AN INVESTIGATION OF THE FATIGUE STRENGTH OF DECK SLABS OF COMPOSITE STEEL/CONCRETE BRIDGES

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An investigation of the effects of repeated loads on slabs of steel/concrete composite bridges - the most common type in highway construction - was undertaken to supplement static studies conducted under the Ontario Joint Transportation and Communications Research Program No. Q50. The study which involved tests of a number of 1/8th scale direct models of a typical bridge, was aimed at determining endurance limits of the slabs under repeated concentrated loads. Main variables were stress range, and percentage and arrangement of reinforcement. Emphasis was focused on slabs with 0.2 percent top and bottom isotropic reinforcement, this being the amount recommended for use as a result of the static model testing phase of the program. The study showed that the deck slabs of conventionally designed steel/concrete bridges have large reserves of fatigue strength. An endurance limit of 50 percent of the ultimate capacity can be expected in such slabs. In the case of slabs with 0.2 percent isotropic reinforcement, an endurance limit of 40 percent of the ultimate static capacity can be safely adopted for design. These slabs have also performed adequately in static tests; and adoption of their use, where appropriate, would result in considerable reduction of reinforcement requirements in bridge decks.

Background

The experimental study reported in this paper is an integral part of a major program aimed at obtaining a thorough understanding of the behaviour of concrete bridge deck slabs supported by beams or girders. Other parts of the project have included extensive testing of deck slabs of model and prototype bridges, measurement of horizontal membrane forces in circular slab specimens, and an investigation of scale effects in model deck slabs subjected to concentrated loads.

It has been found that, contrary to the classical plate bending theory developed by Nadai and promoted successfully by Westergaard in North America, slabs of practical thicknesses carry concentrated loads primarily by compressive membrane action, often referred to as "internal arching". The development of membrane forces in typical deck slabs is a direct result of lateral confinement of the loaded slab by its supporting members or by adjacent portions of the slab itself. Indications are that while a bending component is still present, it is not significant as far as the load-carrying capacity of the slab is concerned.

The fact that the major part of the load is carried by compressive membrane action permits a substantial reduction of the slab reinforcing steel requirements. It appears that the minimum amount of steel specified by ACI for crack control against volumetric changes, is adequate to carry all present day and probable future live loads. Since this reduction entails the elimination of two-thirds to three-quarters of the current typical reinforcement requirements, it was considered advisable to investigate various aspects other than the ultimate strengths of these slabs. One such aspect reported in this paper is the fatigue life of reinforced concrete slabs, an area of structural engineering, which has received very little attention.

It is shown that typical deck slabs designed with a consideration of the high compressive membrane stresses developed when subjected to concentrated loads, can be expected to give satisfactory fatigue performance, even though their reinforcement requirements are nominal.

Brief Historical Review

Although it is generally recognized that the repeated load that will cause failure of a structure is likely to be significantly less than the static failure load, the behaviour of bridge decks under repeated loads has in the past received scant attention. A review of the available data related to fatigue of reinforced concrete reveals that the past studies were primarily carried out on isolated beams as opposed to slabs. Exceptions are the studies by Graf (reported by Nordby (1) and Loo (2)). It is recognized that substantial differences exist in the structural responses of reinforced concrete beams and slabs, therefore, it appears questionable whether fatigue behaviour observed in reinforced concrete beams can be reliably extended to slabs. In 1958, Nordby (1) concluded the following in a review of fatigue of reinforced concrete beams:

a) Most failures of reinforced concrete beams were due to the fracture of reinforcing steel. Under-reinforced beams appeared to have a fatigue load limit of 60 to 70 percent of the static ultimate strengths for a life of 1,000,000 cycles of loading.

b) Except for a few cases, compression failures were virtually absent.

c) Failure was often due to diagonal tension, though the real cause of failure was obscured by a combination of bond and shear failures. Beams have failed in shear under repeated loads at levels of 40 percent of static ultimate strength.

d) Accumulation of residual deflections occurred under extensive fatigue loading but some amount of recovery was evident during rest periods.

In an investigation conducted at Queen's University on the slabs of a reinforced concrete double-T beam bridge, Loo (2) reported that different fatigue failure modes occurred in the slabs. At loads close to the static capacity, failure was mainly due to punching after a small number of cycles of loading. At lower levels of loading, failure was caused by fracture of the reinforcing steel after a large number of cycles. Loo found that the fatigue behaviour of slabs subjected to concentrated loads could be predicted from the fatigue properties of the reinforcing steel. Although there was no positive evidence of an endurance limit in the slabs tested, Loo speculated that this may be as low as one-half of their static capacities.

In a model study of composite bridges (3,4) Batchelor and Hewitt concluded that the conventional design of the deck slabs of composite bridges is very conservative. It was shown that the deck reinforcement requirements in such slabs could be safely reduced. However, the study did not include a consideration of the response to repeated loads, of slabs designed by the method proposed which results in a substantial reduction of the reinforcing steel requirements.

Scope of Investigation

The study reported in this paper was, therefore, undertaken to determine the fatigue strength of composite steel/concrete deck slabs and, in particular, the fatigue behaviour of slabs with the reduced amount of reinforcement recommended in the static studies carried out under Project Q50 (3). The fatigue study was considered essential in view of the considerable reduction of reinforcement proposed, and in view of the fact that with the increasing intensity of commercial traffic, bridges are now required to have longer fatigue life than in the past.

The investigation, which was mainly experimental, has shown that deck slabs with either conventional orthotropic reinforcement, or with the recommended reduced reinforcement ratios, have large reserves of strength against fatigue failure, thereby confirming the validity of the design recommendations based on the static tests.

In this report an orthotropic slab is one in which the reinforcement ratios provided in a face, in two orthogonal directions, are unequal. An isotropic slab is one in which the reinforcement ratios in a face are equal in two orthogonal directions.

Model Bridges and Tests

The study was carried out on a total of five 1/8th scale direct models of a 24.4 m (80 ft) simply supported four-beam bridge. The prototype and model bridges are detailed in Figure 1. Dead load stresses appropriate to unshored construction were simulated in one model. The means of inducing dead load stresses, as well as the solutions of problems encountered in modelling material properties, section properties and stud shear connector behaviour, are described in detail elsewhere (4,5).

The investigation started with the fatigue studies by Dixon (6), and, initially slabs with orthotropic reinforcement were investigated. Subsequent studies involved top and bottom isotropic reinforcement which was found suitable in the previous static studies, as well as mid-depth reinforcement which would afford maximum cover.

All panels of a model were tested under concentrated loads. A bridge panel is defined as that portion of the deck bounded by adjacent bridge beams and adjacent diaphragms. The model bridges tested are numbered 1 to 5, and the designation of the bridge panels is shown in Figure 2. To facilitate testing of panels with different percentages of reinforcement, the amount of reinforcement was varied in bridges numbered 3, 4, and 5, since it was concluded from previous static tests that the behaviour of a panel under load is not influenced by the reinforcement of adjacent panels. The slab reinforcement in each bridge is given in Table 1. Dead load compensation was

FIGURE 1: DETAILS OF PROTOTYPE AND MODEL BRIDGES.

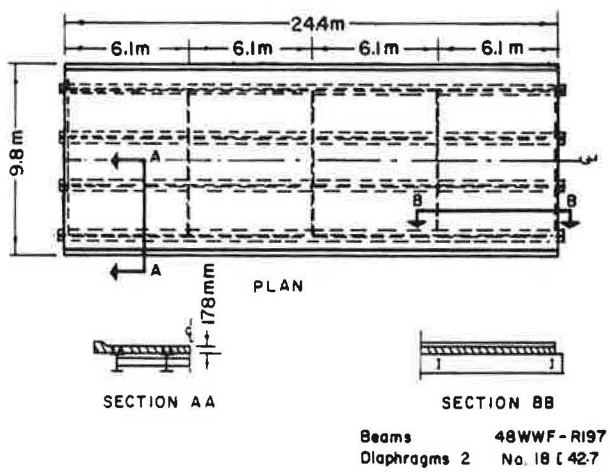


FIGURE 2: DESIGNATION OF BRIDGE PANELS.

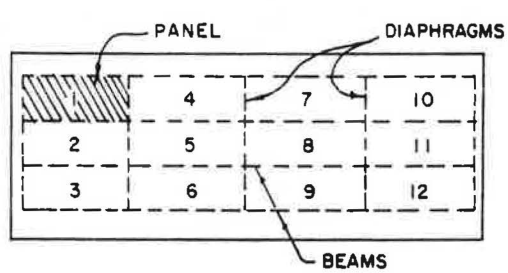
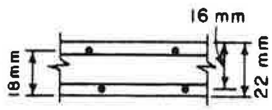


TABLE 1, DETAILS OF MODELS

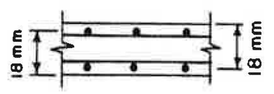
BRIDGE NO.	REINFORCEMENT			
	PANELS 1, 2, 3,	PANELS 4, 5, 6,	PANELS 7, 8, 9,	PANELS 10, 11, 12,
1	ORT	ORT	ORT	ORT
2	ORT	ORT	ORT	ORT
3	0.4%**	0.2%	ZERO	0.2%
4	0.4%	ZERO	2"***	0.2%
5	51 mm	76 mm	NONE	25 mm

LOGITUDINAL SECTIONS

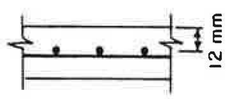
*NORMAL (AS PER AASHTO) ORTHOTROPIC REINFORCEMENT



**TOP AND BOTTOM ISOTROPIC REINFORCEMENT (DESIGNATED BY REINFORCEMENT PERCENTAGE)



***MID-DEPTH ISOTROPIC REINFORCEMENT (DESIGNATED BY REINFORCEMENT SPACING)



applied to bridge No. 2 and the results of tests on this model indicated that for the scale of the model and with the types of slabs involved, dead load compensation could be eliminated.

In all cases the single concentrated load was applied at the centre of a panel through a steel plate bearing on a neoprene pad. The contact area modelled an ellipse with major and minor axes of 762 mm and 508 mm (30 and 20 in) respectively, which represents the assumed contact area of the pneumatic tires of large trucks. Loading was applied by means of an MTS closed loop electro-hydraulic testing system. A number of panels of each model were tested to failure under static loading. The static failure load was then used to set the maximum load of the fatigue loading function for the corresponding panels of the particular model. The fatigue loading was continued until panel failure, or up to a minimum of 2,000,000 cycles of loading. The AASHTO Standard specifications for Highway Bridges (7) specify 2,000,000 as the number of cycles of maximum stress to be considered in design.

The applied dynamic load was in the form of a sinusoidal wave superimposed on a mean value. The applied load was programmed so that the minimum load was of the order of 890 N (200 lb), and the maximum load was a proportion of the estimated static strength of the panel. The frequency of the applied load was varied for different tests because the response of the dynamic loading system was influenced by the travel of the loading ram and therefore by the bridge deflections. Frequencies as high as 5 Hz could be used for tests in regions near the end supports of the bridge where the deflections were small. On the other hand, in tests towards mid-span of the bridges, frequencies as low as 1 Hz were necessary. Nordby (1) has noted that a rate of testing between 1 and 7 Hz has little or no effect on the fatigue strength of plain concrete, and it is considered that the variation of the testing rate in this investigation did not significantly influence the fatigue behaviour of the deck slabs.

Model Material Properties

Concrete

A special concrete suitable for structural models, and which was developed at Queen's University, was used for the bridge deck slabs. Although a very fine aggregate was used, the ratio of tensile to the compressive strength for this concrete is similar to that of prototype concrete.

The properties of the concrete of the deck slabs of the model bridges are given in Table 2. The tabulated compressive and tensile strengths were determined from 102 mm x 203 mm (4 x 8 in) cylinders which were tested about halfway through the testing program for a particular bridge model. The tests conducted under Project Q50 (3) indicated that the static ultimate load carrying capacity of rectangular slabs of a composite bridge deck is not significantly influenced by the strength of the concrete. It was therefore considered reasonable to assume that the concrete strength had little influence on the fatigue behaviour of the bridge decks.

Reinforcing Steel

The reinforcement used in the models was a 13 gauge wire having a diameter of 2.32 mm (0.0915 in.) and manufactured by the Steel Company of Canada Ltd. The wire was indented and specially annealed so that it had a yield stress conforming to ASTM Standards for intermediate grade steel.

Table 2. Model concrete properties

Bridge No.	Age when tested (days)	Compressive strength f'_c MPa	Splitting tensile strength f_t MPa
1	28	35.54	2.48[a]
2	64	37.65	Not done
3	43	33.37	3.30
4	23	31.30	3.35
5	35	37.54	3.29

[a]Different mix from others

Note: 1 MPa = 0.145 ksi

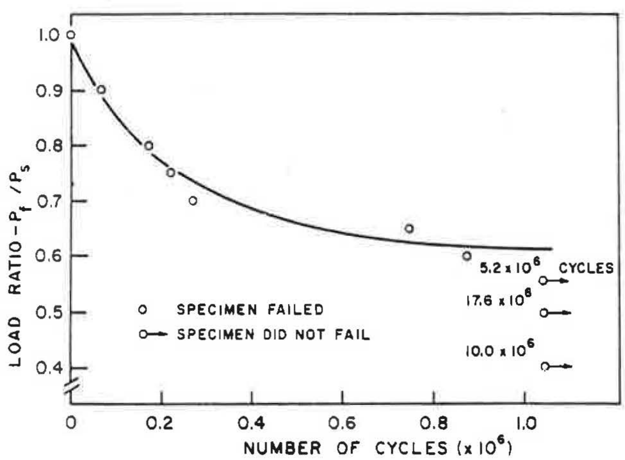
The fatigue behaviour, in tension, of the model reinforcement was investigated in a series of tests using a Sonntag Universal Testing Machine. This was done to determine whether, as suggested by Loo (2), there was any relationship between the life of the slab and that of the reinforcement. As in the case for the model bridges, the applied dynamic load was in the form of a sinusoidal curve superimposed on a mean value. The minimum load was set near zero and the maximum load was a proportion of the ultimate strength of the reinforcement. The load function was applied at a frequency of 30 Hz. In Figure 3 the ratio of maximum load to the ultimate load is plotted against the number of cycles to fatigue failure of the reinforcement. There is some scatter, but it is clear that fatigue failure is unlikely in any practical situation at any load ratio less than about 0.6, which can be taken as an approximate value of the endurance limit for the reinforcing steel.

Observations

Failure Mechanism

A total of 37 panels were tested to failure under fatigue loading and all but a few failed by punching. The crack patterns up to failure were much the same as those obtained in punching failures under static loads. Cracking was observed after a few cycles of repeated loading. The cracks then widened and spread as the number of cycles of loading increased, until a relatively stable condition was reached in which crack propagation was much reduced and any change was imperceptible over a period of a few hours of testing. All visible cracking was observed to be confined to the panel under test, and there was no spreading of the cracks into adjacent panels. The punched area after fatigue failure was often larger and less symmetrical than that resulting in failure under static loading, particularly in slabs

FIGURE 3: LOAD RATIO VERSUS NUMBER OF CYCLES TO FAILURE FOR SLAB REINFORCEMENT.



with little or no reinforcement. In some cases, fracture occurred in the bottom slab reinforcement within the loaded area.

Flexural failures resulted in six panels having little or no reinforcement. It cannot be said that such panels always failed in flexure; however, panels with orthotropic reinforcement or with 0.4 or 0.6 percent isotropic reinforcement always failed in punching. Imminent flexural failure was usually indicated by crushing and spalling of the concrete along lines radiating from the loaded area on the upper surface of the slab.

Deflection Behaviour

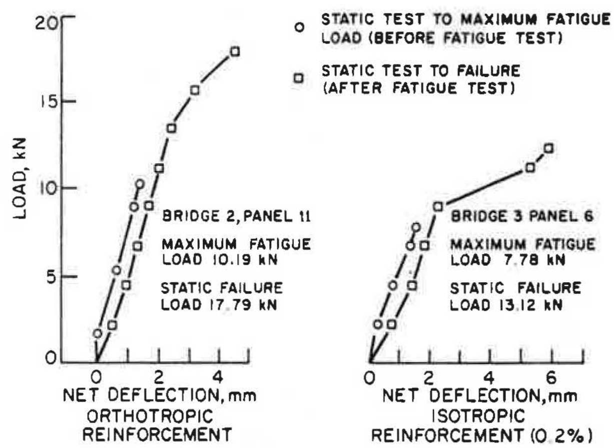
Typical net slab deflections are plotted in Figure 4. These deflections were measured during static tests carried out before and after fatigue tests in which the panel did not fail. In the fatigue test of the orthotropically reinforced panel, over 3 million cycles of loading were applied with the maximum load being 50 percent of the estimated static strength, whereas over 2 million cycles of loading were applied to the isotropically reinforced panel. It can be seen that in each case there was little loss in stiffness up to the maximum fatigue load. The loss of stiffness occurred in the initial stages of the fatigue tests, and can be attributed only to cracking of the concrete.

The maximum net slab deflection was monitored in all fatigue tests. In slabs that did not fail, the net deflection increased initially and then became constant. In the case of slabs that did fail, the net deflection continued to increase up to failure.

Test of Panels with Orthotropic Reinforcement

The results of the tests of panels with orthotropic reinforcement are given in Table 3.

FIGURE 4: TYPICAL NET DEFLECTION VERSUS LOAD, BEFORE AND AFTER FATIGUE.



In each test the maximum load P_f of the fatigue load function was set at a proportion of the static failure load, P_s, of the first tested panel of each bridge. It was indicated in the Project Q50 report of the static tests that there was considerable variation in the static punching loads for panels of the same design dimensions and properties, and that this variation was at least partly attributable to variations in the effective depth of the slab. An assumption that the failure load is the same for all panels with the same amount of reinforcement neglects the influence of slab thickness on punching load. Therefore, in assessing the ultimate strength of an orthotropically reinforced panel, use was made of the relationship in Equation 1 which was derived in Project Q50 for the extremely small slab thickness used in the models:

$$P'_s = -1.139 + 1.361\ d\ (\text{kN}) \quad (1)$$

Table 3. Results of tests of panels with orthotropic reinforcement

Bridge no.	Panel no.	Maximum fatigue load P_f (kN)	$\frac{P_f}{P_s}$	No. of cycles to failure	d (mm)	Estimated static load P'_s (kN)	$\frac{P_f}{P'_s}$
1	2	9.12	0.39	2,052,400	17.6	22.82	0.40
	4	23.36**	1.0	1	17.2	22.24	-
	6	14.41	0.62	756,470	17.4	22.64	0.64
	8	17.93	0.77	1,510	16.5	21.31	0.84
	10	16.77	0.72	91,250	19.1	24.82	0.68
	12	15.52	0.66	103,510	18.1	23.49	0.66
2	1	13.66	0.67	165,000	17.4	22.60	0.60
	2	13.66	0.67	166,900	18.0	23.31	0.59
	3	14.28	0.70	10,940	17.6	22.77	0.63
	4	14.32	0.70	6,010	25.4	21.80	0.66
	5	15.30	0.75	51,040	17.5	22.68	0.67
	6	14.32	0.70	46,190	17.6	22.82	0.63
	7	12.23	0.60	1,143,130	17.2	22.24	0.55
	8	18.37	0.90	35	17.1	22.15	0.83
	9	17.35	0.85	248	17.0	23.04	0.75
	10	20.02	-	1	17.1	22.15	-
	11	10.19	0.50	3,039,000	16.6	21.48	0.47
	12	20.42**	1.0	1	16.9	22.02	-
				Design	17.9	23.17	

Note: 1 mm = 0.039 in
1 kN = 0.225 kip

** Assumed static strength P_s

where P'_s is the punching load, and d is the slab effective depth in mm. The effective depths of panels were determined by averaging the effective depths measured at a number of points in each panel after the completion of testing. Table 3 shows average effective depths, static punching loads obtained from use of the Equation 1, and the resultant ratios P_f/P'_s of maximum fatigue load to the estimated static punching load. Panel No. 2 of bridge No. 1 and Panel No. 11 of bridge No. 2 did not fail under fatigue loading.

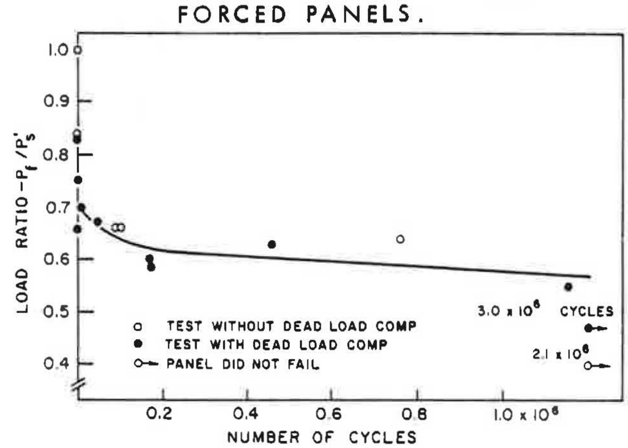
In Figure 5 the ratio of the maximum load to the estimated static punching load is plotted against the number of cycles to failure, while Figure 6 is a semi-log plot of the test results. The second order 'line of best fit' shown in Figure 6 was determined using the method of least squares and has the equation:

$$\frac{P_f}{P'_s} = 1.0 - 0.102N + 0.006N^2 \qquad (2)$$

where N is the log. of the number of cycles to failure. The static test point $P_f/P'_s = 1.0$ was weighted in the least squares analysis. The load ratio is equivalent to the stress ratio S, thus Equation 2 also defines the S-N curve for orthotropically reinforced panels.

The results of tests with and without dead load compensation are indicated in Figures 5 and 6. It appears that the panels without dead load compensation gave slightly higher fatigue strength than the others. However, the scatter of the results is no more than is to be expected in fatigue studies of reinforced concrete structures; and it is unlikely that the lack of dead load compensation could have a significant effect on the

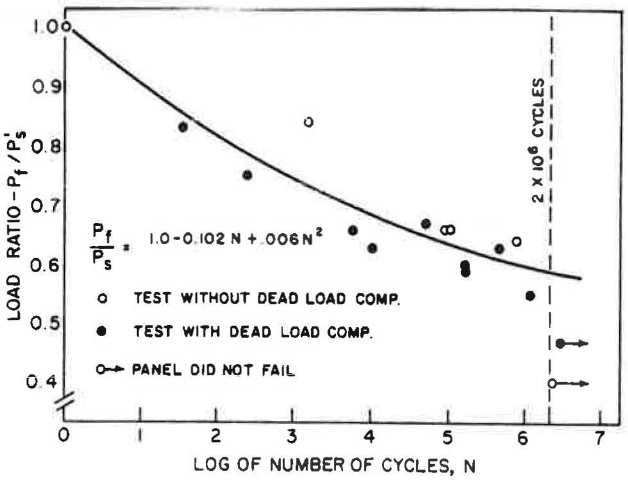
FIGURE 5: LOAD RATIO VERSUS NUMBER OF CYCLES TO FAILURE FOR ORTHOTROPICALLY REINFORCED PANELS.



fatigue behaviour of the slabs.

Figures 5 and 6 indicate that the endurance limit of orthotropically reinforced panels is at least 50 percent of the static ultimate load. The static tests suggest that a 1/8th scale model of a panel of design dimensions has a static strength of 23.17 kN (5.21 kips). If an endurance limit of $0.5P'_s$ is adopted, the strength of a panel of design dimensions when subjected to fatigue loading can be assumed to be 11.56 kN (2.60 kips). If a prototype design wheel loads of 71.16 kN (16 kips) is assumed, the corresponding model design wheel load is 1112 N (250 lbs) and so the factor of safety against punching failure in fatigue is approximately 10. If a prototype design wheel load of 71 kN (16 kips) and an impact factor of 0.3 are assumed, the factor of safety is approximately 8. Clearly, conventional orthotropic bridge deck design is very conservative and can be expected to have indefinite life when subjected to repeated loads up to the design wheel

FIGURE 6: LOAD RATIO VERSUS LOG OF NUMBER OF CYCLES TO FAILURE FOR ORTHOTROPICALLY REINFORCED PANELS.



loads.

Tests of Panels with Isotropic Reinforcement

The results of the tests of panels with isotropic reinforcement are given in Tables 4 and 5. Table 4 shows the results of tests of plain concrete panels and of panels with top and bottom reinforcement. Table 5 shows the results of tests of panels with mid-depth reinforcement. In each case, either the effective depth or the thickness of the slab is given, so that the possible errors in the experimental load ratios can be assessed.

There are not sufficient results to permit a reliable correction of load ratios with variation in slab thickness for any slab panels except those with 0.2 percent reinforcement. This is the amount of isotropic reinforcement recommended in Project Q50 (3,4) for the type of bridge slabs under consideration. The following equation was proposed for calculating static ultimate strength (P_s') of panels with 0.2 percent isotropic reinforcement:

$$P_s' = -51.650 + 92.674d \text{ (kN)} \qquad (3)$$

The ratios of maximum fatigue load to the estimated static load P_f/P_s' for panels with 0.2 percent isotropic reinforcement are shown bracketed in Table 4. It can be seen that there was little variation in the effective depths of these panels, therefore correction would not have significantly affected the experimental load ratios.

Table 4 indicates that the endurance limit appears to increase with the reinforcement percentage, and for panels with 0.4 and 0.6 percent reinforcement an endurance limit of 0.5, i.e. the same as that for orthotropically reinforced panels, could be adopted. However, with the very high factors of safety against static punching failure in mind, the tests of the panels with lower reinforcement percentages are of particular interest. Consequently, allowing for the scatter of the test results and errors in the experimental load ratios, it is recommended that an endurance limit of 0.4 be accepted. Thus, a fatigue load factor of 2.5 may be adopted. This value appears adequate for unreinforced panels and for panels with 0.2 percent top and bottom isotropic reinforcement.

On the basis of ultimate strength and shrinkage and temperature reinforcement requirements, 0.2 percent isotropic reinforcement has been recommended (3,4) as the minimum reinforcement for the deck slabs of composite steel/concrete bridges. The static tests of panels with 0.2 percent reinforcement have indicated that a 1/8th scale model of a panel of design dimensions has a static strength of 13.66 kN (3.07 kips). If an endurance limit of 0.4 is adopted, the strength of a panel of design dimensions when subjected to fatigue loading can be assumed to be 5.47 kN (1.23 kips). Thus, if a prototype design wheel load of 71 kN (16 kips) is assumed, the factor of safety against punching failure in fatigue is approximately 5 for a life of at least 2,000,000 cycles. If a design wheel load of 71 kN (16 kips) and an impact factor of 0.3 are assumed, the factor of safety is approximately 4, assuming a load scale factor of 64 for the 1/8th scale models.

Table 5 shows that increase of mid-depth reinforcement did not significantly increase either the static strength or the endurance limit of a panel, although the static strengths of panels with mid-depth reinforcement were higher than those of panels without reinforcement. An endurance limit of 0.4 also appears adequate for panels with mid-depth reinforcement.

Panel 9 of bridge number 5, which has been left unreinforced, was subjected to over 6,000,000 cycles of fatigue loading. The maximum fatigue load was progressively increased until failure occurred. The results of this test are given in Table 6. Noting the maximum fatigue loads expressed as multiples of the scaled design wheel load, and the accumulated cycles of loading, it is clear that in practical situations fatigue failure of composite bridge deck panels of the types studied is extremely unlikely.

Reserve Static Strengths

Any panels that did not fail under fatigue loading were statically tested to determine the panel reserve static strength P_r. Details of the fatigue tests and the reserve strengths of these slabs are given in Table 7. Again the ratios of reserve strength to static strength, i.e. P_r/P_s, can only be considered to be approximate because of probable errors in the assumed static strengths. However, it can be seen that, generally, there was little loss of strength of a panel after it had withstood 2,000,000 or more cycles of fatigue loading. This suggests that if failure under the same load has not occurred in less than 2,000,000 cycles, then fatigue life under the same load function is likely to be infinite. In all cases the maximum fatigue load was well in excess of the scaled design wheel load of 1112 N (250 lb).

Conclusions and Recommendations for Design

Deck slabs of steel/concrete composite bridges have large reserves of strength against fatigue failure. The endurance limit for conventionally reinforced slabs can be assumed to be 0.5. The endurance limit for slabs with less reinforcement than that in conventional slabs can be assumed to be 0.4. This holds for even unreinforced slabs.

On the basis of ultimate strength and shrinkage and temperature requirements, 0.2 percent isotropic reinforcement has been

Table 4. Results of tests of panels with top and bottom isotropic reinforcement

Nominal percentage reinforcement (p)	Bridge no.	Panel no.	Maximum fatigue load P_f (kN)	$\frac{P_f}{P_s}$	No. of cycles to failure	d (mm)	t (mm)	Mode of failure P-Punching F-Flexural
Zero	3	7	6.67	0.51	2,000,000*	-	22.1	-
		8	8.01	0.61	1,326,000	-	22.7	P
		9	13.08**	1.0	1	-	22.8	P
	4	4	11.56**	1.0	1	-	21.4	F
		5	8.10	0.70	17,740	-	22.5	P
		6	6.94	0.60	2,029,240*	-	20.9	-
	5	7	6.09	0.50	682,940	-	21.4	F
		8	12.19**	1.0	1	-	20.9	P
		9	2.67	0.22	2,490,000*	-	21.2	-
0.2	3	5	7.78	0.50 (0.44)	2,000,000*	19.1	-	-
		6	15.57**	1.0	1	19.2	-	P
	4	10	13.92**	1.0	1	18.0	-	F
		11	9.74	0.70 (0.70)	244,580	17.9	-	F
		12	8.36	0.60 (0.60)	235,840	17.9	-	F
0.4	3	1	8.67	0.50	2,155,660*	20.0	-	-
		2	10.41	0.60	3,260	20.6	-	P
		3	17.35**	1.0	1	20.4	-	P
	4	1	8.90	0.50	2,000,000*	17.4	-	-
		2	12.45	0.70	266,000	18.2	-	P
		3	17.75**	1.0	1	18.1	-	P
0.6	3	10	22.24**	1.0	1	20.1	-	P
		11	11.12	0.50	2,000,000*	20.0	-	-
		12	13.34	0.60	425,450	20.0	-	P
					Design	17.9	22.2	

Note: 1 mm = 0.039 in
1 kN = 0.225 kip

* Did not fail under fatigue loading
** Assumed static strength (P_s)

Table 5. Results of tests of panels with mid-depth isotropic reinforcement

Reinforcement spacing (mm)	Bridge no.	Panel no.	Maximum fatigue load P_f (kN)*	$\frac{P_f}{P_s}$	No. of cycles to failure	t (mm)	Mode of failure P-Punching F-Flexure
76	5	4	10.81	0.60	77,020	21.7	P
		5	18.01**	1.0	1	21.4	P
		6	9.03	0.50	822,250	21.9	F
51	4	7	11.83	0.70	45,030	21.6	P
		8	10.14	0.60	453,338	21.7	P
		9	16.90**	1.0	1	22.0	P
25	5	1	9.34	0.50	507,150	21.4	F
		2	18.68**	1.0	1	21.7	P
	5	10	7.12	0.40	2,000,000*	21.9	-
		11	17.79**	1.0	1	21.5	P
		12	8.90	0.50	817,420	21.3	F
					Design	22.2	

Note: 1 mm = 0.039 in
1 kN = 0.225 kip

* Did not fail under fatigue loading
** Assumed static strength P_s

Table 6. Fatigue test of panel no. 9 of bridge no. 5 (zero reinforcement)

Maximum fatigue load P_f (kN)	$\frac{P_f}{P_s}$	Multiples of design wheel load	No. of cycles at load level	Total no. of cycles	Comments
2.67	0.22	2.4	2,490,000	2,490,000	
3.27	0.27	2.9	1,999,940	4,489,940	
3.87	0.32	3.5	550,000	5,039,940	
4.94	0.40	4.4	1,070,000	6,109,940	
6.09	0.50	5.5	240,000	6,349,940	Failure

Table 7. Reserve strengths of slabs

Reinforcement	Bridge no.	Panel no.	$\frac{P_f}{P_s}$	No. of cycles of loading	Reserve static strength P_r (kN)	$\frac{P_r}{P_s}$
Orthotropic						
	1	2	0.39	2,050,000	22.24	0.95
	2	11	0.50	3,040,000	17.79	0.87
Isotropic						
None	3	7	0.51	2,000,000	14.06	1.07
	4	6	0.60	2,030,000	12.85	1.11
0.2%	3	5	0.50	2,000,000	13.12	0.84
0.4%	3	1	0.50	2,160,000	18.33	1.06
	4	1	0.50	2,000,000	12.85	0.72
0.6%	3	11	0.50	2,000,000	21.93	0.99
Mid-Depth						
25.4 mm	5	10	0.40	2,000,000	17.79	1.00

Note: 1 mm = 0.039 in
1 kN = 0.225 kip

recommended as the minimum amount required. The fatigue tests have confirmed that this is adequate and that considerable strength against fatigue failure is ensured by such reinforcement, together with the inherent slab boundary restraint due to the composite bridge substructure of beams, diaphragms and shear connectors. An endurance limit of 0.4, and therefore a fatigue load factor of 2.5, should be adopted in design with this type of reinforcement.

Although mid-depth reinforcement offers the advantage of maximum cover, adoption of this type of reinforcement is not recommended since it does not satisfy the conventional temperature and shrinkage reinforcement requirements. However, if mid-depth reinforcement is used, then, for ultimate strength determination, the slab should be assumed to be unreinforced and an endurance limit of 0.4 and a fatigue load factor of 2.5 can be adopted.

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References

1. G.M. Nordby. Fatigue of Concrete - A Review of Research. Proceedings of the American Concrete Institute, Vol. 55, August 1958, pp. 191-215.
2. C.Y. Loo. The Behaviour of Reinforced Concrete Slabs Subjected to Concentrated Loads Repeated Many Times. M.Sc. Thesis, Queen's University, Kingston, Ontario, 1968.
3. B. deV. Batchelor and B.E. Hewitt. An Investigation of the Ultimate Strength of Deck Slabs of Composite Steel/Concrete Bridges. Research Report Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1976.
4. B.E. Hewitt. An Investigation of the Punching Strength of Restrained Slabs with Particular Reference to the Deck Slabs of Composite I-Beam Bridges. Ph.D. Thesis, Queen's University, Kingston, Ontario, 1972.
5. B. deV. Batchelor and B.E. Hewitt. Direct Model Studies of Some Reinforced Concrete Structures. Proceedings of the First Australian Conference on Structural Models, University of Sydney, 1972.
6. E.B. Dixon. Fatigue Behaviour in Slabs of a Composite Steel/Concrete Bridge. M.Sc. Report, Queen's University, Kingston, Ontario, 1970.
7. American Association of State Highway Officials. Standard Specifications for Highway Bridges. (10th edition), 1969.

Notation

d	effective depth of concrete slab, mm
f'_c	concrete compressive strength, MPa
f_t	concrete tensile strength, MPa
P_f	maximum load of fatigue load function, kN
P_r	reserve static strength, kN
P_s	assumed static strength, kN
P'_s	estimated static strength, kN
p	percentage of reinforcement
N	log. of number of cycles
S	stress ratio
t	slab thickness, mm