Load Capacity of Isotropically Reinforced, Cast-in-Place and Precast Panel Bridge Decks

I. K. Fang  
Assistant Professor  
of Civil Engineering  
National Cheng-Kung University  
Tainan, Taiwan  
Republic of China  
Former Graduate Research Assistant  
The University of Texas at Austin  
Austin, Texas

C. K. T. Tsui  
Bridge Division  
Texas State  
Department of Highways and Public Transportation  
The University of Texas at Austin  
Austin, Texas

N. H. Burns  
Zarrow Centennial Professor in Civil Engineering  
The University of Texas at Austin  
Austin, Texas

R. E. Klingner  
Phil M. Ferguson Professor in Civil Engineering  
The University of Texas at Austin  
Austin, Texas

A 20 x 50 (6.10 x 15.24 m), full scale bridge deck (both cast-in-place and precast panel), detailed using Texas SDHPT provisions for decks with Ontario-type isotropic reinforcement, was constructed in the laboratory. The ultimate load performance of the deck was investigated. Under single and tandem concentrated loads, the deck failed in punching shear. The ultimate concentrated load capacity of the deck was closely predicted by a proposed general punching shear model. Both the current ACI and AASHTO punching shear formulas were very conservative.

Recent research in the United States and Canada has suggested that the flexural capacity of bridge decks is increased by in-plane compressive forces, created when the cracked deck is restrained by supports that cannot move laterally. This phenomenon, commonly referred to as "arching action," is the basis for the semi-empirical design provisions of the current Ontario (Canada) Bridge Design Code. That code permits the use of less flexural steel than would be required by current AASHTO Specifications, resulting in bridge decks which are generally more economical and resistant to corrosion.

Previous research on arching action has been carried out mainly using small scale models with artificial boundary conditions. The overall objective of this research project was to study the performance of full scale bridge decks designed taking arching...
action into account. Using a full scale model of a realistic prototype highway bridge, both cast-in-place and precast, prestressed panel decks were considered. Deck behavior under static and fatigue loads is discussed in other publications dealing with this project.  

DESCRIPTION OF SPECIMEN

As shown in Figs. 1 and 2, the test specimen was a full size composite bridge with a 71/2 in. (191 mm) thick concrete deck on three 36 in. (914 mm) deep steel W sections, spaced at 7 ft (2.13 m). Half the deck had two layers of isotropic reinforcement [#4 bars at 8 3/4 in. (222 mm)] designed in accordance with Texas SDHPT details for Ontario-type decks. The other half had 4 in. (102 mm) thick precast, prestressed panels which replaced the lower grid of reinforcement in the CIP deck. The test specimen was simply supported on a 40 ft (12.2 m) span.

The ready mix concrete used in the CIP deck had a 28-day design compressive strength of 3600 psi (25 MPa). The average compressive strength at 28 days was 4240 psi (29 MPa). All reinforcement used for the cast-in-place deck and the top mat of the panel deck came from a single heat, met the requirements of ASTM A615 for Grade 60 steel, and had an actual yield strength of 73 ksi (504 MPa).

The precast, transversely prestressed panels spanned between longitudinal girders. The panels were 6.5 ft (1.98 m) wide by either 7 or 8 ft (2.13 or 2.44 m) long. They were of 6000 psi (41 MPa) strength concrete, 4 in. (102 mm) thick, and were later covered by a 3 1/2 in. (89 mm) topping of CIP concrete. Prestressing strands were 3/8 in. (9.5 mm), 270 ksi (1862 MPa), stress-relieved, 7-wire strands.

EXPERIMENTAL INVESTIGATION

Test Setup and Loading Program

As shown in Fig. 1, concentrated load tests were carried out at four locations on the deck: NE, SE, NW and SW.
SW. Single load tests were conducted at the first two locations, and tandem load tests at the last two. The SE single load test was placed at a joint between precast panels, while the SW tandem loads were spaced on each side of a joint.

The loading apparatus for the concentrated load tests is shown in Fig. 3. Two W8x67 beams were connected by 1 in. (25.4 mm) thick plates bolted on their top and bottom flanges at 4 ft (1.22 m) apart to form the loading support. Holes were drilled in the top and bottom plates to allow the four Dywidag bars to go through and tie the loading reaction frame to the test floor. Two sets of double beams were placed 4 ft (1.27 m) apart in parallel and transferred the reaction to a stiffened W21x67 beam.

A hydraulic actuator was placed directly underneath the center of the frame for single load tests, as shown in Fig. 3. For the tandem load tests, two actuators were placed 4 ft (1.22 m) apart under the loading frame. The actuator reacted against the bridge deck; the loading frame, which was tied down to the floor, reacted against the 8 x 20 in. (203 x 508 mm) steel plate footprint resting on the deck, as shown in Fig. 3. The oil pressure of the hydraulic actuator was applied by a hydraulic hand pump.

The deck was loaded to failure in 10 kip (44.5 kN) increments. At each load stage, crack propagation and the deflections under the loading point were recorded. The loading frames were checked regularly to ensure that the loads were applied vertically.

**Instrumentation**

Because the effective actuator area was known, the loads were monitored using a 10,000 psi (69 MPa) pressure gage. For tandem load tests, the two...
Load-Deflection Behavior

Single Load Tests — As shown by the typical results of Fig. 4, the load-deflection curves for both the CIP and the panel decks are almost linear until the load reaches 60 kips (267 kN), about three times the AASHTO service wheel load of 20.8 kips (92.5 kN) (including an impact factor). The slab failed at 142 kips (632 kN), about seven times the service wheel load. For the panel deck, the curve stays essentially linear up to a load of about 90 kips (400 kN), about four times the service wheel load. The panel deck failed at 180 kips (801 kN), about nine times the service wheel load and 1.27 times the ultimate capacity of the CIP deck. The results correlated very well with those of Ref. 12.

The load-deflection curve for the panel deck remains linear up to a higher load than that of the CIP deck, and also has a slope about 1.6 times greater. This implies that the panel deck was stiffer, due to the higher strength of concrete and the presence of prestressing strands in the precast, prestressed panel. The CIP deck had more extensive flexural cracking than the panel deck, and was more flexible, as shown in Fig. 4.

Tandem Load Tests — As shown by the typical results of Fig. 5, deflections under the two individual loading points were measured in both the CIP and the panel decks. As shown in Fig. 5, the load-deflection curves at the two points almost coincide. This implies that the loads from the two actuators were almost equal, and that the loading frame was maintained vertical in both tests.

Once again, the curve for the panel deck is steeper than the one for the CIP deck. The curves become nonlinear at a load of about 50 kips (222 kN) for the CIP deck, and about 70 kips (311 kN) for the panel deck. Both values are slightly lower than those for the corresponding single load test. The CIP deck failed at a total load (both points) of about 204 kips (907 kN), approximately 1.4 times the ultimate single load capacity of the same CIP deck. The panel deck failed at a total load of 267 kips (1188 kN), approximately 1.5 times its ultimate single load capacity. In both cases, the tandem load capacity was less than twice the single load capacity, indicating that the areas affected by each loading point overlapped and interacted.

For both tandem load cases, the failure load for the panel deck was 1.28 times that of the CIP deck, very close to the corresponding ratio for the single-load case. This good correlation implies homogeneity of the deck material and consistency of the loading setup.
Cracking Behavior

Single Load Tests — The first crack at the bottom surface of the CIP deck (NE) was observed at the loading point, at a load of 30 kips (133 kN). As load increased, the cracks propagated longitudinally and reached the top flange of the girders, as shown in Fig. 6. The first top surface crack, 3.5 ft (1.07 m) away from the loading point, was recorded at a load of 90 kips (400 kN). The top cracks propagated much more slowly than the bottom ones. Failure occurred by punching shear. The failure surface intersected the top of the deck in the form of a rectangle around the perimeter of the loading plate.

As shown in Fig. 7, the first crack in the panel deck (SE) was observed on the bottom surface at the loaded point, at a load of 60 kips (267 kN). The top surface crack, which first formed at 140 kips (623 kN), was about 3.5 ft (1.07 m) from the center of the loading plate. These cracks propagated much more slowly and much less than those in the CIP deck. Failure again occurred by punching shear, similarly to the CIP deck. In both cases, the top cracks propagated past the web of the exterior girder at high load. As shown from the results of both tests, the prestressing strands in the panels, which produced compression in the bottom 4 in. (102 mm) of the deck, delayed crack formation much more than did the reinforcement in the CIP deck.

Tandem Load Tests — In the CIP deck, the first bottom surface crack formed at a load of 20 kips (89 kN) per actuator, between the two loading points in the CIP deck (Fig. 8). The first top surface crack was observed at a load of 50 kips (222 kN) per actuator, at a distance of 3.75 ft (1.14 m) from the south loading point. Both the top and bottom cracks propagated faster and more extensively than did those in the corresponding single load test. Some of the top cracks, shown in Fig. 8, extended 5 ft (1.52 m) past the web of the interior girder. Only the top surface under the north loading plate was punched in shear. At failure, a wide crack developed on the top surface between the loads. At failure, the widest crack was about 0.05 in. (1.27 mm) wide.

In the panel deck, the first crack formed at 50 kips (222 kN) per actuator on the bottom surface, and at 90 kips (400 kN) per actuator on the top surface. As in the corresponding single load test, cracks in the panel deck did not propagate as rapidly nor as far as did those in the CIP deck. The top cracks (Fig. 9) formed about 4 to 5 ft (1.22 to 1.52 m) from the perimeter of the loading zone. Once again, only the south loading plate caused a punching shear failure of the deck. At failure, a top surface crack formed between the loads, and was about 0.05 in. (1.27 mm) wide.

Fig. 8. Deck cracking from tandem load test, CIP end.

Fig. 9. Deck cracking from tandem load test, panel end.
From equilibrium principles, the punching shear capacity equals the sum of the vertical components of the ultimate tensile forces acting on the four inclined failure surfaces:

$$V_c = 2d^* \tan \theta$$

$$= \frac{(b_1 + b_2 + 2d^*/\tan \theta) f_t}{(1)}$$

where

- $d^*$ = average effective depth of section
- $V_c$ = nominal shear capacity due to concrete
- $\theta$ = angle of inclination of assumed failure surface, measured from horizontal
- $f_t$ = ultimate tensile capacity of concrete
- $b_1, b_2$ = short and long sides of concentrated load footprint (Fig. 10)

In carrying out this calculation, $f_t$ was estimated using Eq. (11-36) of ACI 318-83:

$$f_t = (2 + 4/\beta_c) \sqrt{f_c}$$

but not more than $4 \sqrt{f_c}$, where $\beta_c$ is the ratio of $b_2$ to $b_1$.

For the single load tests, the actual values of $b_1$ and $b_2$ were used. Because the slab was never actually punched through in any of the four tests, it was impossible to measure the real failure angles. However, in the single load tests, judging from the distance between the failure surfaces at top and bottom of the deck in each test, a failure angle of about 39 degrees appeared to be quite reasonable. That angle of 39 degrees, used in the equation for punching shear, also gave an excellent correlation between experimental and calculated results for both the CIP and panel decks.

As shown in Fig. 11, the actual crack patterns suggested that the two loads were actually acting like a line load of length $b_2$. For the tandem load case, therefore, $b_2$ was taken as the distance between the outside edges of the two loading plates. An angle of 38 degrees gave a reasonable correlation between the experimental and calculated results for both CIP and panel deck.

Results of the punching shear calculations are summarized in Figs. 12 through 15.

**Deck Capacity as Governed by Yield Line Theory Without Arching Action**

The observed cracking patterns were idealized by the yield line mechanisms of Figs. 16 and 17. In developing these mechanisms, the longitudinal dimension $l_x$ was taken as the average of the measured longitudinal distances $a$ and $b$. Also, the transverse dimension $l_y$ was measured between the inside edges of the top flanges of the girders. Flexural capacities along yield lines were calculated including the effects of both positive and negative moment resistances of the deck in each direction, and using actual material properties. No capacity reduction factor was applied. Small displacements were assumed, and internal work due to membrane stresses (arching action) was neglected.

The deck capacities predicted by this analysis method are also included in Figs. 12 through 15. Reflecting the fact that flexural failure did not govern in these tests, predicted flexural capacities were about twice the actual capacities for the single load tests, and about 1.8 times the actual capacities for the tandem load tests. This analysis assumed that the slab exhibited two-way action. However, in all of the actual tests, the crack pattern on the top surface never developed enough to show complete formation of crack as on a two-way slab. This suggests that the deck slabs of these tests did not behave entirely like two-way slabs.
Fig. 12. Comparison of analytical and experimental capacities, single load test, CIP end.

Fig. 13. Comparison of analytical and experimental capacities, single load test, panel end.

Fig. 14. Comparison of analytical and experimental capacities, tandem load test, CIP end.

Fig. 15. Comparison of analytical and experimental capacities, tandem load test, panel end.
Yield Line Theory With Arching Action Included (One-Way Slab Action)

As mentioned before, the flexural strength of a slab can significantly exceed the values predicted by yield line theory as above (neglecting the effect of arching action). Assuming one-way action in the transverse direction, an axial force-moment interaction diagram was developed for the slab, and is shown in Fig. 18. Using this interaction diagram, and assuming that the transverse membrane force in the cracked slab increased linearly with applied load, a modified ultimate flexural capacity \( m_{u,*} \) of the slab was obtained including the effect of arching action. As shown in Figs. 19 and 20, the observed cracking pattern corresponded to a one-way slab with the length of the crack taken as \( l_y \).

With \( m_{u,*} \) and the crack pattern shown in Figs. 19 and 20, the ultimate flexural capacity of the deck was calculated for each test. These results as well are presented in Figs. 12 through 15. A flexural resistance equal to only \( \frac{1}{4} m_{u,*} \) was assigned to the yield line adjacent to the exterior girder. This is due to the smaller arching forces acting there because of lower lateral restraint. The capacities from this analysis were from 1.4 to 2.1 times the failure loads, again reflecting the fact that flexural capacity did not govern in these tests.

CONCLUSIONS

1. Under single concentrated loads, the deck failed in punching shear. Results of the concentrated single load test in this study correlated very well with previous tests. Tandem load tests were also carried out, and the deck again failed by punching shear.
2. A general punching shear model closely predicted the deck capacity under both single and tandem loads. Yield line models correctly predicted that flexural capacities would not govern.
3. Both the ACI and AASHTO formulas for punching shear capacity were very conservative in estimating the load capacity of the deck.
4. Overall, the experimental pro-
program showed that the precast, prestressed panel deck was stronger, stiffer and more crack resistant than the cast-in-place deck.

**RECOMMENDATIONS FOR IMPLEMENTATION**

1. Based on the results of this study, composite decks with precast, prestressed panels are superior to cast-in-place concrete decks in terms of stiffness, crack resistance, flexural strength, and punching shear capacity. They should be considered seriously. The standard panels simply and efficiently replace the lower portion of the deck slab and the bottom layer of the two-level, Ontario-type deck reinforcement.

2. Current AASHTO Code procedures for computing punching shear resistance of bridge decks are conservative, and should be modified if used in design situations where deck thickness requirements are governed by calculated punching shear capacity. In this study, general punching shear models, based on a variable failure surface angle, were found to be very accurate, and should be considered for use.

**ACKNOWLEDGMENTS**

This research was funded by the Texas SDHPT (Project 3-5-80-350), in cooperation with the U.S. Department of Transportation (Federal Highway Administration), and was carried out under the auspices of the Center for Transportation Research of The University of Texas at Austin. Texas SDHPT contact was LeRoy Crawford, and the FHWA contact was Don Harley. The authors appreciate their support and advice. The experimental work was conducted in the Phil M. Ferguson Structural Engineering Laboratory at the Balcones Research Center of The University of Texas at Austin. The work of the Laboratory’s technical and administrative staff is gratefully acknowledged.

**METRIC(SI) CONVERSION FACTORS**

<table>
<thead>
<tr>
<th>U.S. Unit</th>
<th>SI Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 ft</td>
<td>0.305 m</td>
</tr>
<tr>
<td>1 in.</td>
<td>25.4 mm</td>
</tr>
<tr>
<td>1 kip</td>
<td>4.448 kN</td>
</tr>
<tr>
<td>1 ksi</td>
<td>6.895 MPa</td>
</tr>
<tr>
<td>1 psi</td>
<td>0.006895 MPa</td>
</tr>
</tbody>
</table>

**Fig. 18.** Moment-axial force interaction diagram for deck slab, showing increase in flexural capacity due to arching action.

**Fig. 19.** Yield line pattern used for single load case, arching action included, one way action.

**Fig. 20.** Yield line pattern used for tandem load case, arching action included, one way action.
REFERENCES


