FIELD TESTS OF A STEEL-COMPOSITE BOX-GIRDER BRIDGE

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Field tests were performed to assess the behavior of a steel box-girder bridge with a composite-concrete roadway deck slab. The structure was instrumented at a mid-span cross section and at a section near the support of 1 of the simple approach spans to permit studies of bending and shear strain distributions. Live load was applied, and theoretical analyses were performed with a finite element program. Live load membrane strains can be accurately predicted by the program, but fiber strains, especially in the transverse direction, are greatly influenced by plate warpage and initial local dead load deflections. Observed dead load strains exhibit significantly greater shear lag than that predicted by the program. Live load and ultimate dead load deflections are closely approximated by the theoretical analysis. The program overestimated the torsional stiffness of both the bare steel girder and the composite section. Success of the finite element program in assessing structural behavior of the steel box girder coupled with previously demonstrated deficiencies in methods of design employing distribution factors provides a convincing argument for using such a program in future designs of this type of structure.

•TESTS were performed by the California Department of Transportation on a steel box-girder bridge with roadway slab of composite concrete to study structural behavior and assess the validity of a computerized finite element analysis for predicting strains and deflections of the structure (1).

FIELD TESTING

Description of Prototype

The prototype is a twin structure that is 4,050 ft (1234 m) long and crosses the Sacramento River at Bryte Bend on I-880. The test span is the twentieth simple span of the right structure. It is a 146.5-ft (44.7-m) span with a uniform cross section between supports as shown in Figure 1.

The structure was designed on the basis of gross moment of inertia. Secondary effects of torsion were considered. It was assumed that 4.5 lanes were loaded with AASHTO design loads. These loads were distributed transversely, and we assumed that a simple beam distribution existed between webs.

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*Mr. Castleton was with the Division of Structures, California Department of Transportation, when this research was performed.

Instrumentation

The structure was instrumented at a primary plane near mid-span as shown in Figure 2. A secondary plane near a support was instrumented with a small number of SR-4 rosette transducers on each of the webs.

Housed in an instrumentation trailer beneath the span was the major item of electronic readout hardware: a computer-controlled, high-speed, multiplexing, data-acquisition system capable of scanning up to 100 transducer circuits at 5,000 readings/s. Thus when a test vehicle traversed the span at crawl speeds of 3 ft (1 m)/s, each of 100 circuits in the system was scanned for each ¾ in. (19 mm) of vehicle travel.

Test Vehicles

Two vehicles were employed in the research. One was the FHWA 1959 Autocar, the dimensions of which approximate those of the standard AASHTO HS20 truck. The other was a 7-axle, double-goose-neck, low-bed hauler. The simulated AASHTO vehicle was ballasted with reinforcing bar ingots to a gross load of 75,000 lb (34 000 kg); the 7-axle vehicle was ballasted to 220,000 lb (100 000 kg).

The simulated AASHTO vehicle could produce a maximum bending moment of 2,300 kip·ft (3100 kN·m), which is about 20 percent of the 11,000 kip·ft (14 900 kN·m) design moment for live load plus impact. The heavier vehicle produced a total moment of 5,900 kip·ft (8000 kN·m). A profile drawing of the heavier vehicle is shown in Figure 3. This figure also shows measured 4-wheel axle reactions. Longitudinal runs were made in 16 transverse locations on the roadway at approximately 2-ft (0.6-m) intervals.

Data Acquisition and Processing

Road switches placed on the deck at the eighth points of the span provided a means of accurately locating each axle at any time. Of the 370,000 items of data recorded on magnetic tape during each crossing, 10 readings/transducer circuit were read and averaged by a data processing program. Transverse distribution of transverse and longitudinal strains was plotted for various transverse vehicle positions.

THEORETICAL ANALYSES

Finite Element Program

Analyses of the structure by means of a computerized finite element code (FINPLA) were employed in comprehensive studies of observed strain patterns (2, 3). This program is especially fitted to this type of structure because most of the structural elements including the steel plates, steel flanges, concrete slab, transverse and longitudinal ribs (stiffeners), cross-frame members, and diaphragms can be mathematically modeled. The program employs a rectangular finite element with 24 degrees of freedom; 6 are at each node. Membrane actions are described by 2 in-plane displacements and a rotation about a normal to the plane of the element at each node. Slab actions are described by 2 rotations about in-plane axes and a displacement normal to the plane of the plate at each node.

Structure Separation

The tested span was separated by 34 longitudinal blocks of 33 elements, each of which was bounded by transverse and longitudinal stiffeners. The steel flanges of the box were idealized as longitudinal stiffeners because they were some distance below the

Figure 1. Cross section of prototype.

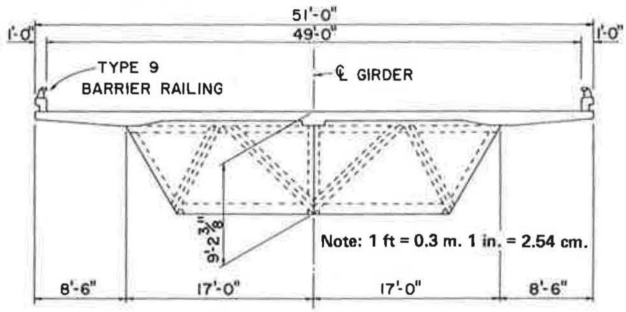


Figure 2. Prototype half cross section showing instrumentation at mid-span.

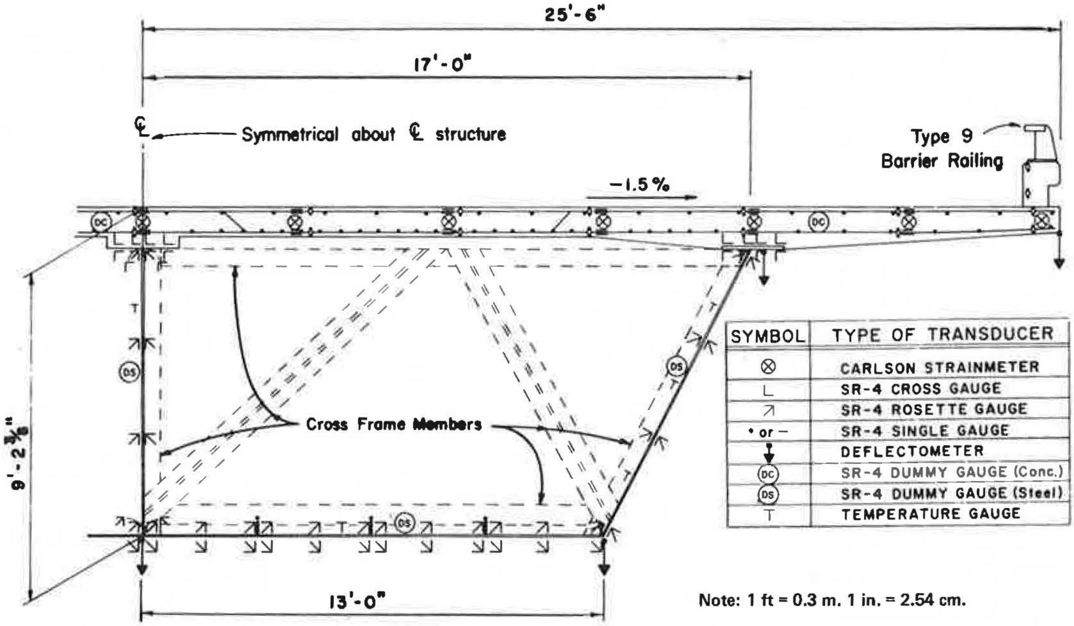
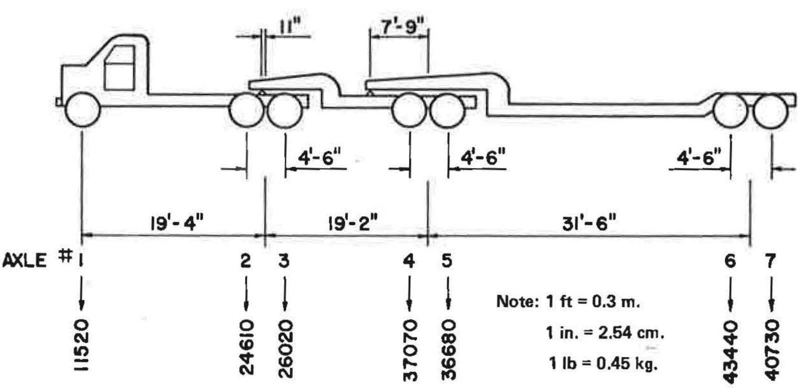


Figure 3. Profile and axle reactions for 7-axle test vehicle.



nodal points marking the intersections of webs and the mid-plane of the concrete roadway slab. A second possible idealization existed in which flanges were moved up to the intersections and were treated as separate plate elements with suitable modifications in cross-sectional areas. This idealization also was applied successfully, but it will not be discussed in this paper.

Longitudinal Live Load Strains

Membrane strains for the various concrete and steel plate elements as computed by the FINPLA code with the 7-axle test vehicle in its leftmost transverse position on the roadway are shown in Figure 4. Theoretical strains on opposite faces of the plates differed very little from each other, which indicates the small influence of both general and local bending except for relatively thick concrete roadway slab.

Longitudinal fiber strains measured by the SR-4 gauges on the steel plate surfaces or extrapolated from SR-4 gauges on reinforcing bars and strain meters embedded in the concrete slab are also shown in Figure 4 for the same transverse position of the test vehicle.

Marked discrepancies between measured and theoretical strains were immediately evident. Primarily, they took the form of differences in magnitudes of strains on opposite sides of the plates. However, plotted mean values of measured fiber strains (mid-plane strains) exhibited marked agreement with theoretical strains.

Influence of Plate Warpage

Differences between theoretical and measured extreme fiber strains can be attributed to the nonplanar configuration of the steel plates, which is caused by plate warpage and dead load deflections. Such warpage is especially pronounced in webs because transverse stiffeners are welded to their inner faces. The bottom plate also exhibited warpage between the longitudinal stiffeners. Of greater consequence was the dead load deflection that occurred between cross frames in the bottom plate. These frames are spaced at 22-ft (6.7-m) intervals and provide a measure of support to the plate, which deflects between these support points under the dead load of plate and stiffeners.

Warped plates, under the influence of tensile stresses produced by live load application, tend to straighten and produce pronounced local bending strains, which are superposed on the membrane strains. The bottom plate tends to assume the configuration of a number of straight chords and manifests strains commensurate with upward bending midway between these supports. The result is that tensile strains measured on the upper surface of the bottom plate exceeded those on the lower surface, and the tensile strains measured by the single-element SR-4 gauges atop the longitudinal stiffeners were much greater than those on the plate even though they were appreciably closer to the neutral axis of the box section (Figure 4). Two strain gauges mounted on the stiffener and the plate near the cross frames exhibited the opposite behavior. The gauge atop the stiffener produced appreciably lower strains than those on the plate, which might be expected in the upward-curving supported plate at these points.

Transverse Live Load Strains

Theoretical and experimental transverse strains for the leftmost transverse position of the test vehicle are compared in Figure 5. Strains in the concrete roadway slab showed good agreement for all transverse positions except at the intersection of the center web and the slab when no attempt was made in the theoretical treatment to simulate slab thickening at the fillets. This decreased the measured bending strains below those that might be expected in a slab of uniform thickness.

Superficial comparisons of the theoretical and measured fiber strains in the steel plates immediately suggest many anomalies. However, mean values, which represent

Figure 4. Comparison of theoretical and measured longitudinal strains for live load.

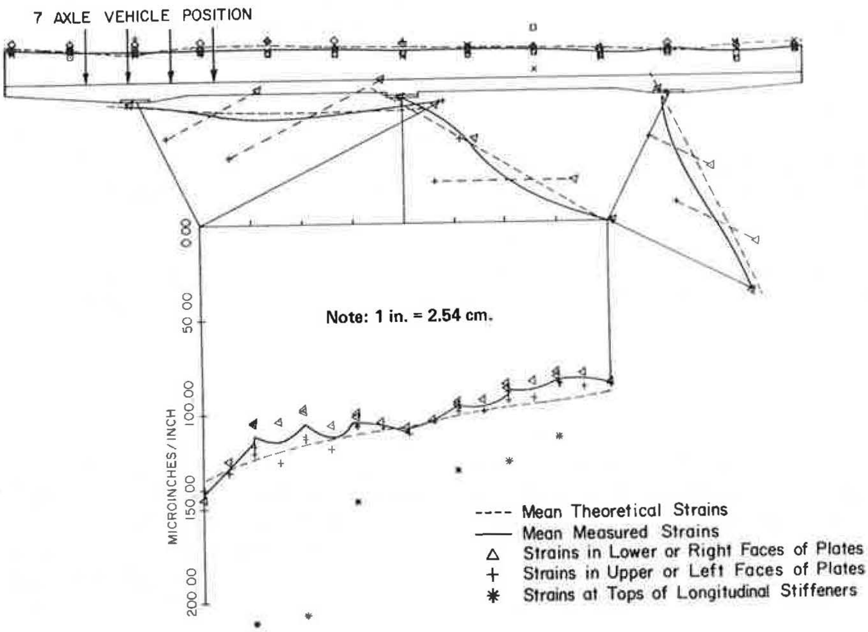
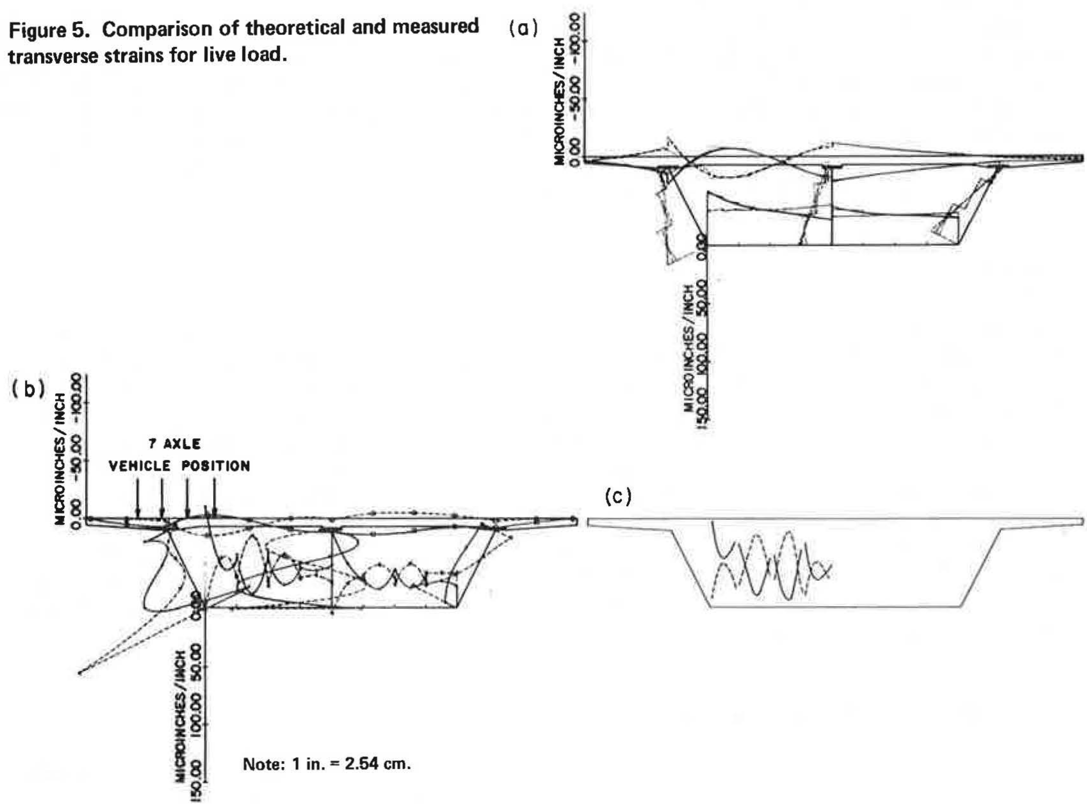


Figure 5. Comparison of theoretical and measured transverse strains for live load.



the mid-plane strains without the influence of local bending, compare favorably. Because questions have been raised frequently concerning the predictability of transverse strains in steel box girders, we tried to assess these prediction methods.

Plate warpage was assessed with some degree of precision by stretching threads at intervals along a 63-in.-long (160-cm-long) section over half the width of the bottom plate and by measuring ordinates to the plate surface. A fine finite element mesh was established by means of measured or interpolated vertical coordinates that were input at the nodes. Output displacement fields at the boundaries from the FINPLA code were then input to a structural design language (STRUDL) program. The resulting fiber strain fields are shown in Figure 5. These strains agreed with measured strains well enough in most areas to suggest that predicting transverse strain anomalies caused by plate warpage is feasible. Practically, of course, predicting maximum strains that result from statistically predicted plate warpage would be the only feasible course.

Dead Load Strains

Theoretical and measured dead load strains are compared in Figure 6. Initial comparisons indicated significant discrepancies, but a field survey revealed that the concrete roadway slab was uniformly thicker than plan dimensions by 0.1 ft (3 cm), which is about 12 percent. Figure 6 shows results of calculations revised to include the relatively significant effects of the increased load of the thicker slab on the bare steel girder. Because of the high costs of running the program, live load calculations were not repeated to assess the less significant influence of the increased stiffness caused by the thicker slab.

Theoretical strains approximated mean, mid-plane, experimental strains, but the program failed to assess anomalies caused by plate warpage and some shear lag, which occur especially in the bottom plate.

The program overestimated the torsional rigidity of the structure. Observed bottom plate strains were larger than theoretical bottom plate strains on the right, and they were smaller on the left. A similar, but much less distinct, tendency can be observed in the live load strain patterns shown in Figure 4. The aforementioned survey indicated no evidence in the slab load of imbalance toward either side of the structure. The asymmetry of the section caused by a 1.5 percent cross slope and a vertical center web were included in the programmed structure geometry.

Deflections

The same anomaly was manifested in the deflection patterns. Theoretical dead load deflections shown in Figure 7 closely approximated those measured along the right upper flange the day after the deck pour, but they exceeded those on the other 2 flanges. Dead load deflections increased because of creep and shrinkage of the slab. Deflections measured 6.5 months after the pour agreed more closely with theoretical deflections at the center web. Measured deflections were greater than theoretical deflections on the right and smaller than theoretical deflections on the left. This anomaly also resulted in curves of theoretical live load deflections as functions of transverse test vehicle position that were significantly flatter than those for measured deflections (Figure 8). The excessive stiffness of the section caused by the thick deck slab, which, as noted previously, was not included in the program, produced theoretical deflections that were uniformly greater than those observed.

SUMMARY AND CONCLUSIONS

With minor exceptions, differences between measured and theoretical deflections and strains were within customary design tolerances. Accurate assessment of fiber stresses requires predicting plate warpage. One project objective entailed determining

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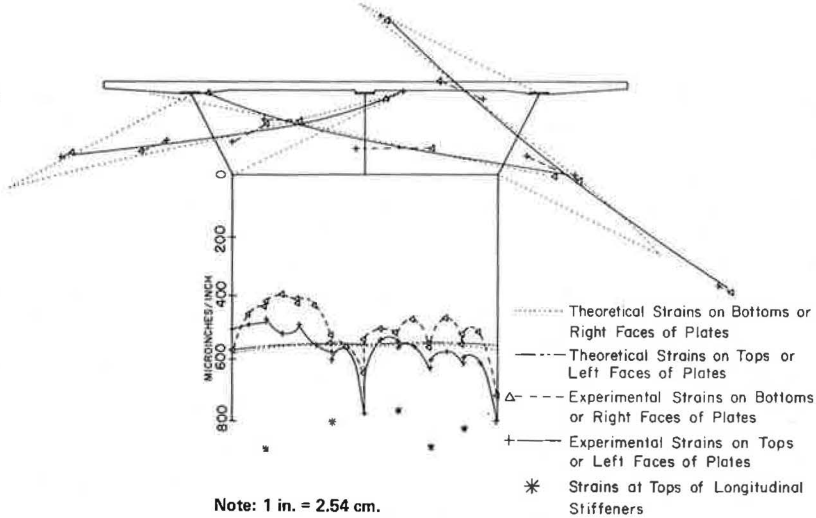
ıl box section.



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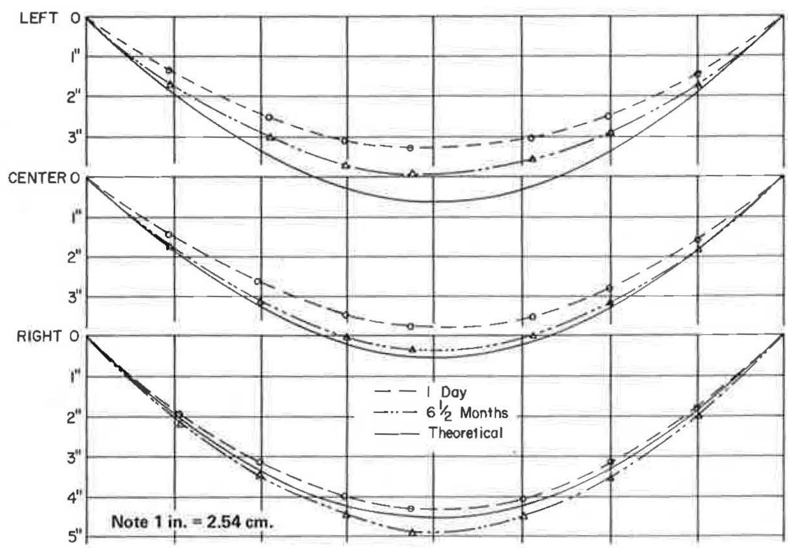

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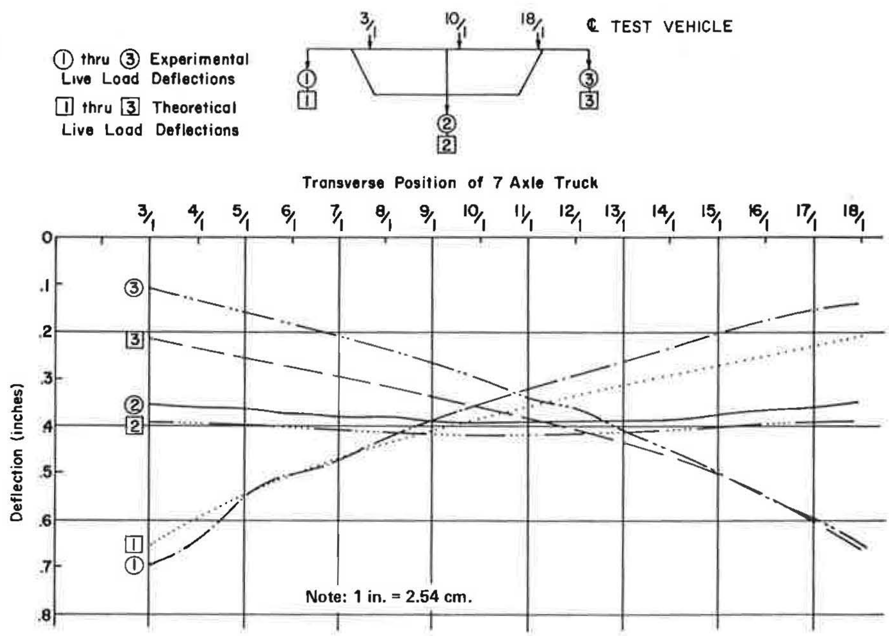

distribution factors for this type of structure. However, previous studies have demonstrated that the distribution factors that employ only stringer spacing may ignore other factors that have important influence in characterizing live load distribution (4). Moreover, in a structure having sloping webs, as most steel box sections do, some question might arise concerning the dimensions for stringer spacing.

The gross moment of inertia method used in the original design of the tested prototype offers little provision for accurate assessment of the true distribution characteristics of the cellular box section and is, as a consequence, somewhat conservative. Moreover, the method cannot provide even approximate values of transverse stresses. We believe that future designs of this type of structure could be improved and that significant economies could be realized in design by applying proven numerical procedures comparable to the procedure described in this paper.

Because few composite steel box girders are designed in the United States and because of the reasonable agreement between theoretical and measured strains in this study, we recommend that those engaged in design and analysis of such structures use sophisticated, computerized, finite element codes such as FINPLA instead of distribution factors that may have questionable theoretical bases.

ACKNOWLEDGMENTS

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LABORATORY EVALUATION OF FULL-SIZE, ELASTOMERIC, BRIDGE BEARING PADS

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Elastomeric, bridge bearing pads, which were reinforced by layers of steel, fiberglass, or polyester cloth, were evaluated by laboratory testing. Full-size specimens were loaded in simulated service conditions. Steel and fiberglass reinforcement showed decided superiority.

ABRIDGMENT

•ELASTOMERIC pads were first used as bridge bearings in the United States in the late 1950s generally because a satisfactory bearing device that could accommodate the relatively severe end rotation and translation associated with prestressed concrete structures was needed. Bearings more economical and maintenance free than those used previously also were desired. At that time, California started a research project to establish design guidelines and specifications for these pads (1). That study revealed that neoprene pads reinforced at ½-in. (12.7-mm) intervals with steel sheet or polyester fabric performed satisfactorily in the bridges constructed during that period. The polyester fabric became the most commonly used reinforcement in California. Polyester fabric is less expensive than steel because large pads can be fabricated, stockpiled, and sliced into custom sizes on demand. Steel-reinforced pads must be fabricated individually to the desired size because the edges of the steel must be covered with elastomer for corrosion protection.

During the 1960s, use of prestressed concrete bridges became more common and typical span lengths became longer because of designers' interest in economy, safety, and aesthetics. Consequently, bearing pads became larger in both plan area and thickness to accommodate increased loads, translations, and rotations. As pad sizes increased, construction personnel began to notice pad deflections that were considerably different from those anticipated. At that time pad deflections were predicted on the basis of tests performed on relatively small pads. Some design data (2) were extrapolated to estimate the behavior of the pads being used. When it became apparent that extrapolation of data from small pads would not ensure satisfactory performance of large pads, the research project that will be discussed in this paper was initiated to evaluate the physical characteristics of full-size bearing pads and to modify the pertinent specifications and design criteria if necessary.

The objective of this research was to evaluate the performance of full-size bearing pads under test conditions simulating actual field use. Various shapes and sizes of pads up to 7 ft^2 (0.65 m^2) in plan area and 5 in. (127 mm) in thickness were subjected to compressive, cycling, creep, translation, rotation, and ultimate-strength tests. Typical pads consisted of 55-durometer-hardness neoprene reinforced at ½-in. (12.7-mm) intervals with steel, polyester, or fiberglass reinforcement.

This abridgment is a condensed version of a more detailed paper that is available elsewhere (3).

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CONCLUSIONS

These conclusions are based on laboratory testing at approximately 70 F (21 C) and apply to pads fabricated in accordance with California specifications (3). The pads exhibited 55 ± 5 durometer hardness (ASTM D 1149, type A); they were reinforced at intervals of $\frac{1}{2} \pm \frac{1}{8}$ in. (12.7 ± 3.2 mm). Reinforcement was fabric or 20-gauge (0.91-mm) mild steel with a minimum ultimate tensile strength of 700 lb/in. (122.5 kN/m) at top and bottom of pad and 1,400 lb/in. (245 kN/m) within the pad.

Polyester-Reinforced Pads

Compressive deflection of polyester-reinforced pads is difficult to predict accurately for 4 reasons.

1. Magnitude of deflections of polyester-reinforced pads is much greater than that of steel- or fiberglass-reinforced pads because of the relative tensile flexibility of the polyester fabric.
2. Compressive stiffness decreases as overall pad thickness increases.
3. Compressive creep of polyester-reinforced pads under sustained dead load stresses is 2 to 3 times that of steel- or fiberglass-reinforced pads because of polyester fabric creep.
4. Compressive deflections due to live load cycling tend to remain in the pad after the live load is removed.

Translation and ultimate strength properties of polyester-reinforced pads are similar to those of fiberglass-reinforced pads.

Fiberglass- and Steel-Reinforced Pads

The following conclusions hold for fiberglass- or steel-reinforced pads:

1. Compressive deflections can be reliably predicted within the normal range of construction tolerances;
2. Compressive stiffness is not significantly dependent on overall pad thickness;
3. Compressive creep under sustained dead load stresses is approximately 25 percent of initial deflection after 10 years of service;
4. Compressive deflections due to live load cycling tend to diminish after live load is removed;
5. Ultimate compressive strength is more than 1,600 lb/in.^{2} (11 040 kPa) (mode of failure is fabric tearing or steel yielding);
6. Under a nominal compressive load of 800 lb/in.^{2} (5516 kPa), fiberglass- or steel-reinforced pads may be subjected to rotational forces until compressive strain at an extreme edge is 0 without damaging the pad; and
7. Shear modulus is approximately 100 lb/in.^{2} (690 kPa) at 70 F (21 C). This value is not significantly dependent on pad size, shape, skew angle, or compressive stress.

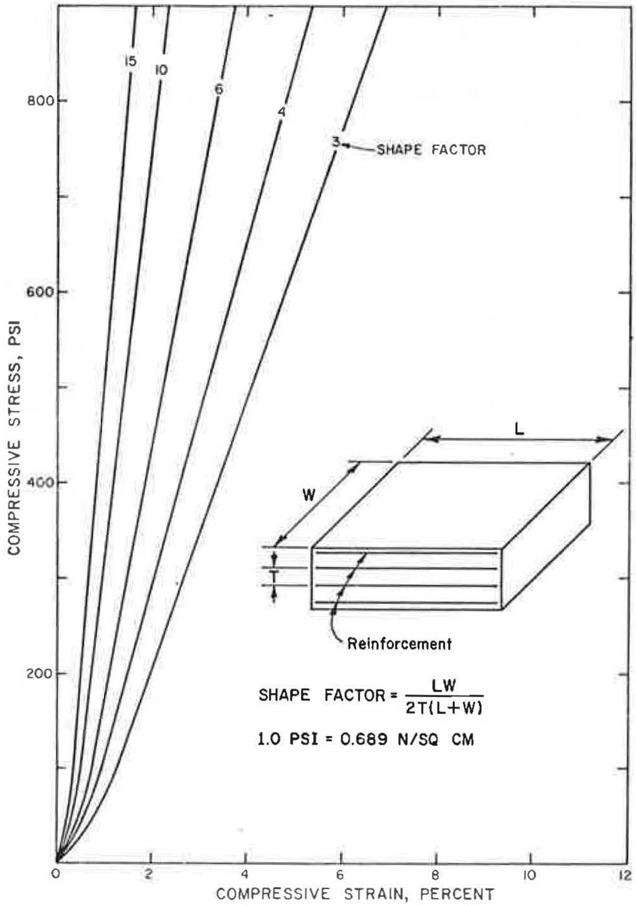
RECOMMENDATIONS

Polyester-reinforced pads more than 1 in. (25 mm) in thickness should not be used in bridge bearings because they make it difficult to predict compressive deflection.

For pad thicknesses normally used in bridge construction, steel- or fiberglass-reinforced pads should be specified in accordance with the specifications presented elsewhere (3).

Compressive deflections for steel- or fiberglass-reinforced pads should be predicted by using the data shown in Figure 1. The accuracy of these curves is considered to be

Figure 1. Recommended compressive stress-strain curves for steel- or fiberglass-reinforced pad of 55 durometer neoprene.



well within the range of normal construction tolerances. If long-term compressive creep is to be included in the prediction, then the values obtained from Figure 1 should be increased by 25 percent. For special situations requiring extreme accuracy, sample pads should be tested to determine the stress-strain behavior of each lot of pads.

Further research is needed to improve specifications and test methods used to ensure the quality of bridge-bearing pads. Based on field performance to date, current specifications and test methods result in high-quality pads, but these requirements vary considerably throughout the nation. Some tests are difficult or expensive to perform, and, in some cases, requirements may be unnecessarily conservative and restrictive. Research is needed to develop simple and inexpensive test methods of performance requirements.

If further research is contemplated for large bearing pads, careful consideration must be given to test method details (3).

The recommendations on pad thickness were implemented by the California Department of Transportation in late 1972. Since that time, there have been no reports of adverse performance of fiberglass-reinforced pads.

ACKNOWLEDGMENTS

This research was accomplished in cooperation with the Federal Highway Administration. The contents of this report reflect the views of the Transportation Laboratory, California Department of Transportation, which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the state of California or the Federal Highway Administration.

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