



DISCUSSION

Development of a rational design methodology for precast concrete slender spandrel beams

In the Spring 2011 issue of the *PCI Journal*, the authors of “Development of a Rational Design Methodology for Precast Concrete Slender Spandrel Beams”¹ refer on page 110 to ledge punching failures occurring at loads below those predicted by the *PCI Design Handbook: Precast and Prestressed Concrete*² equations and to similar results encountered by Klein.³ PCI has funded and awarded a research grant to study this issue in depth, to develop the design methodology to more accurately predict the punching shear capacity, and to devise appropriate reinforcing and detailing, as necessary, to accommodate more heavily loaded cases. Results of this research are expected to be available in early 2014.

The purpose of this letter is to consolidate this information, noting the significance of the comparison between actual failure loads and the applicable *PCI Design Handbook* equations and suggesting interim precautions in the design of ledges until this research is completed.

Section 5.5.1 of the seventh edition of the *PCI Design Handbook* gives Eq. (5-44) (interior) and Eq. (5-45) (end, modified to show allowable punching stress applied to direct shear area) to calculate the nominal shear strength V_n by applying an allowable shear stress to simplified direct punching shear areas for spandrel end conditions and interior conditions (away from the ends of the spandrel). In Eq. (5-44) and (5-45) l_p is defined as $b_l - b$, where b_l is L beam width of web and ledge and b is beam web width.

Based on specific dimensional and concrete strength information provided by both North Carolina State University (NCSU)¹ and Klein,² following are the V_n capacities per the current *PCI Design Handbook* equations.

Interior condition: NCSU

Specimens 14 and 15: actual $f'_c = 6000$ psi (41.37 MPa)

$$V_n = 3\sqrt{f'_c}h_l(2l_p + b_l + h_l)$$

where

f'_c = specified compressive strength of concrete

h_l = height of ledge = 8 in.

l_p = 8 in.

b_l = width of bearing area = 3.75 in. (95.3 mm)

$V_n = 51.6$ kip (230 kN)

Interior condition: Klein

Specimen 2: actual $f'_c = 5640$ psi (38.89 MPa)

$$V_n = 3\sqrt{f'_c}h_l(2l_p + b_l + h_l)$$

where

$$l_p = 6 \text{ in.}$$

$$b_t = 4.0 \text{ in. (100 mm)}$$

$$h_t = 12 \text{ in.}$$

$$V_n = 75.7 \text{ kip (337 kN)}$$

End condition: NCSU

Specimen 16: actual $f'_c = 5200 \text{ psi (35.85 MPa)}$

$$V_n = 2\sqrt{f'_c}h_t\left(l_p + d_e + \frac{b_t}{2} + \frac{h_t}{2}\right)$$

where

$$l_p = 8 \text{ in.}$$

$$h_t = 8 \text{ in.}$$

d_e = distance from center of load to end of ledge = 21 in.

$$b_t = 3.75 \text{ in. (95.3 mm)}$$

$$V_n = 40.2 \text{ kip (179 kN)}$$

End condition: Klein

Specimen 2: actual $f'_c = 5640 \text{ psi (38.89 MPa)}$

$$V_n = 2\sqrt{f'_c}h_t\left(l_p + d_e + \frac{b_t}{2} + \frac{h_t}{2}\right)$$

where

$$l_p = 6 \text{ in.}$$

$$h_t = 12 \text{ in.}$$

$$d_e = 23 \text{ in.}$$

$$b_t = 4.0 \text{ in. (100 mm)}$$

$$V_n = 66.7 \text{ kip (297 kN)}$$

The actual ledge punching failure loads were also provided by NCSU and Klein, and **Tables 1** and **2** show the comparisons with the applicable V_n capacities per the current *PCI Design Handbook* equations.

Table 1. Comparison of predicted and actual punching shear from NCSU

Specimen	Condition	f'_{cr} psi	Calculated V_n kip	V_{test} kip	% of calculated
SP14	Interior	6000	51.6	32.7	63
SP15	Interior	6000	51.6	28.2	55
SP16	End	5200	40.2	25.6	63

Source: Lucier et al. 2011.

Note: f'_c = specified concrete compressive strength; V_n = nominal shear strength; V_{test} = tested shear strength. 1 kip = 4.448 kN; 1 psi = 6.895 kPa.

Table 2. Comparison of predicted and actual punching shear from Klein

Spandrel mark number	Condition	f'_c psi	Calculated V_n kip	V_{test} kip	% of calculated
2	Interior	5640	75.7	42.7	56
2	End	5640	66.7	42.7	64

Source: Klein 1986.

Note: f'_c = specified concrete compressive strength; V_n = nominal shear strength; V_{test} = tested shear strength. 1 kip = 4.448 kN; 1 psi = 6.895 kPa.

Observations

The actual capacity at punching failure in the NCSU tests,¹ as well as in tests conducted by Klein,³ ranged from 55% to 64% of the capacity calculated by the current *PCI Design Handbook* equations.

Table 3 shows the effects if the calculated values for ledge capacities by the applicable *PCI Design Handbook* equations are reduced by 50%.

A 50% reduction appears to accommodate the five known cases of actual recorded failure loads, though subsequent research might result in a further reduction (Table 3).

Research is under way to examine a broad range of cases, loading conditions, and failure modes to ensure that the final design criteria conservatively cover all cases. Actual test loading on ledges will be conducted.

Suggested interim procedure

As an interim precaution, I suggest that designers give consideration to limiting the allowable punching shear capacity of ledges in L-spandrels, L-beams, and inverted-T beams to not more than 50% of the values calculated by the *PCI Design Handbook* equations.

For normalweight concrete:

$$\phi V_n(50\%) = 1.5\phi\sqrt{f'_c}h_l(2l_p + b_l + h_l) \text{ interior condition} \quad \text{Eq. (5-44R)}$$

$$\phi V_n(50\%) = 1.0\phi\sqrt{f'_c}h_l\left(l_p + d_c + \frac{b_l}{2} + \frac{h_l}{2}\right) \text{ end condition} \quad \text{Eq. (5-45R)}$$

The formula was restated to show the allowable punching stress applied to a direct shear area.

For the end condition, check the interior condition equation and use the lesser of the two values.

Table 3. Comparison of predicted and actual punching shear

Specimen or spandrel mark no.	Condition	Calculated $V_n/2$, kip	V_{test} kip	% of $V_n/2$
SP14	Interior	25.8	32.7	127
SP15	Interior	25.8	28.2	109
SP16	End	20.1	25.6	127
2	Interior	37.9	42.7	113
2	End	33.3	42.7	128

Sources: Lucier et al. 2011 and Klein 1986.

Note: V_n = nominal shear strength; V_{test} = tested shear strength. 1 kip = 4.448 kN.

Effects of using Eq. (5-44R) and Eq. (5-45R) in the design of ledges

If the calculated $V_n/2$ capacity is less than the V_n required for the ledges on a project, a small increase in depth or width of the ledge will usually suffice and is not likely to have unfavorable consequences to the configuration of the ledge/double-tee dap relationships or to forming considerations. For example, by increasing the depth of the ledge from 8 in. to 9 in. the calculated capacity (50%) of SP15 Interior is increased from V_n of 25.8 kip (115 kN) to 30.1 kip (134 kN).

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Authors' response

Donald Logan recommends interim procedures for ledge design based on review of punching failure loads in laboratory tests. We commend him for this effort, which compares test results to strength predictions. As noted by Logan, test results from multiple full-scale laboratory tests indicate that ledge designs based on the *PCI Design Handbook*² equations are unconservative with respect to punching shear capacity. Logan's interim recommendation of using 50% of the punching shear capacity calculated by the *PCI Design Handbook* provides a conservative estimate of ledge punching capacity for all known tests in the literature.

The following points are offered for further perspective on the ledge punching issue:

- Full-scale tests from Lucier et al.,¹ Klein,³ and Krauklis and Guedelhofer⁴ all indicate that the current *PCI Design Handbook* equations are unconservative, sometimes by significant margins, as pointed out by Logan. Tests conducted by Mirza et al.⁵ indicated that the current equations are appropriate and conservative. Thus, it appears that the current PCI equations do not appropriately recognize the parameters relevant to ledge capacity.
- Widespread ledge failures have not been observed in practice.
- Current design equations are based on ledge projection. The punching resistance perimeter is actually also a function of the position of the bearing and depth of the ledge, which are not always directly related to the ledge projection.
- As we pointed out,¹ punching shear capacity is reduced by global flexural and shear stresses in the ledge. The ratio between punching shear stress and global flexural and shear stresses varies significantly with spandrel geometry and load position. In most of the laboratory tests, the global tension and shear stresses in the ledge were greater than the global stresses associated with the factored loads. If the global shear and tensile stresses in the ledge were at or below the level associated with factored loads, punching shear strength would have been somewhat greater. A few ledge punching failures were observed in limited laboratory tests where global stresses were near the factored levels.
- There is a significant eccentricity between the load position and the centroid of the punching shear resisting section, which increases punching shear stresses at the vertical face of the ledge. This eccentricity is discussed in greater detail by Klein.³
- The current *PCI Design Handbook* Eq. (5-44) and (5-45) have been criticized as unclear regarding the location of the bearing with respect to the end of the ledge. The derivation of these equations and *PCI Design Handbook* Fig. 5.5.1 seem to indicate that the equations are based on an interior and an end load

location, respectively. However, the text of the *PCI Design Handbook* requires that both equations be checked and the minimum value be used for all load locations when dealing with widely spaced loads. Logan's calculations assume an interior and exterior load location, as is often done in design practice. When ledge capacity is taken as the minimum of both equations, the predictions are still significantly unconservative with respect to the measured test data.

As noted, we are working to evaluate ledge design procedures in a new research program sponsored by PCI in response to the concerns we raised in our paper.¹ The program is just getting under way, and our research team will consider these factors and others while endeavoring to develop design procedures that are conservative, rational, and easy to use.

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