

READER COMMENTS

Spandrel Beam Behavior and Design*

by Charles H. Rath

Comments by A. T. Krauklis and O. C. Guedelhoefer,
V. G. Naidu, and Author

**A. T. KRAUKLIS and
O. C. GUEDELHOEFER†**

This discussion is limited to a portion of the special report dealing with the design and analysis of ledges at a free end. The author discusses both unreinforced and reinforced ledges in his paper. Since the principal focus of the author was to present a design method, he necessarily made several conservative assumptions.

The observations made by these writers during both full-scale and reduced-scale tests of heavily loaded ledge elements are intended to provide additional information on the behavior of ledge type elements. A paper based on these tests is currently being prepared to treat the topic more fully. Other references provided at the end of this discussion provide additional information on the topic.¹⁰⁻¹³

The Unreinforced Ledge

This discussion for the unreinforced ledge is based on one model test conducted by these writers. The model test contained the following parameters:

$$\begin{aligned}l_p &= 6.00 \text{ in.} \\ b &= 4.67 \text{ in.} \\ d_e &= 3.00 \text{ in.} \\ h &= 9.50 \text{ in.}\end{aligned}$$

The load was applied 3.33 in. from the face of the stem. The ultimate force required to break the ledge was 17.5 kips.

Using Eq. (5.10-2) of the PCI Design Handbook, the predicted capacity of the ledge without ϕ is 21.6 kips. The use of this equation results in an overestimation of the actual ledge capacity by 23 percent. This unconservative margin of error does not appear to be acceptable for design criteria.

The use of Eq. (8) on p. 85 of the author's paper results in a ledge capacity without ϕ of only 14.8 kips. The use of this equation results in an underestimation of the actual ledge capacity by 15 percent. This may be an acceptable margin in a new design, however this underestimation could result in unnecessary condemnation or repair of an existing structure.

The Reinforced Ledge

This discussion for reinforced ledges is based on observations made during four full-scale load tests of free end ledges. The failure loads of all four tests were in the 500-kip range. A method of analysis was developed based on the results of these

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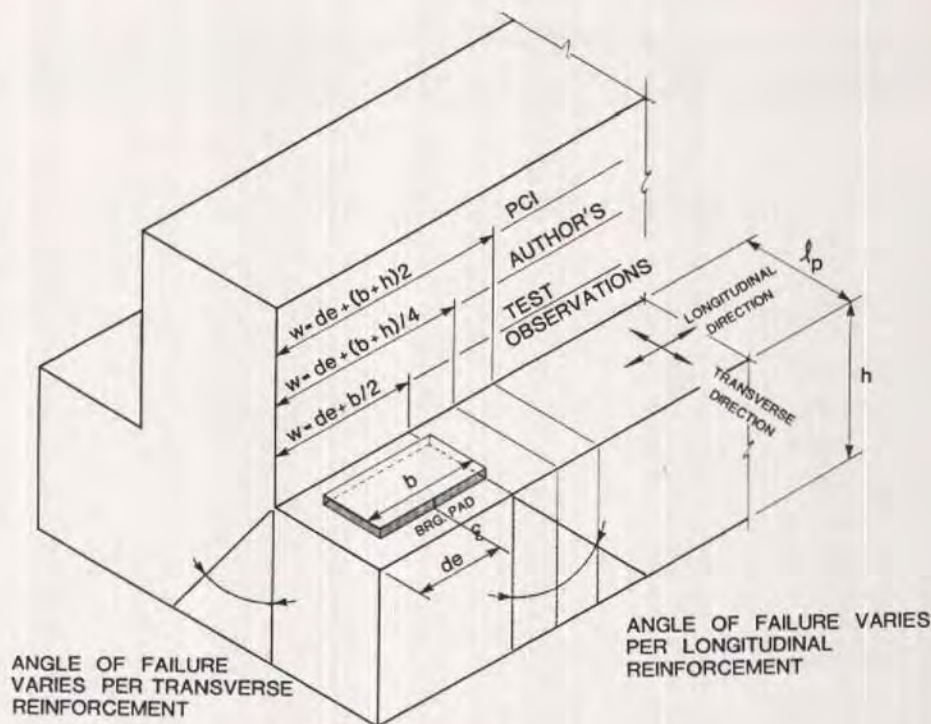


Fig. A. Comparison of failure planes in a spandrel beam.

tests and is presently being evaluated using a series of model studies. This method will be presented at a later date after completion of the model tests.

Failure Planes — In all four tests, the widths (w) of the failed ledges were narrower than those calculated by the method recommended in the PCI Design Handbook.

$$w = d_e + (b + h)/2$$

Three out of four tests exhibited failure widths smaller than those suggested by the author.

$$w = d_e + (b + h)/4$$

The minimum failure width resulted from an almost vertical failure plane that started at the inside edge of the bearing pad which is defined by:

$$w = d_e + b/2$$

A very distinct relationship was observed between the location of the failure plane and the amount and location of the reinforcement within the failure volume. The angle of the failure plane varies from vertical to a slope of 45 deg starting at the inside edge of the bearing pad in the longitudinal direction, and at the stem to ledge intersection in the transverse direction. The amount and the position of the reinforcement that crosses the crack plane controls the angle of the failure surfaces, mode of failure (shear friction or flexure), and the capacity.

A large amount of vertical reinforcement with a small amount of horizontal reinforcement results in a vertical crack close to 90 deg or a flexural failure. A large amount of horizontal reinforcement with a small amount of vertical reinforcement results in a sloping failure plane near 45 deg or a shear failure. Fig. A illustrates the different design widths and failure planes.

The PCI method considers reinforcement only transverse to the ledge; longitudinal ledge reinforcement is neglected. The author recognized the need for longitudinal reinforcement, but only added horizontal reinforcement (A_h). Vertical stirrups in the ledge adjacent to the bearing pad will significantly increase the shear capacity of the ledge. Both flexure and shear reinforcement should be balanced to efficiently maximize the ledge longitudinal capacities.

The equating of A_h reinforcement with half of A_{sh} seems arbitrary. The amount of load that is transmitted along the ledge longitudinally should be determined by a rational method. The portion of the vertical load that is transmitted along the ledge may be derived from the ratio of longitudinal ledge stiffness to the summation of the longitudinal ledge stiffness plus the transverse ledge stiffness. Once the amount of load that is transmitted longitudinally along the ledge is determined, the proper reinforcement can be selected for both shear and flexure. The longitudinal reinforcement must, obviously, be developed between the free end of the ledge and the failure plane.

Based on the above definition, it is suggested that if the PCI equation, $w = d_e + (b + h)/2$, is used to determine the distribution width of the transverse reinforcement, the moment arm used to determine the longitudinal flexural reinforcement should be from d_e to w , or $(b + h)/2$. If the author's equation, $w = d_e + (b + h)/4$, is used for the distribution width, the moment arm need only be $(b + h)/4$. Shear reinforcement should be provided over the length of the moment arm, for whichever method is selected.

Further, it is suggested that the longitudinal reinforcement should have the same designations as the transverse reinforcement. The longitudinal flexural reinforcement should be A_{sl} , and the longitudinal shear friction should be A_{sl} . The vertical stirrups in the ledge should be designated A_{vt} .

In the transverse direction, the vertical

reinforcement consists of the A_{sh} bars, and the vertical component of the diagonal reinforcement. The horizontal reinforcement consists of the A_s , A_{vh} , and the horizontal component of the diagonal bars. It should be noted that even though the diagonally placed bar may be theoretically efficient, the placement problems far outweigh the benefits. Diagonal reinforcement is extremely sensitive to the angle of placement, and is therefore not recommended, without extensive inspection.

In order to simplify the design, the PCI method and the author's paper separates the flexural and horizontal shear friction reinforcement. In analyzing the capacity of the ledge, this separation is too conservative. The flexural capacity in the transverse direction can be calculated using all the reinforcement, including the A_s , A_{vh} , and the horizontal component of a diagonal bar A_{sh} . A strain compatibility computer program can simply calculate the capacity. Similar calculations can be made for the flexural capacity in the longitudinal direction. This capacity does not change as the angle of the failure plane changes.

The shear capacity can be calculated using all of the above reinforcement and any stirrups (A_{sh} or A_{vt}) that crosses the failure plane. A shear friction analysis can provide a capacity at any failure plane angle. The capacity will vary with the angle of failure.

The smallest shear friction capacity is compared to the flexural capacity and the lesser is the capacity of the section. This analysis is run in both transverse and the longitudinal directions. The summation of both the transverse and the longitudinal capacity is the capacity of the ledge. The analysis described is for the capacity of the reinforced member after that member has cracked. A deep ledge, however, can have a higher ultimate concrete strength than reinforced section strength.

In conclusion, if design constraints cause heavy reactions to be placed near the end of the ledge, caution must be exercised. Selection and detailing of sufficient

reinforcement, both vertical stirrups and horizontal bars, to distribute the load along the ledge should be considered. The detailing of both the longitudinal and transverse reinforcement within the failure volume, under the bearing pad, is extremely important to the development of the full capacity of the ledge.

References

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11. Mirza, S. A., and Furlong, R. W., "Strength Criteria for Concrete Inverted T-Girders," *Journal of Structural Engineering*, ASCE, V. 109, No. 8, August 1983, pp. 1836-1853.
12. Mirza, S. A., and Furlong, R. W., "Serviceability Behavior and Failure Mechanisms of Concrete Inverted T-Beam Bridge Bentcaps," *ACI Journal*, Proceedings V.80, No. 4, July-August, 1983, pp. 294-304.
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V. G. NAIDU*

The spandrel beam failures described pictorially in the article could have been avoided had the design engineers followed the PCI Design Handbook, the PCI Connections Manual, and basic fundamentals of structural engineering. Figs. 1 through 9 clearly indicate a lack of understanding of engineering fundamentals on the part of the designer. I doubt whether many engineers realize that the PCI handbooks are not written for those without structural engineering backgrounds.

The author has failed to present all the facts (reinforcing details, loadings, concrete type, member sizes, connection details, etc.) so that the reader may fully understand the failure mechanism. Specifically:

1. Fig. 4a: The corbel design is questionable rather than the spandrel beam design. For example, no reinforcement is

seen across the failure plane. Also, the bearing pad is set too close to the edge of the corbel.

2. Fig. 8: The dap is very shallow and, therefore, insufficient dap reinforcement, A_{sh} , is not the real problem. The crack would not have occurred had the designer provided adequate shear and torsion reinforcement.

3. Fig. 9: It is not possible for anyone to analyze the mode of failure unless the reinforcement details, casting methods, and loads are known.

4. In my opinion, poor manufacturing and careless construction techniques have a greater impact on failures than an error in design. It is not clear from the article as to what extent the above factors contributed to the failures.

5. The design engineer's knowledge of engineering mechanics combined with sound plant cast production and erection techniques could eliminate the need for specialists involved in investigations and special reports.

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AUTHOR'S CLOSURE by CHARLES H. RATHS*

I wish to express my appreciation to Messrs. Krauklis and Guedelhoefer and Naidu for their comments on my paper. I also wish to thank the many other individuals who have verbally commented on the paper.

The article was based on my many years of experience in the design of spandrel beams and also as a consultant who has examined numerous structural failures and problems of cracking and distress.

The subject of spandrel beams is a fairly complicated subject. There are obviously many facets involved in the design of spandrel beams as pointed out by the discussers. These aspects should certainly be considered in specific cases of spandrel beam design. The author will look forward to seeing the published results of the spandrel beam tests undertaken by Messrs. Krauklis and Guedelhoefer.

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DISCUSSION NOTE

The Editors welcome discussion of papers published in the PCI JOURNAL. The comments must be confined to the scope of the article being discussed. Please note that discussion of papers appearing in this issue must be received at PCI Headquarters by May 1, 1986.