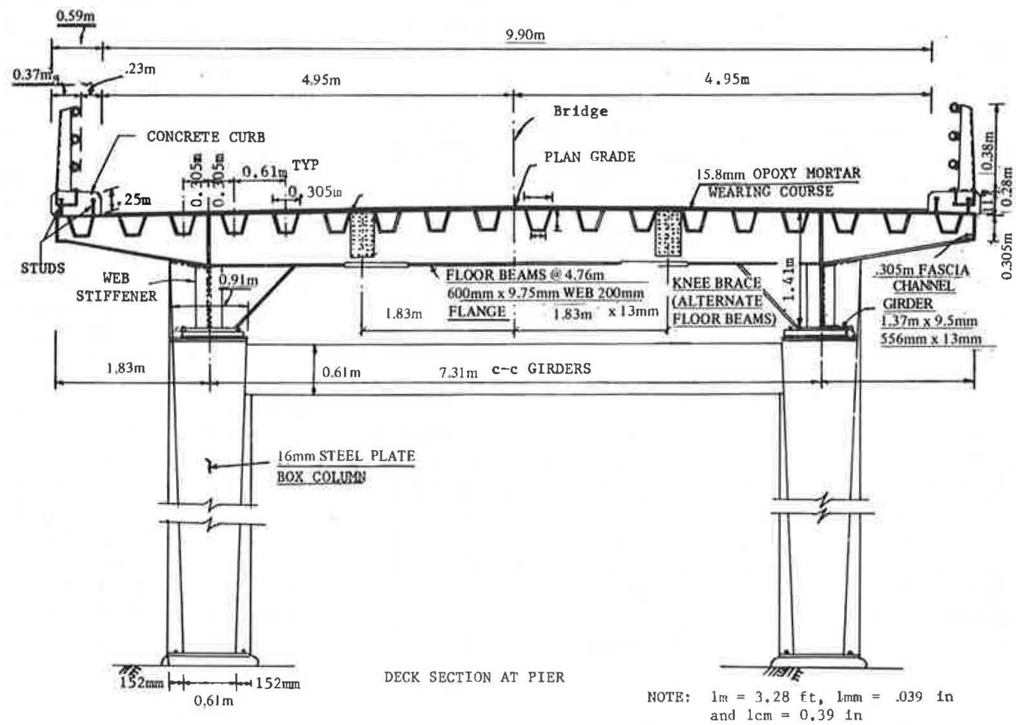


Figure 2. Deck section of continuous orthotropic steel plate deck bridge.

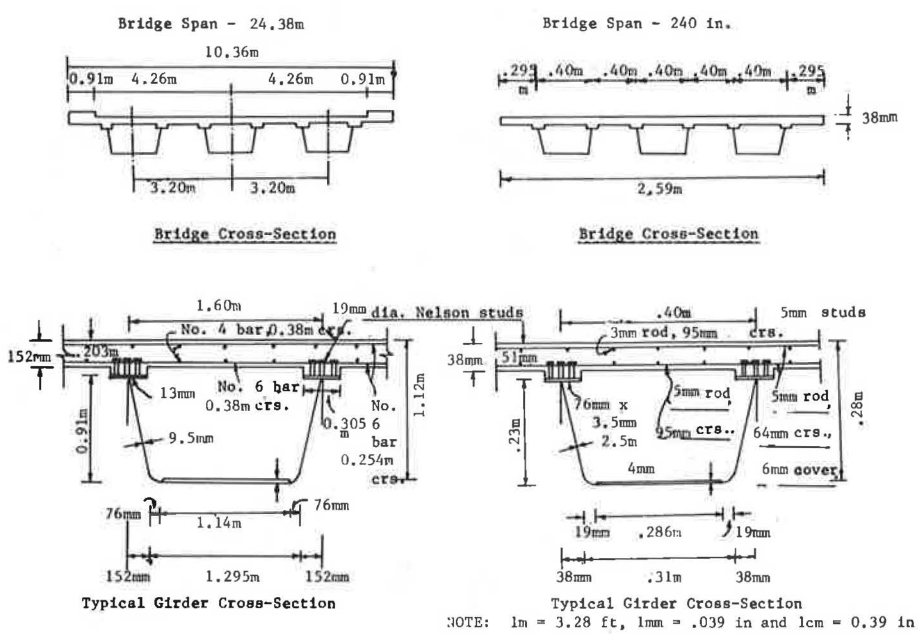


Figure 3. Typical composite box-girder bridge with prototype and model dimensions.

Composite Box-Girder System

Figure 3 shows a general view of a bridge with a composite box-girder system as well as the details of a two-lane, 24.4-m (80-ft) span prototype system with three box units and a corresponding quarter-scale model. The bridge was designed (12) for an HS20-44 loading with a distribution factor of S/6.5, instead of S/5.5 as suggested by AASHO (2) (S is defined as the girder spacing in feet). (The AASHO values for load distribution are empirically derived in customary units; therefore, no SI equivalents are given.)

The model was tested under concentrated loads and for simulated truck loads. Transverse influence lines for midspan deflection and the average strain (at the bottom plate) of each girder were constructed from these load tests. When test results were compared against the folded plate theory, based on Fourier series expansion, there was good agreement. According to Johnston and Mattock (12), the calculated transverse distribution of loads was in close agreement with the measured distribution. They also found that the measured maximum loads carried by exterior and interior girders were equivalent to the load distribution factors of S/6.91 and S/6.48 respectively, which agreed closely with the assumed distribution factor of S/6.5. Finally, Johnston and Mattock concluded that the transverse load distribution factors derived from the AASHO code were very conservative and that the most economical composite box-girder arrangement was one box-girder system for each lane of traffic.

U-Beam Superstructure System

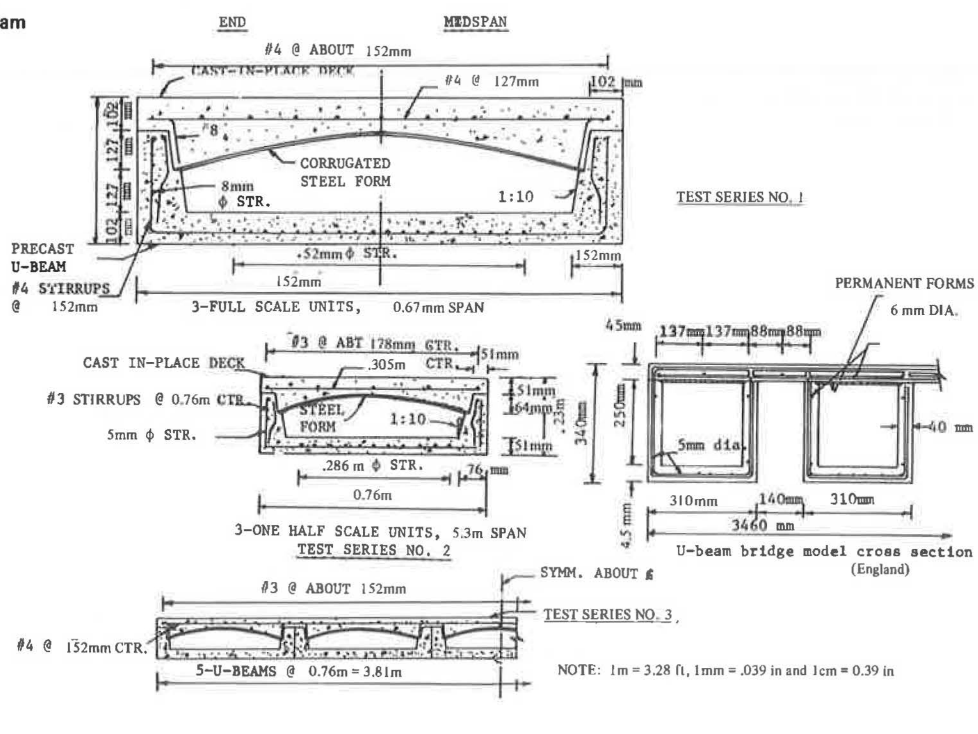
A composite U-beam bridge superstructure was developed in the United Kingdom as well as in the United States. Typical cross-sectional details of the test specimens are shown in Figure 4. The Missouri system developed in the United States (20, 21) was tested with single and multiunit beams for single concentrated load as well as simulated AASHO HS20-44 truck loads. Field tests of the multiunit British system (8, 14) were conducted for eccentrically loaded HB vehicles. Single-unit test results—i.e., continuous and linear strain distributions through the depth of the test specimens at various load levels and deflections based on an idealized composite action of the Missouri system (19)—indicated complete composite action between the U-beam and the cast-in-place deck.

The Guyon-Massonnet load distribution theory (16) was used to predict lateral distribution of moments and deflections of the multiunit system. The measured and calculated results are given by Salmons and Mokhtari (21). As was observed in earlier investigations, the distribution characteristics were found to change with the position of the applied load. The maximum wheel-load distributions from the tests were compared with the distribution factors from the AASHO code, and it was found that the AASHO code results were very conservative. Since the multiunit system was tested to failure, Salmons and Mokhtari (21) have observed ductile behavior of the whole unit in which the ultimate deflection was more than 20 times the elastic deflection.

As a result of extensive experimental and theoretical investigation of the U-beam system, Cusens and Rounds (10) recommended for design purposes the finite strip method, which, when it was compared with the load distribution method or the orthotropic theory, was found to be accurate and economical. Cusens and Rounds observed initial cracking and final collapse at 1.68 and 3.73 times the design loads respectively, and the crack pattern indicated effective distribution of the load between the girders. In addition, transverse load distribution through the top slab, taken from the strain readings between the slab and the beam webs, appeared to be adequate. However, the authors recommend further experimental work to study the effectiveness of various types of beam-slab joints and their influence on the overall behavior of bridges.

In spite of a detailed cost analysis by Salmons and Kagay (20) that found the U-beam system to be economical for 10 to 24.4-m (30 to 80-ft) span ranges, recent correspondence with the Missouri State Highway Commission revealed that, because this system is not economically competitive with some of the existing ones, its use has been discontinued.

Figure 4. Cross-sectional details of U-beam models.



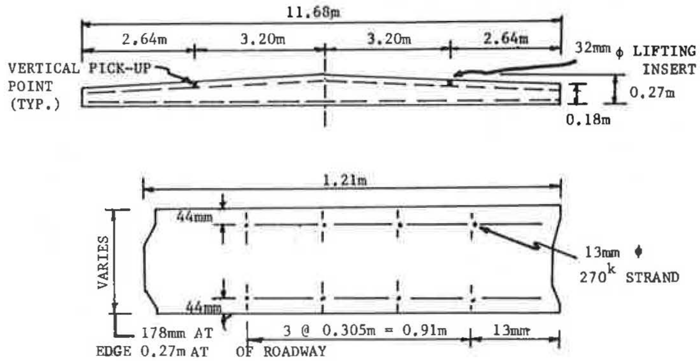


Figure 5. Typical elevation and cross section of concrete bridge slabs.

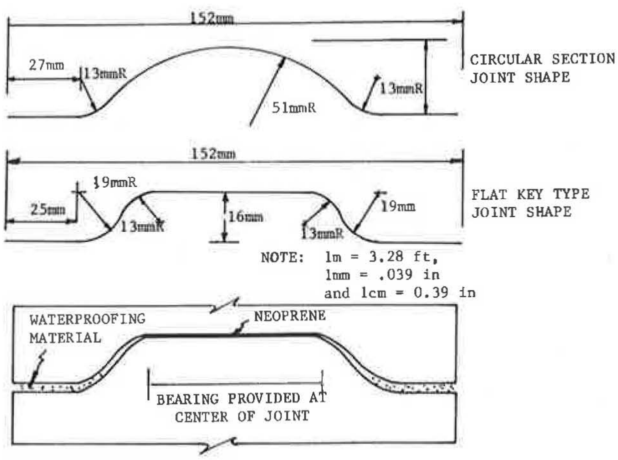


Figure 6. Joint shapes.

Precast, Prestressed Concrete Slab Units

A system of precast, prestressed concrete slab units placed on stringers has been found to be feasible and easy to erect compared with conventional methods. Two such systems were built in 1970 on Ind-37 and Ind-140 (14); the cross-sectional details for the slabs are shown in Figure 5. Features common to both bridges included the following:

1. Precast slabs as long as the road width and 2.4-m (8-ft) sections were placed on longitudinal stringers.
2. A tongue and groove shear keyway ran the entire width of the slab to join individual units. Joints were sealed by bonding a 0.16 × 12.1-cm (0.06 × 4.75-in) neoprene strip that ran along the slab joint.
3. Slab units were anchored to the girders by 115-RE-F railroad clips and 1.91-cm-diameter (0.75-in) bolts.
4. After slab units were placed on stringers, they were posttensioned along the bridge length and anchor bolts were tightened at a specified torque level.

Cracks running transverse to the slabs appeared near the joint during and after posttensioning operations as a result of irregularities in the slab crown section that caused poor fit between adjacent units and possibly because of the application of excessive prestressing force. Although the Indiana systems seem to have performed well over a 4 or 5-year period, the following maintenance problems were noted:

1. Leaking of joints—Of two kinds of joint sealants, Dupont's Imron sealant proved to be superior to the liquid polyurethane sealant. It was advised that the manufacturer's directions be carefully followed during the application of the joint sealant.
2. Loosening of bolts—Bolts clamping slabs to beams worked loose. They had originally been tightened to 67.75-J (50-ft-lb) torque; they were retightened to a torque of 169.37 J (125 ft-lb).

To alleviate the spalling of concrete near the joint, three different joint configurations, shown in Figure 6, were tested in the laboratory under repeated loads; the joint type recommended is shown at the bottom of the figure. The bearing was limited to the flat center portion of the joint, which eliminated the spalling of corners. Joint sealant of sufficient thickness was provided in the gap above and below the flat center part of the joint to prevent leakage of water.

Concrete Box-Girder System

A large number of reinforced or prestressed concrete box-girder bridges were built in California and Pennsylvania during the past decade (9) because they had proved to be durable and economical, particularly in spans of 18.3 to 30.5 m (60 to 100 ft). Since the construction of a number of these systems in the state of California, an extensive investigation has been conducted on the structural behavior of the two-span, reinforced-concrete, box-girder bridge model (5, 23). Details of the model are shown in Figure 7.

Three different analytical methods were used to mathematically model the multicell box-girder system. The modeling approach is based on idealizing the system into one- and two-dimensional elements and solving for the displacements and forces within the elastic range by finite-element techniques (15, 17). Scordelis and others (23) concluded that the transverse distribution of the total moment at a section, in terms of percentage to each girder, was predicted accurately at working stress levels for single point loads and uniform loads across the width of the bridge. Although premature failure was observed in the undiaphragmed span, both undiaphragmed and diaphragmed spans exhibited excellent load distribution characteristics under ultimate load conditions with a factor of safety of four against live load overloads. The magnitude and distribution of live load deflections were also predicted accurately when theoretical values based on uncracked sections were multiplied by a factor of about 1.5 to account for cracking (15). Interestingly, the total reaction and the total moment at any section were found to be independent of the transverse position of the point loads.

LOAD DISTRIBUTION FACTORS

The AASHO code (2) recognized through experimental results that the transverse load distribution of a stan-

Figure 7. Dimensions and cross section of box-girder bridge model.

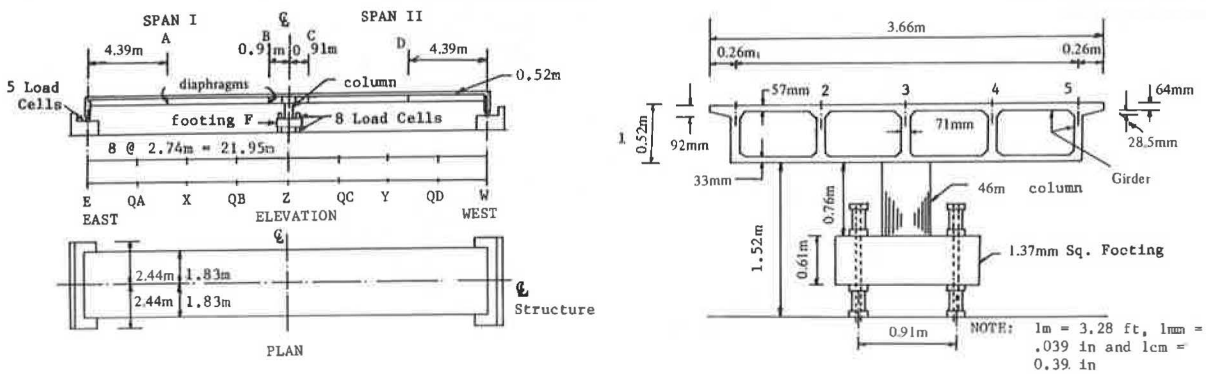
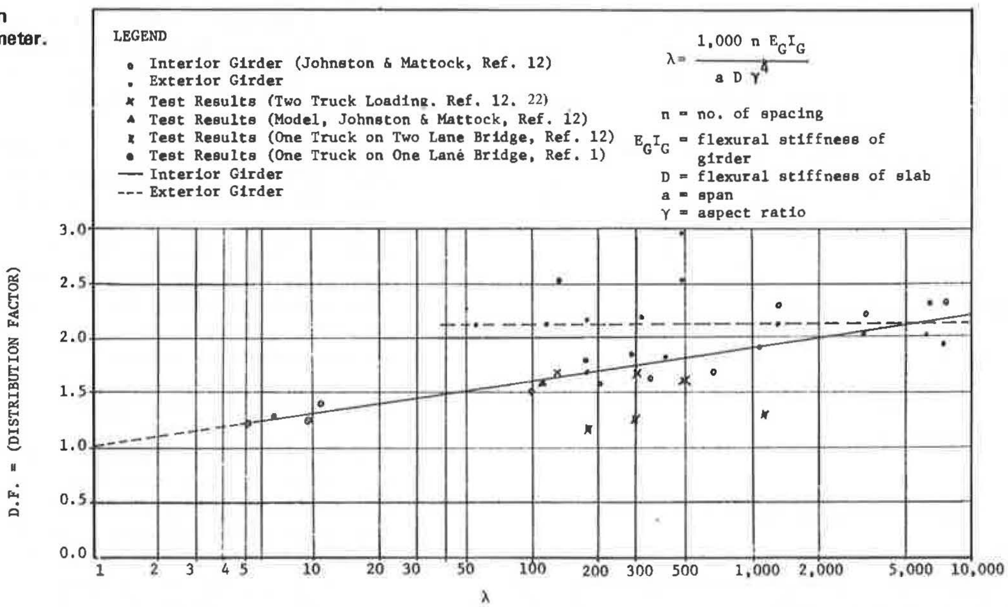


Figure 8. Load distribution factor versus flexural parameter.



dard slab-stringer system is different from that of a spread box system, a composite box-girder system, or even the multibeam superstructure system. However, a comparison of experimental distribution factors with those from the orthotropic theory and the harmonic analysis revealed no consistent agreement with either one of the analytical methods (22). This is substantiated by the fact that the experimental and AASHO load distribution factors of a two-lane bridge are 2.54 and 1.76 respectively (22). In addition, test results for a three-span, two-lane bridge (13) yielded a maximum distribution factor of 0.389, whereas the AASHO distribution factor for a corresponding two-lane bridge was 1.667.

Variation of the transverse load distribution factor with a nondimensional flexural parameter (λ), based on existing experimental information, is plotted in Figure 8 (25). The distribution factor varies exponentially with λ for interior girders, and the same variation is a constant of 2.13 for exterior girders. Because the distribution in Figure 8 does not represent a wide range of systems or types of construction, a simple mathematical relation can be suggested only after conducting additional tests. It should be noted that, for composite steel box-girder bridges, the value of I_G in the flexural parameter (λ) is computed by idealizing the box as two wide-flange sections in which half of the bottom-flange effectiveness goes to each wide-flange unit.

CONCLUSIONS

Test results for orthotropic bridge systems revealed that dual-tire stresses are less severe than those caused by single tires and that the impact factor for girders was about one-third of the suggested AASHO value although the local effects on the deck and the floor beams reached about 30 percent of the equivalent static load. Precast, prestressed concrete slab units appeared to lend themselves to renovation of commonly built deck-stringer bridges. However, a special joint with a flat center portion between the precast units was found to be essential to preventing spalling during posttensioning operations.

Laboratory tests of composite and all-concrete box-girder systems indicated a better transverse load distribution relative to the other systems discussed. In addition, the box system, as observed in service in states like Pennsylvania and California, was found to be durable and economical. Although the concrete U-beam girder system with cast-in-place deck provided complete composite action and behaved as a box system, its use has been discontinued for economic reasons. The experimental results yielded better transverse load distributions for deck length-to-width ratios of the order of 1:2. The transverse distribution was also found to be a function of the number, the type, and the positioning of loads. Finally, exponential variation of the transverse load distribution factor with the flexural parameter (λ) is tentatively suggested for interior girders, and a constant distribution factor of 2.13 is recommended for exterior girders.

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Highway Bridge Vibration Studies

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The results of acceleration studies of highway girder bridges are presented. Deflection limitations and maximum span-depth ratios used in present bridge design codes do not necessarily ensure the comfort of bridge users. Vertical accelerations have been shown to be significant in producing adverse psychological effects on pedestrians and occupants of stopped vehicles. The effects on bridge accelerations of major bridge-vehicle parameters, including the properties of the bridge and the vehicle as well as the initial conditions of the roadway, were investigated analytically and compared to criteria for human response. Numerical solutions are obtained from a theory in which the bridge is idealized as a plate continuous over flexible beams for simple-span bridges and as a continuous beam with concentrated point masses for two- and three-span bridges. The vehicle is idealized as a sprung mass system. The results indicate that, for simple-span bridges, accelerations that might psychologically disturb a pedestrian are primarily influenced by bridge-span length, vehicle weight and speed, and especially roadway roughness. Less significant factors are