

INVESTIGATION OF THE ULTIMATE STRENGTH OF DECK SLABS OF COMPOSITE STEEL/CONCRETE BRIDGES

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A theoretical and experimental study of the ultimate strength of the deck slab of composite I-beam bridges is summarized. A theory, based on a mechanical model proposed by Kinnunen and Nylander, for punching failure of simply supported slabs, is developed which permits the calculation of the punching strength of restrained slabs. The theory suggests that a deck slab can be expected to have a high inherent strength due to boundary restraints ensured by the presence of shear connectors, beams, diaphragms and the neighbouring slab areas. One-eighth scale direct models of a 24.4 m (80 ft) span prototype bridge were tested in laboratory studies of both orthotropically and isotropically reinforced slabs. Shear connector behaviour and dead load stresses appropriate to unshored construction were simulated. The results of the tests show that conventionally reinforced deck slabs have very high factors of safety against failure by punching and are wastefully reinforced. From considerations of ultimate strength as well as shrinkage and temperature reinforcement requirements, 0.2 per cent isotropic reinforcement is recommended as being adequate for bridge slabs of the type studied. Although this amounts to approximately 30 per cent of the current reinforcement requirements for such slabs, a high factor of safety can still be expected.

Background

The research described in this paper was undertaken to investigate the ultimate strength of the concrete deck slabs of composite steel/concrete bridges under concentrated loads. It was considered that the inherent restraint of slabs of structures of this form could result in enhancement of load carrying capacity, and if this were considered in design, the reinforcement of the slab could be reduced. This report is a summary of an extensive investigation carried out at Queen's University (1).

It is current practice (2,3) to design the deck slab of a composite bridge by assuming a wheel load

to be distributed over transverse slab strips of unit width, that are perpendicular to the direction of the traffic. After approximating the maximum moments to be resisted, the slab strips are designed as if they were concrete beams. It is understood that slabs designed in this manner can be regarded as being safe against shear type failures.

An investigation (1,4) sponsored by the Ontario Ministry of Transportation and Communications, has confirmed that the conventional design of deck slabs of composite bridges is very conservative, and that even with considerable reduction of the slab reinforcement, satisfactory factors of safety against failure by punching can be anticipated. Similar high strengths for restrained slabs have been noted by researchers including Taylor and Hayes (5) and Batchelor and Tong (6).

It is shown in this report that, by considering the mechanism of failure and following an ultimate strength method of design rather than an elastic approach, an adequate and more economical slab design can be achieved.

Outline of Theory

The idealized model of failure proposed by Kinnunen and Nylander (7) has been proven to give a good estimate of the punching strength of simply supported slabs (1). The punching shear strength of restrained slabs has been investigated (1,4) by modifying Kinnunen and Nylander's theory to incorporate a boundary restraining force, F_b , and a boundary restraining moment, M_b . Both F_b and M_b are per unit length of slab, and act at the level of the tensile reinforcement at the boundary. The idealized mechanical model adopted for a restrained slab at punching failure is shown in Figure 1.

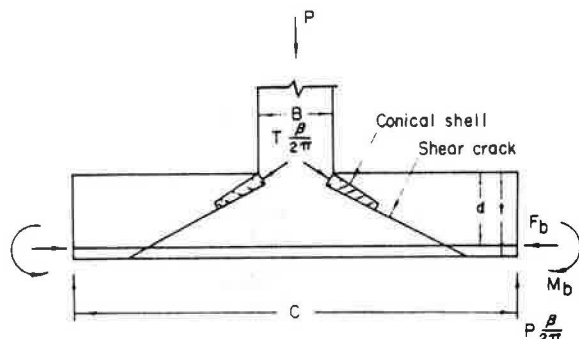
In Fig. 1, the outer portion of the slab, which is bounded by the shear crack and by radial cracks, is considered to be loaded through a compressed conical shell that develops from the perimeter of the loaded area to the root of the shear crack. The conical shell is assumed to have the shape shown in Figure 1a, and its thickness is assumed to vary in such a manner that the

compressive stresses at the intersection with the column and at the root of the shear crack are approximately equal.

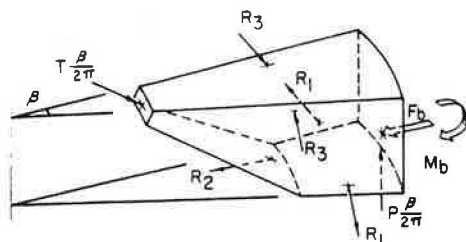
The sector element shown in Figure 1b is acted upon by the external load $P\beta/2\pi$ and by the following forces which are caused by rotation:

1. The oblique compression force $T\beta/2\pi$ in the compressed conical shell.
2. Horizontal forces in the reinforcement at right angles to the radial cracks, with resultants R_1 .
3. Horizontal forces in the reinforcement crossing the shear crack, with resultants R_2 .
4. Horizontal tangential compressive forces in the concrete, with resultants R_3 .
5. The boundary restraints F_b and M_b acting at the level of the tensile reinforcement at the boundary.

Figure 1. Mechanical model of a slab with boundary restraints at punching failure.



(a) SECTION SHOWING BOUNDARY FORCES



(b) SECTOR ELEMENT SHOWING SLAB FORCES

The criterion of failure is that punching occurs when the tangential strain at the top surface of the slab in the vicinity of the root of the shear crack reaches a critical value. By considering the equilibrium of the sector element and adopting an empirical criterion of failure used by Kinnunen and Nylander, the theoretical punching load, P , can be determined in an iterative process using a computer program (1) developed for this purpose. This theoretical punching load can then be corrected for the dowel effect to give the ultimate punching load, V , of the restrained slab.

During the analysis of reported shear tests, it became apparent that the theory of Kinnunen and Nylander was unreliable for estimating the punching shear strengths of slabs of extreme dimensions and material properties. It was found that, for slabs with any of the parameter functions outside the following limits, the theory generally gave inaccurate estimates of the punching load or because of limited test results, the theory could not be verified. These limits are:

Parameter function	Limits
C/d	4 - 17
q	0.05 - 0.45
E_{steel}	186 - 228 GPa (27×10^6 - 33×10^6 psi)
γ	4 - 20

The equations used in computing the theoretical punching load have been derived and presented in Reference (4).

Influence of Boundary Restraints

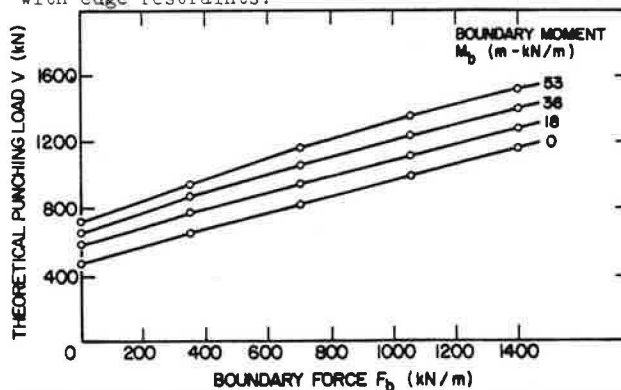
A hypothetical slab was analysed in order to demonstrate the influence of variations of the boundary restraint on the punching load of a restrained slab. The assumed dimensions and properties of the slab were as follows:

Slab thickness (t)	178 mm (7 in.)
Slab effective depth (d)	140 mm (5.5 in.)
Diameter of the loaded area (B)	305 mm (12 in.)
Slab diameter (C)	1.83 m (72 in.)
Reinforcement Ratio (p)	1.0 per cent
Yield stress of reinforcement (f_y)	310 MPa (45 ksi)
Compressive strength of concrete (f'_c)	34.5 MPa (5.0 ksi)

The boundary force was varied from 0 to 1400 kN/m (0 to 8,000 lb/in.), and the boundary moment from 0 to 53.3 kN.m/m (0 to 12,000 lb.in/in.). The theoretical punching load is plotted against the boundary force in Figure 2 for various boundary moments. The theory evidently suggests that considerable increase of the punching shear strength of a slab can result from boundary restraining forces.

Few punching tests of restrained slabs have been reported in which the magnitudes of the restraining forces were known or could be inferred. However, tests of prestressed concrete slabs by Scordelis, Lin and May (8) have permitted a limited investigation of the accuracy of the punching load calculated using the theory proposed for restrained slabs. Seven prestressed concrete slabs were analysed assuming the prestressing cable forces to be boundary restraining forces acting in the plane of the slab. Estimates of the dowel and tensile membrane effects were made for each slab and the influence of the unbonded prestressing cables was considered. The ratio of the test load to the calculated load was found to

Figure 2. Variation of theoretical punching load with edge restraints.



have a mean value of 1.01 and a standard deviation of 0.044. Considering the assumptions made in estimating the dowel and tensile membrane effects

and in assessing the influence of the unbonded prestressing cables, the calculated punching loads were remarkably consistent and accurate. Using an empirical approach, Scordelis, Lin and May obtained ratios with a mean value of 1.02 and a standard deviation of 0.075. A more in-depth comparison is given in Reference (4). Additional comparisons of tests by others are provided in Reference 1.

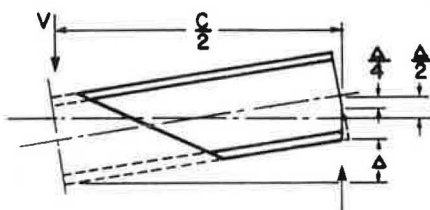
Restraint Factor

In practical situations the boundary restraining forces on a slab loaded to punching failure are usually not known and would be difficult to calculate or to measure accurately. In a slab such as the deck slab of a composite bridge, the difficulty is further increased by the likelihood of varying support and boundary restraint conditions at adjacent boundaries. For these reasons a single factor, F_r , to be termed the 'restraint factor', is proposed which expresses the slab strength enhancement due to practical boundary conditions. This factor can be used for the calculation of the punching load of restrained slabs by means of the largely rational approach already described, though it must be noted that the factor is itself empirical.

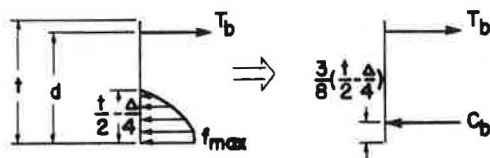
The idealized geometry of displacement of a slab at failure, as used by Brothie and Holley (9), and the resultant maximum boundary stresses and forces are given in Fig. 3. The maximum force, T_b , per unit length of the boundary in the tensile reinforcement at the boundary is given by:

$$T_b = d p f_{sy} \quad (1)$$

Figure 3. Idealised displacement and maximum boundary forces for a restraint slab.



(a) GEOMETRY OF DISPLACEMENT



$$T_b = d p f_{sy} \text{ kN/m}$$

$$C_b = k f_{max} \left(\frac{t}{2} - \frac{\Delta}{4} \right) \text{ kN/m}$$

(b) ASSUMED MAXIMUM BOUNDARY STRESSES AND FORCES

T_b is zero if reinforcement is absent or discontinuous at the boundary. The maximum compressive force, C_b , in the concrete per unit length of the boundary is given by:

$$C_b = k f_{max} \left(\frac{t}{2} - \frac{\Delta}{4} \right) \quad (2)$$

in which k is the ratio of the average stress to the maximum stress and depends on the stress

distribution.

If a parabolic distribution of stress is assumed, then k is $2/3$ and f_{max} is assumed to be $0.85 f'_c$. The idealized maximum boundary restraints are then given by:

$$M_{b(max)} = T_b (2d - t) - C_b \left(d - \frac{13}{16} t - \frac{3}{32} \Delta \right) \quad (3)$$

and

$$F_{b(max)} = C_b - T_b \quad (4)$$

A value of $F_r < 1.0$ was introduced to take account of the fact that the maximum boundary restraints would rarely be attained at failure. The actual boundary restraints at failure by punching are then given by:

$$M_b = F_r M_{b(max)} \quad (5)$$

and

$$F_b = F_r F_{b(max)} \quad (6)$$

It is not implied that the distribution of stress at the boundaries and the actual boundary restraints at the instant of punching are known. It is a fact, however, that F_r varies from zero for the simply supported slabs usually tested in investigations of punching, to unity for slabs with a full edge restraint, therefore, F_r must lie between 0 and 1 for all practical cases of restrained slabs. The restraint factor will depend on the properties of the slab as well as on the confining structure and can be determined empirically for a particular slab system.

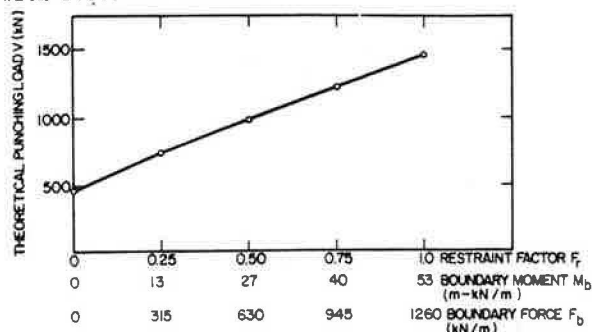
After calculation of $M_{b(max)}$ and $F_{b(max)}$, F_r can be determined for any type of restrained slab for which the punching load is known. The lower limit of this factor can then be used in determining punching loads for design purposes. The values of $M_{b(max)}$ and $F_{b(max)}$ are dependent upon the deflection, Δ , and consequently must be calculated iteratively. A computer program has been developed (1) which calculates the punching load of restrained slabs for values of F_r varying from zero to unity in steps of 0.25.

Influence of Restraint Factor

The typical slab previously described was analysed in order to demonstrate the influence of the restraint factor on the punching load of a slab. The calculated punching load is plotted against the restraint factor in Fig. 4. The boundary moment and force for each restraint factor from 0 to 1, the calculated punching load increases through the full range of feasible punching loads even though actual values of the boundary moment and force are not necessarily used.

If the punching load of a restrained slab is known, the restraint factor can be determined by interpolation after calculating the punching load for given values of the restraint factor. Slabs tested by Taylor and Hayes (5), Aoki and Seki (10) and others have been investigated in this way. It was found that the restraint factor of a slab with no tension reinforcement at the boundary generally decreases as the reinforcement index, q , ($= p f_{sy} / f'_c$) increases. As a rule, with all variables constant except the reinforcing ratio,

Figure 4. Variation of theoretical punching load with restraint factor.



a slab with low reinforcement ratio will have deflected more at failure than one having a high reinforcement ratio. Consequently, the boundary restraining forces, which are dependent upon the slab deflection, are likely to be nearer their maximum values in the restrained slab with the lower reinforcing ratio, and would therefore have the higher restraint factor. Although there were limited available test results, the following can be tentatively suggested to apply to slabs with this form of restraint.

$$\text{For } \frac{C}{B} \leq 6.0 \text{ and } q = 0.1, F_r = 0.50,$$

and

$$\text{For } 6.0 < \frac{C}{B} \leq 9.0 \text{ and } q = 0.2, F_r = 0.25$$

Tests and Observations

The behaviour of I-beam bridge slabs was investigated by testing a total of nine 1/8th scale direct models of 24.4 m (80 ft) span four-beam and three-beam bridges. Apart from the usual difficulties of small scale modelling, particular problems arose in the accurate modelling of all section properties, stud shear connector behaviour and dead load stresses appropriate to unshored construction. The solutions to these problems are described in detail elsewhere (1,11). The four-beam prototype and model bridges are detailed in Figure 5. A view of a four-beam model bridge superstructure is given in Figure 6. The three-beam model was similar to the four-beam model except that the beam spacing

Figure 5. Details of prototype four-girder bridges

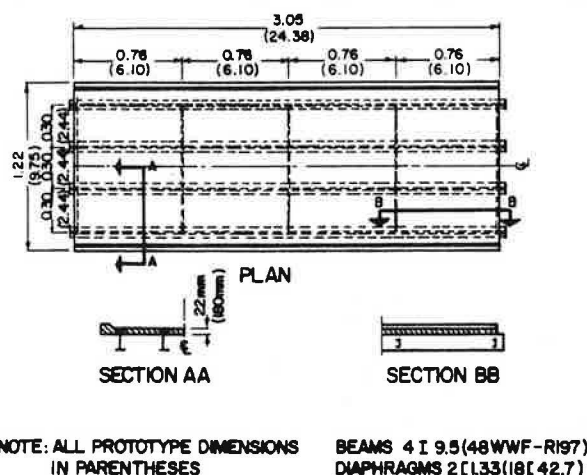
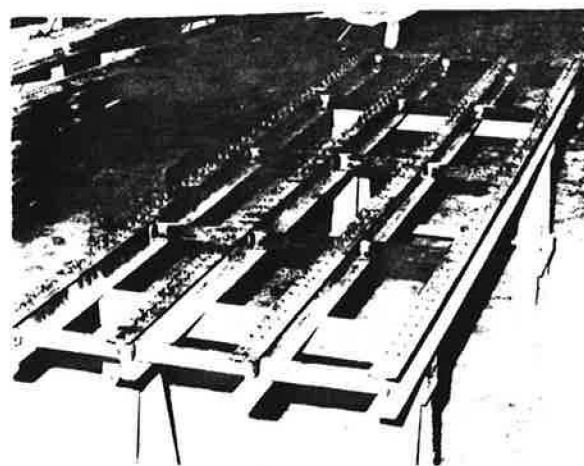


Figure 6. View of four-beam model



was 460 mm (18 in.) and the beams were supplemented with bottom flange plates.

The deck slabs of the bridges were tested to failure under single concentrated loads applied mid-way between adjacent bridge beams through a steel plate bearing on a neoprene pad. The contact area modelled an ellipse with major and minor axes of 760 mm (30 in.) and 510 mm (20 in.), respectively, which represents the assumed contact area of the pneumatic tires of large earth moving equipment. A view of a model bridge and the testing arrangement is given in Figure 7.

Deck slabs with orthotropic and with isotropic reinforcement, as well as plain concrete slabs, were tested. The orthotropic reinforcement modelled the reinforcement of conventional deck slabs. Slabs with isotropic reinforcement at mid-depth of the slab, and therefore with maximum cover, were also tested. The influences of slab span and thickness, load position, dead load stresses, reinforcement ratio, and concrete strength on the ultimate strength of the deck slabs were studied, together with the punching strength in a hogging moment region.

The model bridges tested are numbered 1 to 9. A bridge panel is defined as that portion of the deck bounded by adjacent bridge beams and adjacent diaphragms, and panels across a bridge are referred to collectively as strips. The bridges and the tests are described generally in Table 1. Bridges numbered 1 to 8 were four-beam bridges, bridge No. 9 was a three-beam bridge.

A total of 68 tests to failure were carried out. All but one of the reinforced panels and some of the unreinforced panels failed by punching. Failure by punching usually left a neat elliptical hole, a little larger than the loaded area, in the top of the slab and a pushed-through plug of concrete in the form of a frustum of a cone with an approximately circular base. The cracking patterns were similar for the slabs of the four-beam and three-beam bridges.

As the load was applied, visible cracking in the form of longitudinal and diagonal cracks was observed on the underside of the slab. This occurred directly beneath the loaded area and usually commenced at a load of between 25 and 50 percent of the failure load. Subsequently, transverse cracks appeared on the upper-side of the slab on either side of the loaded area in the vicinity of the adjacent bridge beams. As failure approached, the underside cracking developed into a complete pattern of cracks radiating from beneath

Figure 7. View of testing arrangement

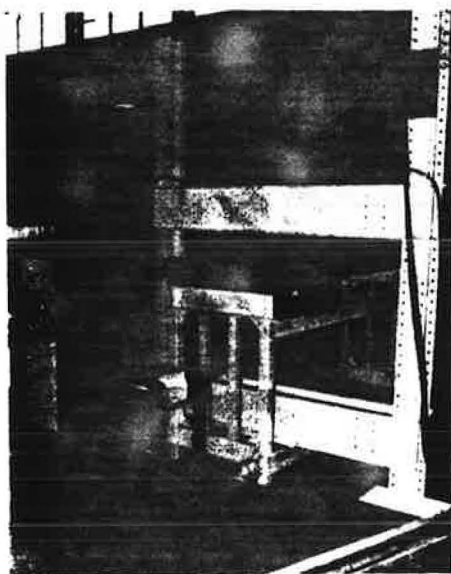


Table 1. Models constructed

Bridge number	Reinforcement*				Special Tests
	Strip 1	Strip 2	Strip 3	Strip 4	
1	ORT	ORT	ORT	ORT	
2	ORT	ORT	ORT	ORT	Full dead load compensation
3	ORT	ORT	ORT	ORT	Some panels with dead load compensation
4	ORT	ORT	ORT	ORT	Full dead load compensation
5	ORT	zero	0.4%	0.2%	
6	0.6%	0.2%	0.6%	M	Hogging moment in strips 2 and 3
7	0.4%	0.2%	zero	0.6%	
8	0.4%	zero	M	0.2%	
9	0.4%	0.2%	zero	0.6%	

Note: ORT indicates orthotropic reinforcement
 0.6 }
 0.4 } - is percentage of isotropic reinforcement
 0.2 }
 M indicates mid-depth isotropic reinforcement

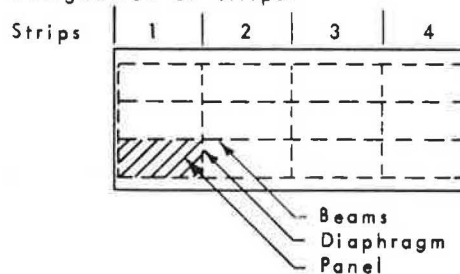
the loaded area, and the upperside longitudinal cracks lengthened and curved around the loaded area. Prior to failure, the cracking pattern indicated imminent flexural failure in an elliptical fan mechanism and never suggested failure by punching. Although accelerated creep usually gave some warning, failure was always explosive. Cracking rarely extended into adjacent panels and, generally, the higher the percentage of reinforcement, the more symmetrical and closely spaced was the cracking. Views of the cracking pattern of a four-beam bridge panel with 0.2 percent isotropic reinforcement after failure by punching, are given in Figure 8.

Some unreinforced panels and one panel with a low percentage of reinforcement failed in flexure in elliptical fan mechanisms. Up to failure the cracking behaviour was similar to that for all other slabs but failure occurred after much creep

Figure 8. View of cracking pattern at failure of panel with 0.2% reinforcement



Designation of strips:

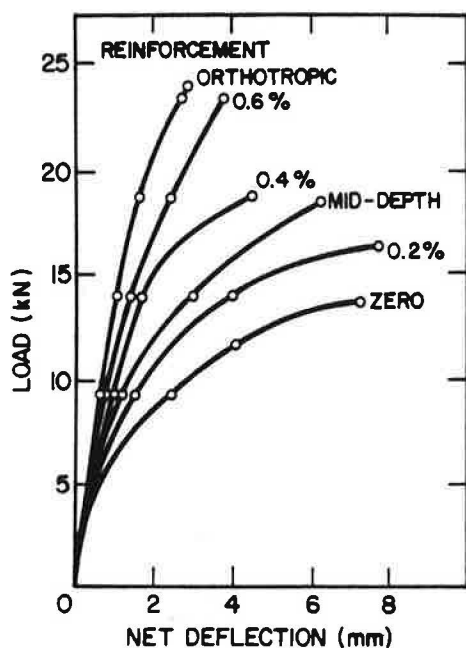


and with some crushing of the concrete on the upperside of the slabs along lines radiating from the loaded area. The major axis of the fan mechanism was aligned with the major axis of the loaded area, and the cracking pattern was contained within adjacent bridge beams.

Deflection Behaviour

Figure 9 shows typical variations of net slab deflection with load for a four-beam bridge. There was considerable scatter of the net slab deflections for panels of the same design but the tests plotted in Figure 9 were chosen to show the general trend of deflection behaviour. Panels with low percentages of reinforcement failed after relatively large deflections. Panels with 0.6 percent isotropic reinforcement behaved similarly

Figure 9. Typical variations of net slab deflection with load for four beam bridges



to panels with orthotropic reinforcement and failed after relatively small deflections.

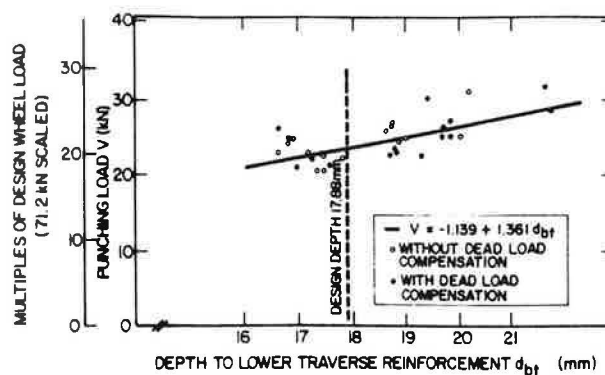
Punching Strength

Slabs with Orthotropic Reinforcement

There was considerable variation of the failure loads for slabs of the same design. A statistical analysis of all the results of tests of orthotropically reinforced panels indicated that the variation of failure load could be attributed to unavoidable variations of the slab dimensions. The analysis showed that the failure load is not significantly influenced by the position of the panel, by previous failures in adjacent panels, and by the strength of the concrete or dead load stresses.

Figure 10 is a plot of test punching load for orthotropically reinforced panels versus effective depth, d_{bt} , to the bottom transverse reinforcement and shows the regression line determined using all results. There was no significant difference between this regression line and those determined using only the results of tests with and without dead load compensation. It should not be inferred that the regression line can necessarily be extrapolated for slabs of properties outside the range of those which have been included in the analysis. The scale of multiples of design wheel load shows that the factor of safety against

Figure 10. Punching load vs depth to reinforcement-orthotropic reinforcement



failure by punching, ignoring impact, was never less than 17.

The punching load for a slab of design dimensions determined using the regression line, is 23.2 kN (5.21 kips). If a design wheel load of 71.2 kN (16 kips) is assumed, the factor of safety against punching is approximately 21. If a design wheel load of 71.2 kN (16 kips) and an impact factor of 0.3 are assumed the factor of safety is approximately 16. The bridge deck design is obviously very conservative.

In a few cases the load was applied at four points simultaneously to simulate a truck with wheels 1.83 m (6 ft) apart on axles 4.3 m (14 ft) apart. By measuring the extreme bottom fibre strains of the bridge beams it was shown that for a conventional I-beam bridge under truck loading, beam failure is to be expected well before slab failure. This is further indication that the bridge deck slab design is conservative.

Slabs with Isotropic Reinforcement

The average test punching loads for each nominal percentage of reinforcement and for panels of both four-beam and three-beam bridges are shown plotted against average reinforcement percentage in Figure 11. The average reinforcement percentages were calculated using the means of all

the measured effective depths of panels with the respective nominal reinforcement. Both flexural and punching failures occurred in four-beam bridge panels with 0.2 percent and with zero reinforcement, and it was assumed that flexural and punching strengths were the same for these panels. Only flexural failures, at an average load of 9.43 kN (2.12 kips), occurred in the unreinforced panels of the three-beam bridge and results of the tests of these panels were excluded from the analysis.

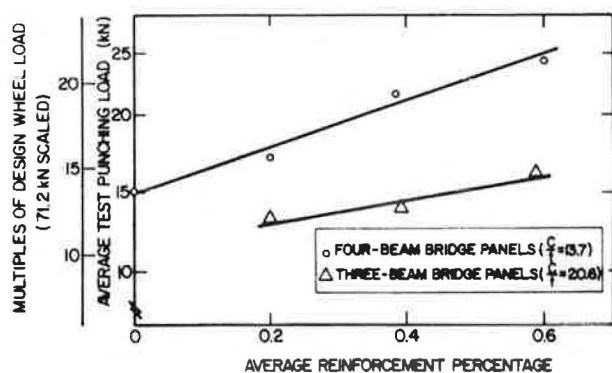
Figure 11 shows that the punching strength increases with increase of reinforcement and with decrease of span to thickness ratio (C/t). The scale of multiples of design wheel loads indicates that the factor of safety is always high, even when the span to depth ratio is unusually large, and that the factor of safety for a panel of a bridge of conventional dimensions would be approximately 13, if impact is ignored. There was some scatter of the test results but the factor of safety for unreinforced panels of four-beam bridges was never less than 10.

In the usual elastic design only the strength due to reinforcement is considered. Only in an ultimate strength design can both the strength due to the reinforcement and the very significant additive inherent strength of the unreinforced panel be utilised.

In tests on a model bridge in which hogging moment was induced, it was shown that such a moment has little, if any, effect on the punching load. It became apparent that extensive transverse cracking can be tolerated before any decrease in the punching strength of the bridge deck is to be anticipated.

The restraint factors for panels of the four-beam and three-beam bridges are given in Table 2. The theory was not considered to be applicable to the unreinforced panels of the three-beam bridges, which all failed in flexure, or to the panels with mid-depth reinforcement. The restraint factors here quantify the actual boundary restraints of the panels in terms of the idealized maximum restraints on the boundaries of the idealized equivalent slabs. It is seen that the restraint factors generally increase with increase of percentage reinforcement. This is contrary to

Figure 11. Load vs reinforcement percentage - isotropic reinforcement



the findings from the analysis of the tests of Taylor and Hayes (5) and Aoki and Seki (10) discussed previously, and is thought to be due largely to the fact that in bridge panels the top reinforcement may also contribute to the boundary restraint.

However, noting the reinforcing indices (q) and the diameter ratios (C/B) given in Table 2, it is seen that the limits of the restraint factor

which have been tentatively proposed, also apply here although the increase of restraint factor with increase of reinforcement is not utilised. With the high factors of safety, slabs with very low percentages of reinforcement are of interest, and for design purposes a restraint factor of 0.50 is proposed for use in the calculation of the ultimate punching strength of the deck slabs of composite I-beam bridges. This would be satisfactory for a slab with a span to thickness ratio as high as 20 and with a reinforcement percentage as low as 0.2.

Design Recommendations

In accordance with the philosophy of the ACI Code, the ultimate strength of a structure is expected to exceed about two and a half times the design live load. The factor of safety against failure by punching of an unreinforced panel of a four-beam bridge, assuming a wheel load of 71.2 kN (16 kips) and an impact factor of 0.3, is approximately 10. The study indicated that cracking under working loads is not a problem and that slab failure is highly unlikely, particularly since the bridge beams would probably fail first under truck loading.

Clearly, if only the punching strength is to be considered, reinforcement is theoretically not required in the deck slab of I-beam bridges. However, the AASHTO Standard Specifications for Highway Bridges (2) recommends that 'not less than 125 in.² (81 mm²) of reinforcement per foot shall be placed in each direction of all concrete surfaces to resist the formation of temperature and shrinkage cracks'. This amounts to approximately 0.2 percent reinforcement for 178 mm (7.0 in.) slab with 38 mm (1.5 in.) cover, and is here recommended as the maximum reinforcement required for the deck slabs of composite I-beam bridges. Using 0.2 percent isotropic reinforcement rather than the conventional orthotropic reinforcement as in the prototype bridge, the reinforcement requirement is reduced by about 66 percent.

The tests of panels with 0.2 percent reinforcement are therefore of particular interest. The test punching loads of the four-beam bridge panel with 0.2 percent reinforcement are plotted against the measured effective depths to the bottom transverse reinforcement in Figure 12. The line of regression of the punching load on the effective depth is shown. The scale of multiples of design wheel load shows that a slab of design dimensions has a factor of safety of approximately 12. Considering a wheel load of 71.2 kN (16 kips) and an impact factor of 0.3, the factor of safety is greater than 9.

Thus the design of the deck slab of I-beam bridges can be made very simple because only the AASHTO requirements regarding the temperature and shrinkage reinforcement need be satisfied. Assuming a restraint factor of 0.50 the ultimate strength of any slab with a span to depth ratio within the range of 14 to 21 of the tested panels can be estimated. Figure 13 shows a design curve, derived using the outlined theory, which relates punching loads and span to depth ratios of panels with 0.2 percent reinforcement. The punching loads were calculated assuming slabs of the following dimensions and properties:

Slab thickness	178 mm (7 in.)
Slab effective depth	127 mm (5 in.)
Diameter of loaded area	650 mm (25.6 in.)
Slab diameter	2.44 to 3.66 m (96 to 144 in.)

Reinforcement	0.2 percent
Yield stress of reinforcement	310 MPa (45 ksi)
Compressive strength of concrete	34 MPa (5000 psi)
Restraint factor	0.50

Table 2. Restraint factors for panels of four-beam and three-beam bridges

Four-beam Bridge Slabs					Three-beam Bridge Slabs			
C/B = 3.7					C/B = 3.6			
Isotropic Reinforcement (%)	Number of Tests	q (average)	V _{test} (average) (kN)	Restraint Factor	Number of Tests	q (average)	V _{test} (average) (kN)	Restraint Factor
0.6	7	0.055	23.66	0.74	2	0.054	16.46	0.59
0.4	5	0.036	21.57	0.66	2	0.036	14.14	0.52
0.2	7	0.018	18.01	0.60	2	0.018	13.48	0.55
zero	3	0	16.50	0.57	2 ^a	0	-	-

Note: a) Both slabs failed in flexure
1 kN = 0.225 kip

Figure 12. Punching load vs depth to reinforcement-0.2% isotropic reinforcement

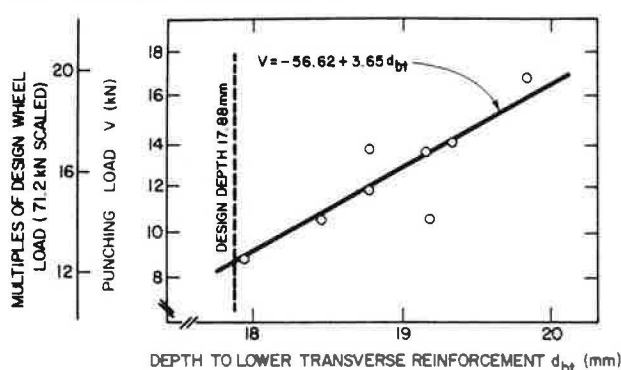
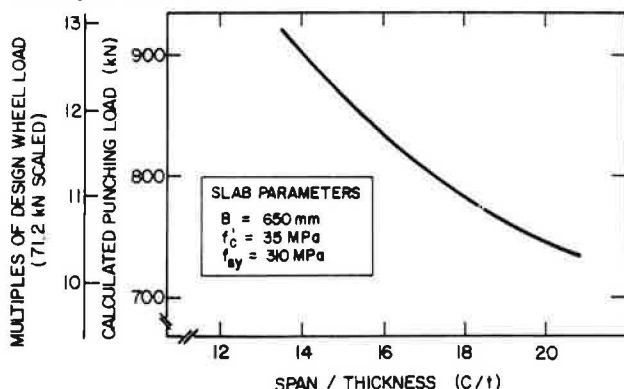


Figure 13. Calculated punching load vs span/thickness ratio with restraint factor of 0.5-0.2% - isotropic reinforcement



The loaded area was assumed to be circular and of the same perimeter as the assumed elliptical tire contact area. The slab diameter was taken as equal to the bridge beam spacing. The design curve could be used to check the ultimate strength and the factor of safety against failure by punching of any deck slab of comparable dimensions and with the required reinforcement of 0.2 percent.

The preceding discussion does not apply to the exterior cantilevering portions of I-beam bridge deck slabs which should be designed by conventional methods. The discussion applies only to panels with adequate boundary restraint the development of which is ensured by the shear connectors, bridge beams, diaphragms, and by the

continuity of the slab itself. According to the AASHTO Specifications, "the maximum pitch of shear connectors shall not exceed 610 mm (24 in.), except over the interior supports of continuous beams where wider spacings may be used to avoid placing connectors at locations of high stresses in the tension flange". The maximum spacing of the shear connectors in the model bridges was 102 mm (4 in.). It follows that theoretically for a prototype bridge, shear connectors at spacings of up to 810 mm (32 in.) provide adequate restraint, and some reduction in the factor of safety for the bridge deck could result from shear connector spacings in excess of 810 mm (32 in.). Diaphragms may need to be provided to prevent spread of the bridge beams due to in-plane forces in the loaded deck slab, and it appears that the AASHTO requirements are adequate. The end edges of the deck slab should always be stiffened.

Mid-depth reinforcement offers the advantage of ensuring maximum reinforcement cover but it does not satisfy the AASHTO requirements regarding temperature and shrinkage reinforcement. If, despite this, mid-depth reinforcement is used to resist the formation of temperature and shrinkage cracks, the ultimate strength of the slab could be conservatively estimated by assuming it to be unreinforced.

Conclusions and Recommendations for Further Research

Conclusions

The mechanical model used by Kinnunen and Nylander (7) can be modified to predict the punching strength of restrained slabs. This factor can be empirically determined for slabs of unknown boundary restraint.

The current method of design of the deck slab of a steel/concrete composite bridge results in the wasteful use of reinforcement. The factor of safety against failure by punching under a single wheel load can be expected to be approximately 16. Under truck loading, beam failure can be anticipated before slab failure. The punching strengths of such slabs vary inversely with span and directly with reinforcement ratio and effective depth.

The punching strength is not influenced significantly by the following factors:

1. The position of the load on the bridge deck slab.
2. Previous failures of the slab in adjacent

panels.

3. A hogging moment.
4. Dead load stresses and deflections.
5. The strength of the concrete of the slab.

It has been shown that, theoretically, no reinforcement is required in the deck slabs of composite I-beam bridges if only the ultimate strength of the designed structure is considered. In view of the AASHTO requirements regarding temperature and shrinkage reinforcement in mind, 0.2 percent isotropic top and bottom reinforcement is recommended as the maximum requirement. This amounts to a reduction of 66 percent of the current reinforcement requirements. For a deck of conventional dimensions with 0.2 percent reinforcement the factor of safety against failure by punching can be expected to be about 9.

A restraint factor of 0.50 should be used for the calculation of the punching strength for design purposes.

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