formations in Segmental Prestressed Concrete Bridges. Civil Engineering Studies, Univ. of Illinois, Urbana, Structural Research Series 495, Oct. 1981, 242 pp.
3. International Recommendations for the Design and Construction of Concrete Structures: Principles and Recommendations. Comite Europeen du Benton (European Committee on Concrete), Paris, June 1970.
4. H.D. Hernandez and W.L. Gamble. Time-Dependent Prestress Losses in Pretensioned Concrete Construction. Civil Engineering Studies, Univ. of Illinois, Urbana, Structural Research Series 417, May 1975.

Publication of this paper sponsored by Committee on Concrete Bridges.

Abridgment

Load Capacity of Concrete Bridge Decks

DAVID B. BEAL

The behavior of two reduced-scale concrete bridge decks subjected to simulated wheel loads was evaluated in a series of tests. One slab was reinforced in accordance with American Association of State Highway and Transportation Officials requirements and the other had three areas with varying amounts of isotropic reinforcement. Results show that with either reinforcement pattern, service load bending moments are from 40 to 65 percent of those predicted by flexural theory. Failures were by punching shear rather than flexure and occurred at loads at least six times larger than design.

The need to determine the influence of heavily loaded, closely spaced wheels and axles on reinforced concrete bridge decks prompted the New York State Department of Transportation (NYSDOT) to initiate an analytical study of bridge deck behavior in 1977. The products of this study were charts that permitted the determination of the induced bending moment in decks due to any pattern of wheel loads (1). During the course of the research, more evidence became available [which culminated in the publication of the Ontario Highway Bridge Design Code (2)] that the failure mode of reinforced concrete bridge decks was punching shear and not flexure as assumed in design. Because of this evidence, the study reported here was started to investigate the ultimate capacity of bridge decks.

The Ontario bridge deck design resulted from extensive physical and analytical research (3). This work demonstrated that not only is the failure mode of reinforced bridge decks different from that historically assumed, but that the load capacity is substantially greater than necessary for safety. The enhanced behavior of bridge decks is explained by hypothesizing large in-plant compressive forces that result from the restraint of deck expansion under load. These compressive forces form an internal couple that enhances the flexural capacity of the deck to a level such that punching shear failure controls. Subject to certain restrictions on span length, slab thickness, and detailing of diaphragms and shear connectors, Ontario permits an empirical slab design that has a minimum of 0.3 percent isotropic reinforcement in each face. For a 9-ft slab span, this represents a reduction in reinforcement of 43 percent from that now required by New York State standards (4).

In addition to the savings that result from the reduction of steel, benefits may accrue from this empirical design by reducing fabrication costs and deck deterioration due to reinforcement corrosion. The reinforcement can be standardized over the normal range of girder spacings and has the potential benefit of modular prefabrication. Reinforcement corrosion is alleviated because the cover on the top steel can be increased without an increase in slab thickness. In addition, the reduction in bar size increases the important cover-diameter ratio (5).

The objective of the work described in this paper was to collect data on bridge slab capacity with different reinforcement schemes. This work was accomplished with several reduced-scale models that are described in detail in the complete report (6).

EXPERIMENTAL WORK

Two reinforced concrete bridge decks were constructed to study behavior under working loads and at failure. Model 1, which represents the current standard bridge deck design (4), was included to demonstrate the great reserve capacity of that design and to provide a standard of comparison with alternative designs. The 8.5-in-thick slab was reinforced with No. 5 bars in the longitudinal and transverse directions. Longitudinally, the top layer of bars was spaced at 18 in and had the bottom layer spaced at 7.75 in in the middle half of the slab span and at 18 in elsewhere. Both layers of transverse steel were spaced at 5.25 in.

Model 2 represented an 8-in-thick deck that has three different isotropic reinforcement patterns comprised of No. 4 bars. Two layers at 8-in spacing represented current Ontario practice (3). A single layer at 8 in was used because of the construction benefits to be gained if this pattern could be adopted. Two layers at 12-in spacing were used to represent the minimum reinforcement now permitted (7). In addition, an unreinforced section was included to demonstrate the inherent strength of confined concrete slabs. Both models were constructed to a linear scale factor of 5.9 and were based on a five-girder, 72-ft simple-span bridge, which is representative of composite highway structures now being built. Details of model materials and construction details are given in the full report (6). Electrical-resistance strain gages were mounted on the rebars and deflection at the center of the slab was measured.

TEST RESULTS

Each instrumented section of the model slabs was subjected to test loads for two distinct purposes: (a) determination of the distribution of bending

Figure 1. Model 1 transverse moment coefficients.

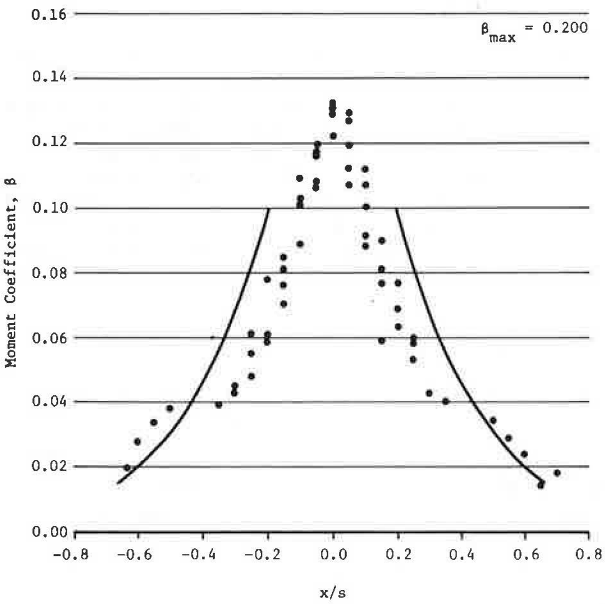
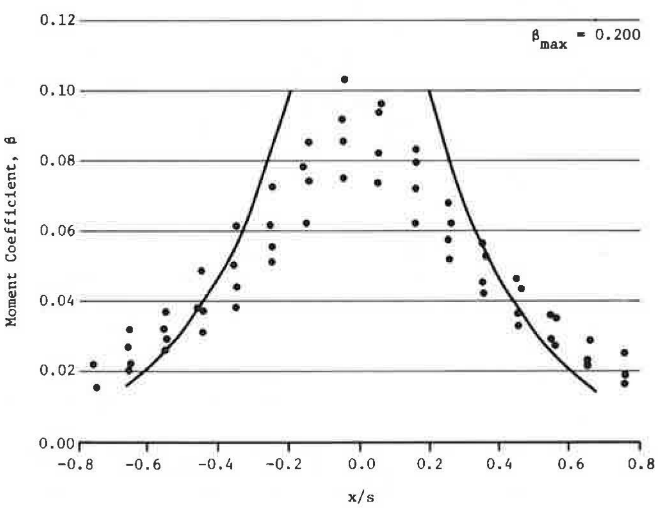


Figure 2. Model 2 interior-panel transverse moment coefficients.



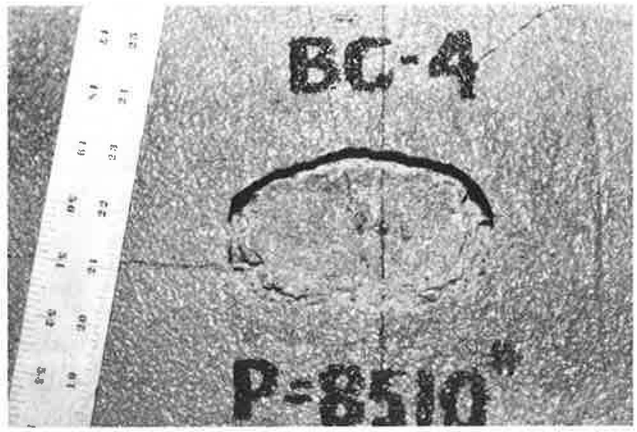
moment and (b) determination of failure load.

Uncracked- and cracked-slab testing was performed. The uncracked-slab testing of model 1 provided data for comparison with results from a test on a prototype structure. The transverse rebar stresses determined in these tests showed good correspondence in trends. The absolute magnitudes differed because of physical differences between model and prototype.

The cracked-slab testing was considered more important, since steel strains are more sensitive to bending moment, and the slab would be in this condition under overloaded vehicles. The slabs were cracked by loading repeatedly at points surrounding the instrumented section until a linear load-strain response was obtained in the transverse rebars. Loads of 5000 and 3000 lb were required on models 1 and 2, respectively, to achieve a nominal rebar strain of 1400 μin/in.

Figures 1 and 2 show typical responses for models 1 and 2, respectively [curves and β_{max} values from Beal (1)]. These results are compared with the

Figure 3. Punching failure on model 1.



results from an elastic analysis (1). The experimental values of moment coefficients are generally less than the theoretical values. The maximum predicted value is 0.2 compared with average experimental results of 0.126 and 0.074, respectively. Under service load, the maximum stress in the transverse rebars was less than 8.3 ksi for conventionally reinforced slabs and less than 11.7 ksi with isotropic reinforcement.

Failure loads were applied through either a 1.69x4.07-in or 1.69x1.69-in pad. The smaller pad was used more frequently because load levels controlled by yield strength of the steel beams were generally reached before slab failure with the larger pad. For some tests, auxiliary supports were used to control beam stresses.

In general, the failure mode at all locations bounded by longitudinal girders was punching shear (Figure 3). The intersection of the failure surface with the tension face of the slab was elliptical and had average major and minor axes of 14 and 12 in, respectively; this face was extensively cracked. The failure at the top surface was only slightly larger than the load pad. Slab deflection at failure never exceeded 10 percent of the slab thickness.

A total of 15 failures were produced in the model 1 deck and 13 failures in model 2. Excluding tests with oversized load pads, the average equivalent prototype failure loads are 260 and 300 kips for the fascia and interior panels of model 1, respectively. For model 2, the failure loads varied, depending on the type of reinforcement and slab boundary conditions. The failure load exceeded 130 kips (six times the design wheel load) in all cases.

In addition to the tests on the reinforced section, the unreinforced section was tested and sustained an equivalent prototype load of 175 kips before loading was stopped because the cracking was propagating toward the reinforced section. In later tests, other load points on the unreinforced slab sustained loads of 70 and 100 kips, respectively.

ANALYTICAL PREDICTIONS

An analytical procedure to predict the punching shear capacity of reinforced concrete bridge decks has been developed by Hewitt and Batchelor (8) based on a theory of punching shear behavior proposed by Kinnunen and Nylander (9). A computer program was prepared based on this procedure.

Kinnunen and Nylander (9) predicted the capacity of circular slabs by hypothesizing the existence of a conical shell of concrete extending from the loaded surface of the slab to the bottom of the shear crack. Hewitt and Batchelor (8) extended this theory to include boundary restraining forces and moments in the plane of the slab. Results from the computer program prepared for this work were satisfactorily compared with their results.

The capacity of the three reinforcement patterns for model 2 interior panels was predicted by this theory. A restraint factor of 0.5 was assumed. In all cases the analytical value was less than the average test load. The analysis does not include the boundary force contribution of compression reinforcement, and thus single and double mats are predicted to have the same strength. A 1000-lb difference in test capacities of the single and double mats indicates that compression reinforcement does make a significant contribution to load capacity.

PRACTICAL APPLICATIONS

Before implementation of a bridge deck design that incorporates reduced reinforcement, many questions must be resolved. First, strength criteria must be established to ensure safety. Because of the suddenness of the punching failures, load factors must be more conservative than values selected when yielding failures are expected. Second, the minimum acceptable reinforcement amount that satisfies both strength and serviceability (i.e., temperature and shrinkage) must be determined. Third, the behavior of the new design under common but nonstandard conditions must be determined. For example, Is adequate performance achieved without shear connectors or in negative-moment regions where the slab is in longitudinal tension? Finally, reinforcement details for the fascia overhang must be developed to ensure adequate performance.

Three important benefits could accrue if an isotropic deck reinforcement pattern were adopted. First, average reduction in steel quantities over current design requirements is estimated at 53 percent for double-mat reinforcement (No. 4 at 12 in). Second, savings in fabrication costs can be expected due to standardization of the reinforcement pattern and reduction in the number of bars. Third, rebar corrosion will be alleviated due to smaller bar diameter and increased cover.

SUMMARY AND CONCLUSION

Tests on reduced-scale reinforced concrete bridge decks have demonstrated that the service load stress levels predicted by existing design procedures and methods based on elastic isotropic thin plates do not develop in bridge decks of ordinary proportions. The maximum induced stress in a conventionally reinforced deck subjected to the design load of 20.8 kips was only 8.3 ksi. Tests on model bridge decks that have substantially less reinforcement than is ordinary caused stresses no greater than 11.7 ksi at the design load. Comparison of induced moments also showed that measured values are less than predicted. The maximum ratios of measured to theoretical induced moment were 0.65 and 0.62 for the conventional and lightly reinforced decks, respectively. Based on these results, it is concluded that even a 100 percent increase in the weight of wheel loads would not overstress the deck reinforcement and, accordingly, no methodology is needed to predict induced stress that results from the passage of occasional overload permit vehicles.

Tests to failure resulted in capacities always larger than six times the design wheel load for slabs bounded by girders, regardless of the reinforcement pattern. In addition, with the exception of two locations where the reinforcement was misplaced, all failures were by punching. Thus, reductions in total reinforcement of 30 to 53 percent had no effect on failure mode and did not reduce the strength below a safe level.

ACKNOWLEDGMENT

The research leading to this paper was sponsored by NYSDOT and the Federal Highway Administration, U.S. Department of Transportation. The preliminary planning for this work was done by Robert J. McDermott, who also supervised the construction and testing of model 1. Laboratory work was performed under the direction of Everett W. Dillon with the assistance of R.L. Morgan.

REFERENCES

1. D.B. Beal. Bending Moment in Bridge Slabs Due to Wheel Loads. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Res. Rept. 63, Aug. 1978.
2. Ontario Highway Bridge Design Code. Ontario Ministry of Transportation and Communications, Ottawa, Ontario, Canada, 1979, Section 7.
3. P. Csagoly, M. Holowka, and R. Dorton. The True Behavior of Thin Concrete Bridge Slabs. TRB, Transportation Research Record 664, 1978, pp. 171-179.
4. Standard Details for Highway Bridges. Structures Design and Construction Subdivision, New York State Department of Transportation, Albany, May 1977, Section 23.1.3.
5. J.T. Houston, E. Atimay, and P.M. Ferguson. Corrosion of Reinforcing Steel Embedded in Structural Concrete. Center for Highway Research, Univ. of Texas, Austin, Rept. CFHR-3-5-68-112-1F, 1972.
6. D.B. Beal. Strength of Concrete Bridge Decks. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, Res. Rept. 89, July 1981.
7. Standard Specifications for Highway Bridges, 12th ed. AASHTO, Washington, DC, 1977, Section 1.5.7, p. 76.
8. B.E. Hewitt and B. Batchelor. Punching Shear Strength of Restrained Slabs. Journal of the Structural Division, ASCE, Vol. 101, No. ST9, Proc. Paper 11548, Sept. 1975, pp. 1837-1853.
9. S. Kinnunen and H. Nylander. Punching of Concrete Slabs Without Shear Reinforcement. Trans., Royal Institute of Technology, Stockholm, No. 158, 1960.

Publication of this paper sponsored by Committee on Concrete Bridges.