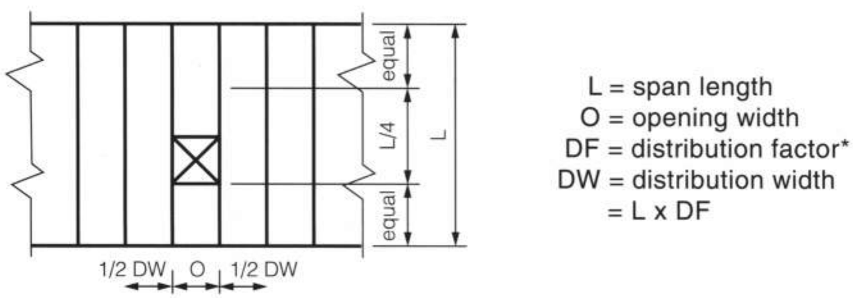
SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

LOAD DISTRIBUTION AROUND CENTRAL OPENINGS

Tests were conducted to determine the effects of openings in the midspan area of a Spancrete® hollowcore system. These tests were one phase of the study of the distribution of non-uniform loads. For this test series, a central opening was defined as an opening located within the center quarter of the span and away from a free edge; the opening width used was 40".



Test Assembly

CONCLUSIONS:

1. Central openings do not negate the ability of the Spancrete plank system to distribute loads.

2. Central openings affect the bending distribution width by reducing the stiffness of the system. In checking width to span ratio, subtract the opening width from the system width.

3. Central openings do not affect the distribution width for shear design near a support.

4. Central openings essentially cause additional loads on the adjoining plank which may be distributed as explained in the Research Notes entitled **"Load Distribution"** and **"Width To Span Ratio Effect On Load Distribution"**.

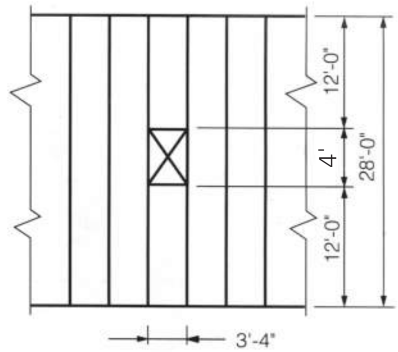
*Distribution factors are listed in the *"Load Distribution"* Research Notes.

A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

LOAD DISTRIBUTION AROUND CENTRAL OPENINGS



GIVEN:

8" Spancrete® hollowcore floor shown; plank dead load = 64 psf
Superimposed dead load = 10 psf
Superimposed live load = 40 psf
Opening sawcut after grouting.

PROBLEM:

Determine the design loads for the plank supporting the opening.

SOLUTION:

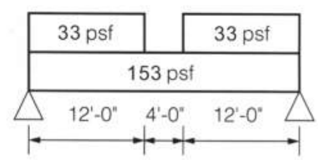
For flexural design, use a total distribution width of 0.55 L to distribute the load from the deck cut by the opening, and then add the factored uniform loads.

$$W_u = \frac{[1.2\ (64 + 10) + 1.6\ (40)]\ 3.33}{0.55 \times 28} = 33 \text{ psf}$$

$$W_u = 1.2\ (64 + 10) + 1.6\ (40) = 153 \text{ psf}$$

$$M_u = 17{,}370 \text{ ft-lb/ft}$$

No special shear design is required, since the opening is located in the middle quarter of the span.



Factored Loads for Flexure
(Working stress conditions will also have to be checked)

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia	*Ireland*
China	*Japan*
Denmark	*Russia*
Guatemala	*South Korea*
Hungary	*Switzerland*

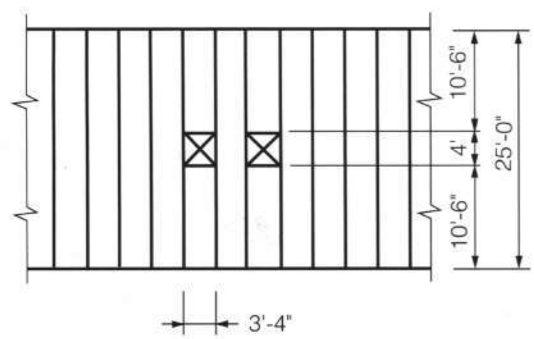
SPANCRETE MANUFACTURERS ASSOCIATION MEMBERS

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

LOAD DISTRIBUTION AROUND MULTIPLE CENTRAL OPENINGS

Tests were conducted to study the lateral distribution of loads in a Spancrete® hollowcore system containing two full plank openings closely spaced at midspan. In the first test, the openings were saw cut after the planks were grouted to evaluate the influence of overcut. For the second test, the openings were created with short slabs supported by angle headers in order to evaluate the distribution of slab weight prior to grouting.



Test Assembly

CONCLUSIONS:

1. The Spancrete planks exhibited sufficient flexural ductility to develop the capacity of the entire system.

2. Overcut adjacent to sawn openings has no adverse affect on performance or ultimate strength.

3. Self weight may be distributed as superimposed load even if headers are used prior to grouting.

4. The previously recommended flexural distribution width of 0.55L is valid for both working load and ultimate conditions. (Distribution factors are listed in the Research Note entitled **"Load Distribution"**)

5. Based on the excellent ductility shown by these tests, the same conclusions may be applied and the results extrapolated for 48" and 60" and 96" Spancrete widths.

A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

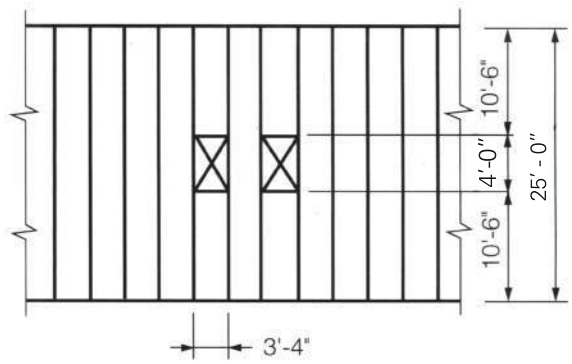
LOAD DISTRIBUTION AROUND MULTIPLE CENTRAL OPENINGS

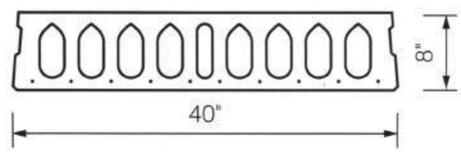
GIVEN:

8" Spancrete® hollowcore system shown; plank dead load = 64 psf
Superimposed dead load = 15 psf and live load = 50 psf

PROBLEM:

Select a Spancrete section to support the given loads;
Check working and ultimate conditions based on flexure





A = 218 in² 1 = 1515 in⁴
Y_b = 3.98 in e = 3.04 in

SOLUTION:

Effective distribution width of midspan =
0.55 (25) + 3.33 = 17.08 ft

Effective self weight $W_D = 64 + \frac{2(3.33)\,64}{17.08} = 89$ psf

Effective superimposed $W_D = 15 + \frac{2(3.33)\,15}{17.08} = 21$ psf

Effective live load $W_L = 50 + \frac{2(3.33)\,50}{17.08} = 70$ psf

Try: 8608 (3/4" clear cover, 8-3/8" 250 ksi strands,
65% initial tension, 20% losses, $f_{PU}\,A_{PS}$ = 20K)

$M_w = \frac{25^2}{8}(.089 + .021 + .070)\,3.33 = 46.9$ ft-k/slab

Check tension: $f_t = 8 \times 20 \times .65 \times .8\left(\frac{1}{218} + \frac{3.98 \times 3.04}{1515}\right)$
$-\frac{46.9 \times 12 \times 3.98}{1515} = 0.432$ ksi

Check LL deflection: $\Delta = \left(\frac{5}{384}\right)\frac{3.33\,(.070)\,25^4\,(1728)}{4300 \times 1515} = 0.31" = L/950$

Check ultimate $M_u = \frac{25^2}{8}[1.2\,(.089 + .021) + 1.6\,(.070)]\,3.33 = 63.54$ ft-k/slab

$\phi M_n \cong \frac{.9}{12}(.98 \times 8 \times 20)\left(7.06 - \frac{.98 \times 8 \times 20}{2 \times .85 \times 4 \times 40}\right) = 76.25$ ft-k/slab (> M_u)

Check slab between openings for headered construction loads of 20 psf before grouting

Header reactions $R = \frac{10.5 \times 3.33}{4}(.064 + .02) = 0.74^k$

$M \cong \frac{25^2}{8}(.064 + .020)\,3.33 + 10.5\,(2)\,.74 = 37.4$ ft-k/slab (< M_w)

$M_u = (1.2)\,[(.064)\,(3.33)\left(\frac{25^2}{8}\right) + (10.5)\,(2)\,(.74)] + (1.6)\,(.020)\,(3.33)\left(\frac{25^2}{8}\right) = 46.96\,\frac{\text{ft-k}}{\text{ft}}$ (<63.54)

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

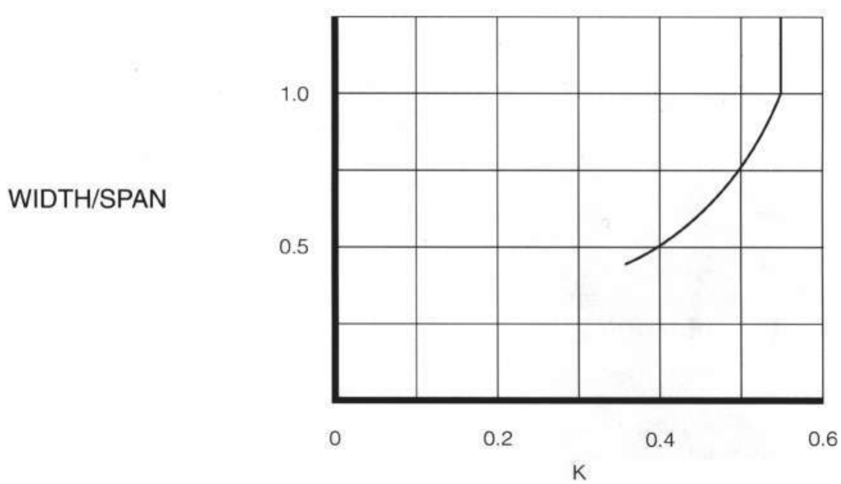
RESEARCH NOTES

WIDTH TO SPAN RATIO EFFECT ON LOAD DISTRIBUTION

In the study of distribution of non-uniform loads*, it was found that the midspan distribution width was a function of the width to span ratio. In most situations, this ratio will be much greater than 1.0.

However, for the special cases where this ratio is less than 1.0, the basic distribution widths* must be expressed as KL, where K is determined from the figure below.

For edge loads, the factor K must be halved. Where central openings are present, a net width should be used for determining the width to span ratio.



*For further information, refer to the Research Notes entitled **"LOAD DISTRIBUTION"**.

A design example is given on the reverse side.

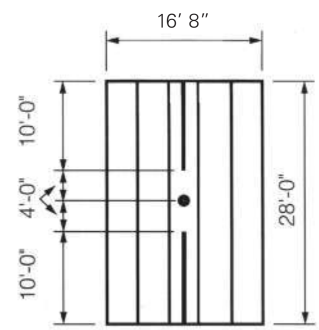
SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

WIDTH TO SPAN RATIO EFFECT ON LOAD DISTRIBUTION

GIVEN:

8" Spancrete® hollowcore floor shown
Superimposed live load = 40 psf
Superimposed dead load = 10 psf   Plank dead load = 64 psf



WALL LOAD = 700 plf DL
WALL LOAD =1100 plf LL
CONCENTRATED LOAD = 2800lb. DL
CONCENTRATED LOAD = 4400lb. LL

PROBLEM:

Determine the equivalent effective design loadings to enable the floor slabs within the allowable distribution widths to carry the loads shown.

SOLUTION:

$$\text{Width/Span} = \frac{16.67}{28} = 0.6$$

From chart, K = 0.44
Figure separately the distribution for the concentrated load, the wall load, and the uniform loads.

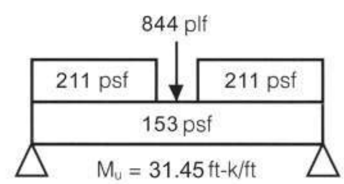
For flexural design:

$$P_u = \frac{1.2\,(2800) + 1.6\,(4400)}{0.44 \times 28} = 844 \text{ plf}$$

$$W_u = \frac{1.2\,(700) + 1.6\,(1100)}{0.44 \times 28} = 211 \text{ psf}$$

$$W_u = 1.2\,(64 + 10) + 1.6\,(40) = 153 \text{ psf}$$

For shear design:
Width to span ratio does not affect design for shear.
See RESEARCH NOTE "**LOAD DISTRIBUTION**."



FACTORED LOADS FOR FLEXURE

(Working stress conditions will also have to be checked.)

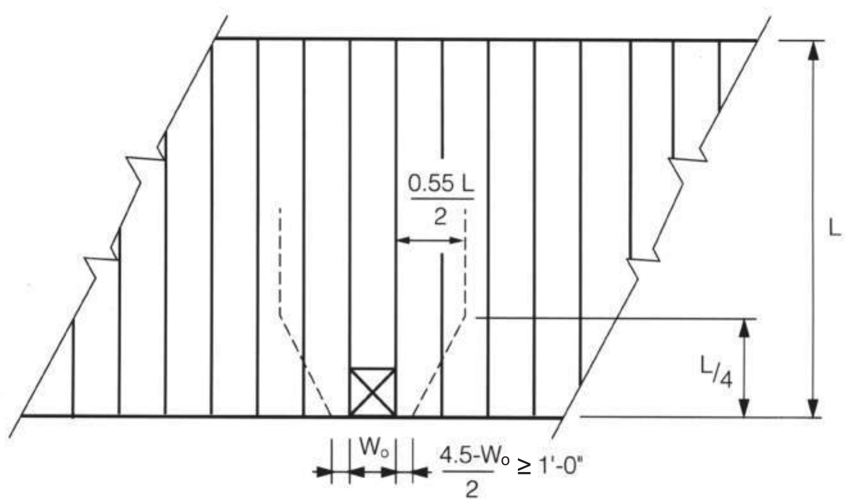
Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SHEAR DESIGN AT END OPENINGS

Research on load distribution has been a major part of the continuing testing program conducted by the Spancrete Manufacturers Association.



EFFECTIVE RESISTING SLAB WIDTH AT END OPENINGS

CONCLUSIONS:

1. Openings at the end of a span will cause a concentration of shear stresses due to shear and torsion at the sides of an opening.

2. The midspan region will not be affected by an end opening as long as strand development occurs after the opening and before the area of maximum moment.

3. The effective resisting section shown is recommended for shear design around end openings. The design procedure is to superimpose on the uniform loads the distributed load concentrations.

A design example is given on the reverse side.

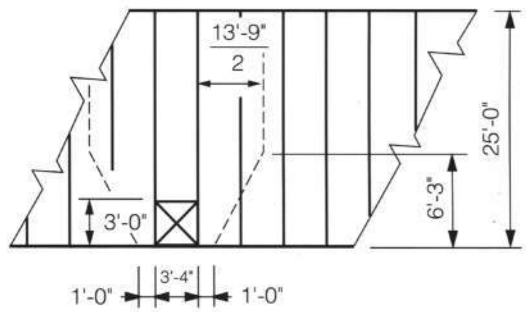
SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

SHEAR DESIGN AT END OPENINGS

GIVEN:

An 8" x 40" Spancrete® hollowcore system as shown. Self-weight = 64 psf, uniform superimposed dead and live loads are 10 psf and 40 psf



PROBLEM:

Select the prestress, check strand development, and check shear.

SOLUTION:

1. From load tables, for a total superimposed load of 50 psf, select eight 5/16" dia. 250 ksi strands with 3/4" clear cover. Use 65% for initial tension and assume 20% losses, and concrete strength of 5000 psi
2. Check strand development from opening

   L_d required = $(f_{ps} - 2/3 f_{se})\, d_b$ = (.98 x 250 - 2/3 x .65 x .8 x 250)5/16 = 50"

   L available = $\left(\frac{25}{2} - 3\right)12$ = 114"> 50" OK
3. Check shear caused by the additional concentration of load from the strip of slab containing the opening.

   Plank bearing = 4"

   At h/2, distribution width DW = $1 + \frac{.333}{6.25}\left(\frac{13.75}{2} - 1\right) = 1.31'$

   $W_u = 1.2\,(10 + 64) + 1.6\,(40) = 153$ psf from uniform loads

   Distribute load from strip with opening and superimpose

   $W_u = 153 + \left(\frac{153 \times 3.33}{1.31 \times 2}\right) = 348$ psf

   Checking shear at critical points, find:

   $\phi v_{cw} = 203$ psi > $v_u = 118$ psi and $\phi v_{ci} = 106$ psi > $v_u = 78$ psi. Shear check is OK.
4. If inclined shear had not checked at some point in the span, calculate a new effective width at that point, determine a new distributed load, and recheck shear.

Additional information for Shear Design is provided in Research Note title, **"SHEAR STRENGTH"**

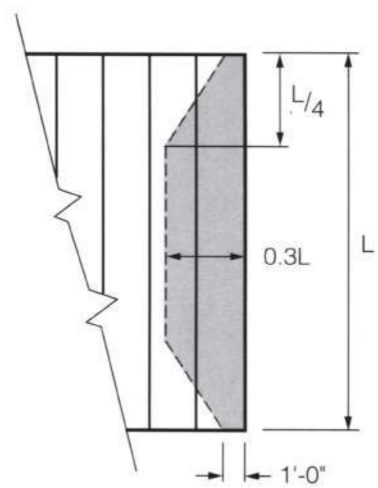
Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wire, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SHEAR DESIGN FOR EDGE LOADS

Research on load distribution has been a major part of the continuing testing program conducted by the Spancrete Manufacturers Association.



The shaded area represents the effective section carrying edge loads
FIGURE 1

CONCLUSIONS:

To account for torsional shear stresses in addition to direct shear stresses, it is recommended that an effective resisting section, as shown in Figure 1, be used to carry edge loads. The significant change from earlier recommendations is the reduction of the width at the support to one foot. Use of this resisting section will result in a prediction of the peak shear stress in the outermost webs.

A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

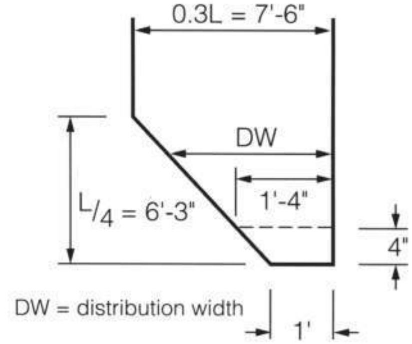
SHEAR DESIGN FOR EDGE LOADS

GIVEN:

An 8" x 40" Spancrete® hollowcore system as shown on the other side.
L = 25'; self weight = 64 psf; uniform superimposed live load = 40 psf and dead load = 10 psf.
A wall load on the outermost free edge represents line loads of 100 plf dead load and 350 plf live.

PROBLEM:

Select a prestressing level for the plank and check shear.



SOLUTION:

1. Select prestressing level on the basis of flexure

$$W_D = 10 + \left(\frac{100}{0.3 \times 25}\right) = 23 \text{ psf} \quad W_L = 40 + \left(\frac{350}{0.3 \times 25}\right) = 87 \text{ psf}$$

From load tables, use eight 3/8" dia. 250 k stands,
with initial stress = 65% of ultimate and clear bottom cover = 3/4"

2. Check web shear at h/2 = 0.33'

$$W_u = 1.2\,(10 + 64) + 1.6\,(40) + \frac{1.2\,(100) + 1.6\,(350)}{DW} = 153 + 680/DW$$

$$DW = \frac{0.33}{6.25}\,(7.5 - 1) + 1 = 1.34'$$

$$v_u = \left(\frac{25}{2} - 0.33\right)\left(.153 + \frac{.680}{1.34}\right) = 8.04\text{k/'} \quad \frac{v_u}{\varnothing} = \frac{8.04 \times 3.33}{.75 \times 17 \times 7.06} = .298 \text{ ksi}$$

$v_{cw} = 3.5\sqrt{f'c} + 0.3\,f_{pc}$ Assume prestress loss = 20%, bearing = 3"

$$f_{pc} = \frac{8 \times 20 \times .65 \times .8}{218}\left(\frac{3 + 4}{50 \times .375}\right) = .142 \text{ ksi (accounting for tranfer length)}$$

$$v_{cw} = 3.5\sqrt{4000} \div 1000 + 0.3\,(.142) = 0.264 < 0.298 \text{ NO GOOD}$$

Web shear capacity can be increased by grouting cores, (See Reserach Note, "**SHEAR STRENGTH WITH FILLED CORES**").

3. Check inclined shear at 2.75' from support. Starting with the same load values at h/2; $\frac{v_u}{\varnothing}$ will be 241 psi and v_{ci} will be only 220 psi. Therefore, recalculate the loads as would be distributed at 2.75'.

$$DW = \frac{2.75}{6.25}\,(7.5 - 1) + 1 = 3.86' \qquad W_u = 153 + 680 \div 3.86 = 329 \text{ \#/ft}$$

Now a new check of inclined shear will show shear is OK at all points.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary or the complete design. This research was done using 40" wire, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

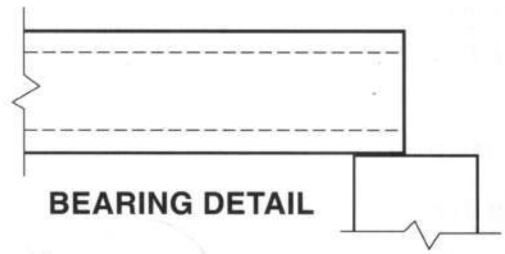
SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SHEAR STRENGTH

Tests were conducted to investigate the applicability of the ACI equations for shear in prestressed members to Spancrete® hollowcore plank. By varying the shear span, both web shear and inclined shear failure mechanisms could be observed.

TEST ASSEMBLY

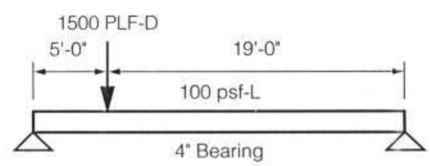


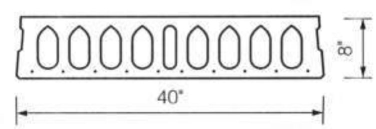
CONCLUSIONS:

1. The ACI equations for shear in prestressed members apply to Spancrete.
2. Satisfactory performance was observed for $V_u = \phi V_c$ without shear reinforcing.

DESIGN EXAMPLE

SHEAR STRENGTH





bw = 17" d = 7.06" I = 1515 in^4
Y_b = 3.98" W = 64 psf f'_c = 4000 psi
(Approach similar for any other plank width)

GIVEN:

8" Spancrete reinforced with (12) 3/8" dia., 250 ksi strands; superimposed loads as shown.

PROBLEM:

Check the member for adequacy in shear.

SOLUTION:

Governing equations (from ACI 318-02)

$$\text{(11 - 10)}\quad V_{ci} = 0.6\sqrt{f'_c}\, b_w d + V_d + \frac{V_i M_{cr}}{M_{max}}$$

$$\text{(11 - 11)}\quad M_{cr} = \frac{I}{y_b}\left(6\sqrt{f'_c} + f_{pe} - f_d\right)$$

$$\text{(11 - 12)}\quad V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right) b_w d + V_p$$

Investigate left 5' of span

$$V_d = \frac{24}{2}(3.33)(.064) - 3.33(.064)X = 2.56 - .213X \qquad M_d = 2.56X - \frac{.213X^2}{2}$$

$$V_i = \frac{24}{2}(3.33)\,1.6\,(.1) + \frac{19}{24}(3.33)\,1.2\,(1.5) - 1.6\,(3.33)(.1)X = 11.15 - .533X \qquad M_{max} = 11.15X - \frac{.533X^2}{2}$$

$$V_u = 1.2(2.56) + 11.15 - [1.2(.213) + .533]X = 14.22 - .789X$$

M_{cr} is a function of the strand transfer length $l_t = 50\, d_b$ = 18.75"

This design example continues on the other side.

SPANCRETE MANUFACTURERS ASSOCIATION

continued from reverse side

$A_{ps} f_{se} = 12(.08)\,250\,(.65)\,.8 = 124.8^K$ (Tensioning to 65% and assuming 20% losses.)

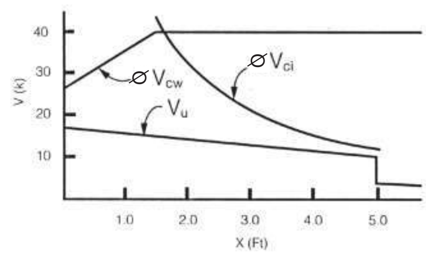
$$M_{cr} = \frac{1515}{3.98}\left[\frac{6\sqrt{4000}}{1000} + 124.8\left(\frac{1}{218} + \frac{3.04 \times 3.98}{1515}\right)\left(\frac{12X+4}{18.75}\right) - \frac{M_d\,(12)\,3.98}{1515}\right]\frac{1}{12}$$

$$= 12.04 + 49.78\left(\frac{12X+4}{18.75}\right) - M_d \text{ where } \left(\frac{12X+4}{18.75}\right) \leqq 1.0$$

$$f_{pc} = \frac{A_{ps}f_{se}}{A} = \frac{124.8}{218}\left(\frac{12X+4}{18.75}\right) \text{ where } \left(\frac{12X+4}{18.75}\right) \leqq 1.0$$

From 0 to 5', the following table can be developed by varying x and evaluating the equations.

x (ft)	V_u (k)	V_d (k)	V_i (k)	M_{max} (ft-k)	M_{cr} (ft-k)	$\varnothing V_{ci}$ (k)	$\varnothing V_{cw}$ (k)
0.33	13.96	2.49	10.97	3.68	32.34	78.2	26.5
1.00	13.43	2.35	10.62	10.88	52.06	43.3	33.1
2.00	12.64	2.13	10.08	21.23	57.13	25.4	35.4
3.00	11.85	1.92	9.55	31.05	55.10	17.6	35.4
4.00	11.06	1.71	9.02	40.34	53.28	13.6	35.4
5.00	10.28	1.50	8.49	49.09	51.68	11.2	35.4



Beyond 5', $\varnothing V_{ci}$ is the minimum per code while V_u drops drastically and therefore need not be checked for this case.
(V_{ci} min = $1.7\sqrt{f'_c}\,b_w d$)

Since $V_u < \varnothing V_c$, the section selected is adequate.

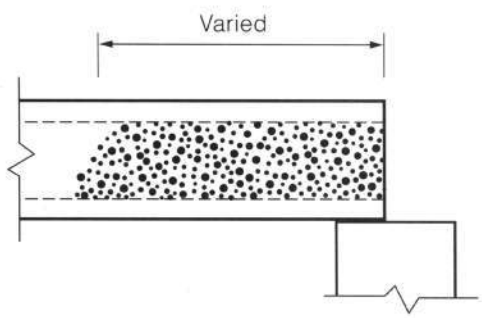
Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete® hollowcore. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

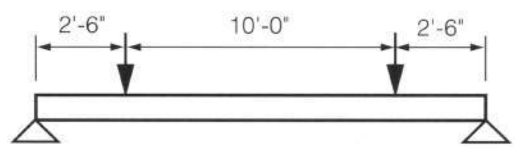
RESEARCH NOTES

SHEAR STRENGTH WITH FILLED CORES

Tests were conducted to determine whether filling the cores of Spancrete® hollowcore plank with grout was an effective method of increasing the shear capacity of the plank. A method to conservatively predict the increased shear capacity was also developed.



FILLED END DETAIL



TEST ASSEMBLY

CONCLUSIONS:

1. A grout fill will increase shear capacity, even when added after prestressing is completed.

2. A 3 :1 sand-cement grout gave adequate bond to act compositely in shear with the plank.

3. Non-shrink grout did not appreciably increase the shear capacity over regular grout.

4. Extending the grout fill only to the critical section gives satisfactory results, provided the reduced section is considered at the end of the grout fill.

5. Shear capacity of the total section can be conservatively calculated by a superposition of the base slab capacity and the grout capacity.

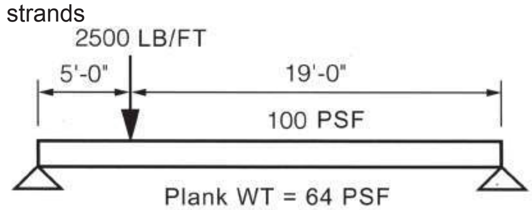
A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

SHEAR STRENGTH WITH FILLED CORES

GIVEN:
8" Spancrete® hollowcore reinforced with (12) 3/8" dia., 250 ksi strands
Superimposed live loads as shown.



PROBLEM:
Determine the number of cores to be filled to satisfy the required shear capacity.

SOLUTION:
At 5 feet from left support

$V_u = 16.1^k/\text{plank} \quad V_c/\phi = 12.6/.75 = 16.8\text{k/plank}$

from ACI (318-02) Eqn. (11-10) or (11-12)

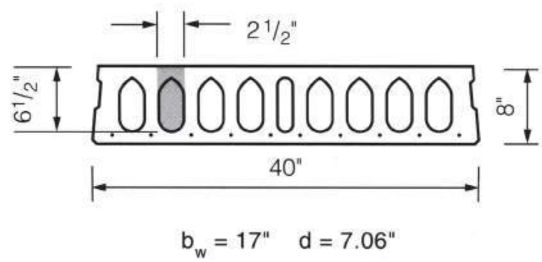
Add grout fill with $f'_c = 3000$ psi

$V_c = 2\sqrt{f'_c} \times b_{core} \times d_{core} = 2\,\dfrac{\sqrt{3000}}{1000} \times 2.5 \times 6.5$

$= 1.78^k/\text{core}$

For $V_u \leq \phi V_c$

$16.1 \leqq 0.75\ [16.8 + (N \times 1.78)]$ $N = 2.61$ **Say 3 cores**



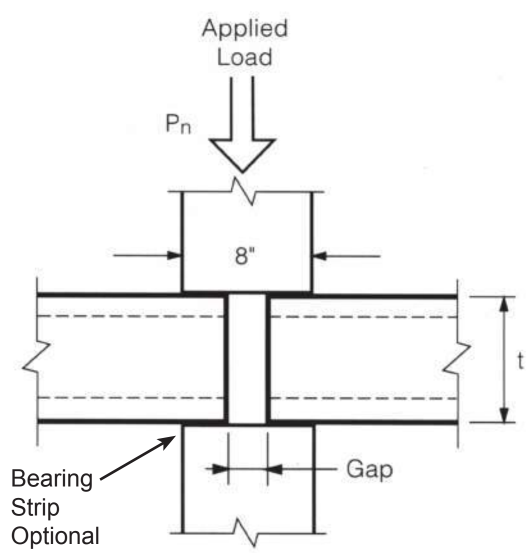
Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

CRUSHING CAPACITY OF PLANK ENDS

Tests were conducted on the end crushing strength of Standard Spancrete® hollowcore sections and Ultralite Spancrete sections. The most common practical occurrence of this loading mechanism is a wall bearing structure as detailed here. The factored load capacities established by the tests are given below.



TEST LOAD SECTION

Plank Thickness	1/2" Gap Ungrouted	2" Gap Ungrouted	Grouted Joint
8"	51 k/ft	56 k/ft	144 k/ft
10"	51 k/ft	46 k/ft	144 k/ft

ULTIMATE CRUSHING STRENGTH

For 8" bearing wall:

- Values listed include $\varnothing$ factor ($P_u = \varnothing P_n$)
- Grout is normal 3:1 mix.

A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

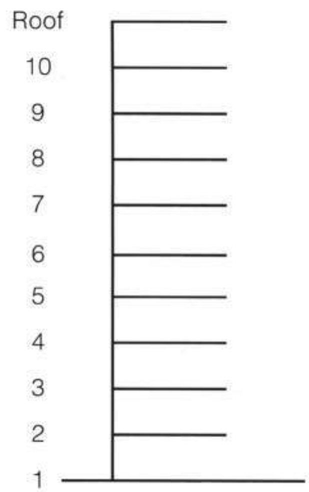
CRUSHING CAPACITY OF PLANK ENDS

GIVEN:

A 10 story building with 28' spans, 8'-8" floor to floor height, floor and roof live loads of 40 psf and dead loads of 10 psf. Live load reduction allowed on floors only.

PROBLEM:

Check capacity of Spancrete® hollowcore to withstand crushing load at lowest level.



SOLUTION:

Floor dead loads (64 + 10) psf x 28 ft x 8 = 16,576 #/ft
Roof dead loads (64 + 10) psf x 28 ft x 1 = 2072 #/ft
Wall dead loads 100 psf x 8 ft x 9 = 7200 #/ft
Floor live loads 40 psf x 40% x 28 ft x 8 = 3584 #/ft
Roof live load 40 psf x 28 ft x 1 = 1120 #/ft

ULTIMATE LOAD
$U = 1.2D + 1.6L + .5\ L_r$ (ACI 318-02, EQ. 9-2)
$= (1.2)(16.6 + 2.1 + 7.2) + (1.6)(3.6) + (.5)(1.1) = 37.5$ k/ft.
Capacity of 8" plank $\geqq$ 51 k/ft ungrouted, or 144 k/ft grouted.

CONCLUSION:

Spancrete plank will support the heaviest vertical load imposed by the given system.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

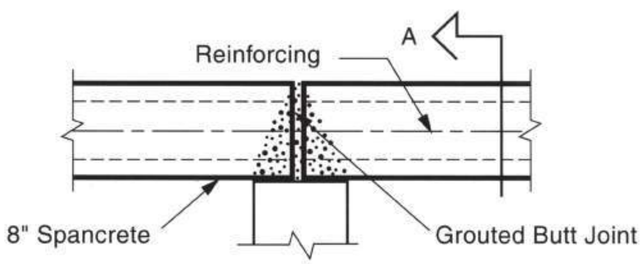
Armenia
China
Denmark
Guatemala
Hungary
Ireland
Japan
Russia
South Korea
Switzerland

SPANCRETE MANUFACTURERS ASSOCIATION

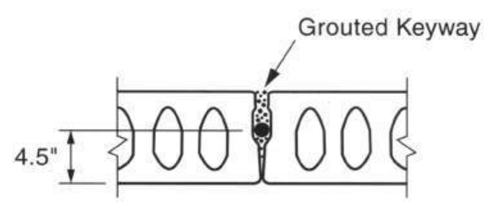
RESEARCH NOTES

BARS IN GROUT KEYS

Mild steel reinforcement in the grout keys is commonly used to "tie a building together." Tests were conducted to determine whether this reinforcement was capable of developing negative moments for continuity.



BUTT JOINT DETAIL



SECTION A

CONCLUSIONS:

1. Bars placed in grout keys can develop their full negative moment capacity.

2. The largest size bar that practically fits in the keyway is #4, and the width of Spancrete® hollowcore plank used (40", 48", 60", 96" depending on locality) limits the spacing available. Therefore, the negative moment capacity available is small, and deflections approach those of a simple span system.

3. The testing showed that #4 bars placed in the grout keys would yield and that negative moments would redistribute so that flexural failure in the positive moment region controlled.

A design example is given on the reverse side.

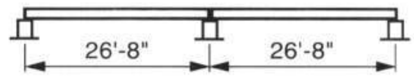
SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

BARS IN GROUT KEYS

GIVEN:

An 8" untopped Spancrete® hollowcore system as shown.
Assume plank weight = 64 psf, superimposed
dead load = 20 psf and live load = 50 psf.



PROBLEM:

Determine reinforcing requirements of plank if #4 bar is used in all grout keys.
[This example is based on 40" wide plank]

SOLUTION:

M_u = [1.2 (DL) + 1.6 (LL)] L^2 ÷ 8 x 1000

= [1.2 (64 + 20) + 1.6 (50)] 26.67^2 ÷ 8000 = 16 k-ft/ft

The negative moment capacity provided by one #4 bar @ 40" is

$\phi M_n^- = \phi A_s f_y (d - a/2)$ where $a = A_s f_y \div 0.85 \times f_c \times b$

= .9 x .2 x 60 (4.5 - [.2 x 60 ÷ .85 x 4 x 40] ÷ 2) ÷ 12 x 3.33

= 1.20 k-ft/ft

A hinge will form at the center support, since the negative moment potential is greater than the negative moment capacity ($M_u^- > \phi M_n^-$)

$R = [wL^2/2 - \phi M_n^-] \div L$ = [.181 x 26.67^2 ÷ 2 - 1.20] ÷ 26.67 = 2.37 k-ft/ft

X = 2.37 ÷ .181 = 13.09'

M_u^+ = 2.37 x 13.09 ÷ 2 = 15.51 k=ft/ft

Select the appropriate series plank from your load tables ≧ 15.51 k-ft/ft

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

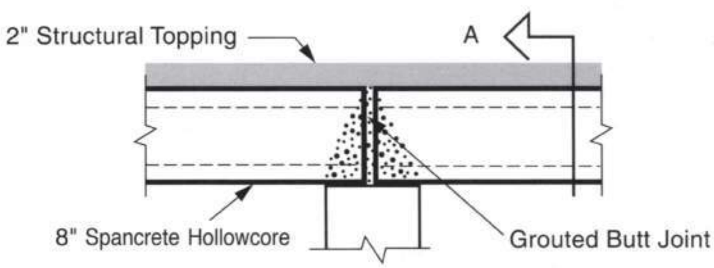
Armenia	Ireland
China	Japan
Denmark	Russia
Guatemala	South Korea
Hungary	Switzerland

SPANCRETE MANUFACTURERS ASSOCIATION

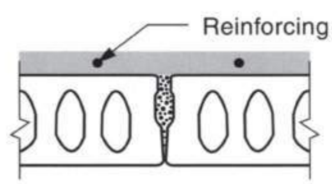
RESEARCH NOTES

CONTINUITY OVER SUPPORTS

Tests were conducted to determine whether mild reinforcements in a structural topping was an effective method to achieve continuity at plank ends. The most common application of this is for an increase in ultimate strength and a decrease in live load deflections.



BUTT JOINT DETAIL



SECTION A

CONCLUSIONS:

1. Negative moments and corresponding increases in ultimate strength can be achieved by using mild reinforcement in a structural topping.

2. Care must be taken to insure adequate bond between the topping and the plank.

3. Mild reinforcing will yield, and moment redistribution can be accomplished with reinforcing ratios ranging from 0.26% to 0.44%.

4. The mild reinforcement will cause a distribution of negative flexural cracking under loading, instead of one crack over the butt joint.

A design example is given on the reverse side.

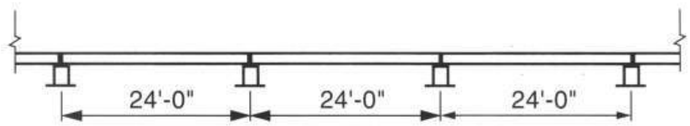
SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

CONTINUITY OVER SUPPORTS

GIVEN:

A 24 foot multispan 8" Spancrete®
system with 2" structural topping.
Superimposed live load = 100 psf,
plank dead load = 64 psf, topping = 25 psf.



PROBLEM:

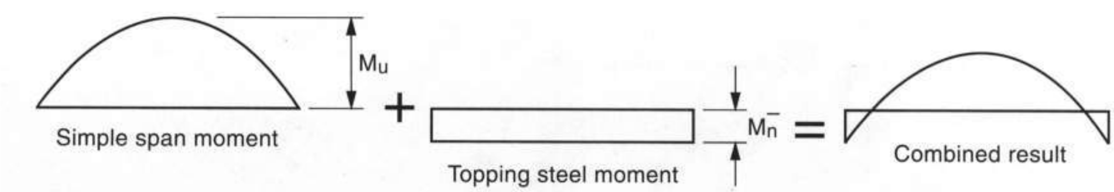
Determine reinforcing requirements of plank and topping.

SOLUTION:

For interior spans (exterior span approach is similar):

$M_u = [\,1.2\,(DL) + 1.6\,(LL)\,]\,L^2 \div 8 \times 1000$

$M_u = [\,1.2\,(64 + 25) + 1.6\,(100)\,]\,24^2 \div 8000 = 19.21$ k-ft/ft



Select a plank series less than required by simple span moment alone, since the plank ultimate moment capacity must equal $M_u + M_n^-$ combined.

Try a plank from the load tables with $M_u = 15.17$ k-ft/ft

If $M_u + M_n^- = 15.17$ k-ft/ft, then $M_n = 15.17 - 19.21 = -4.04$ k-ft/ft

$\varnothing M_n^- = \varnothing A_s^- f_y j_u d$; $A_s^- \cong (4.04 \times 12) \div (0.9 \times 60 \times .9 \times 9) = 0.11$ in^2/ft

This can be supplied by #3 bars at 12" or 6 x 6 - W5.5 x W5.5 mesh.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE
MANUFACTURERS ASSOCIATION

RESEARCH NOTES

DIAPHRAGM SHEAR ON UNTOPPED SPANCRETE® HOLLOWCORE DECKS

Two phases of tests were conducted to determine the longitudinal shear capacity of grouted keyways when a Spancrete deck is used as a diaphragm without topping. The grout placed in the keyways was a 3:1 sand-cement mix with sufficient water added to make it flowable. The grout strength was a minimum of 2500 psi at the time of testing.

The first phase consisted of tests conducted on two simple span plank decks to establish a reliable value for the diaphragm design shear strength of the grout key. A masonry bond beam served as both bearing support and tensile tie. The second phase consisted of four tests designed to impart direct shear on the grout key. These were conducted on a plank setup modeled as the last two slabs in a simple span diaphragm. Both test arrangements are shown on the reverse side.

CONCLUSIONS:

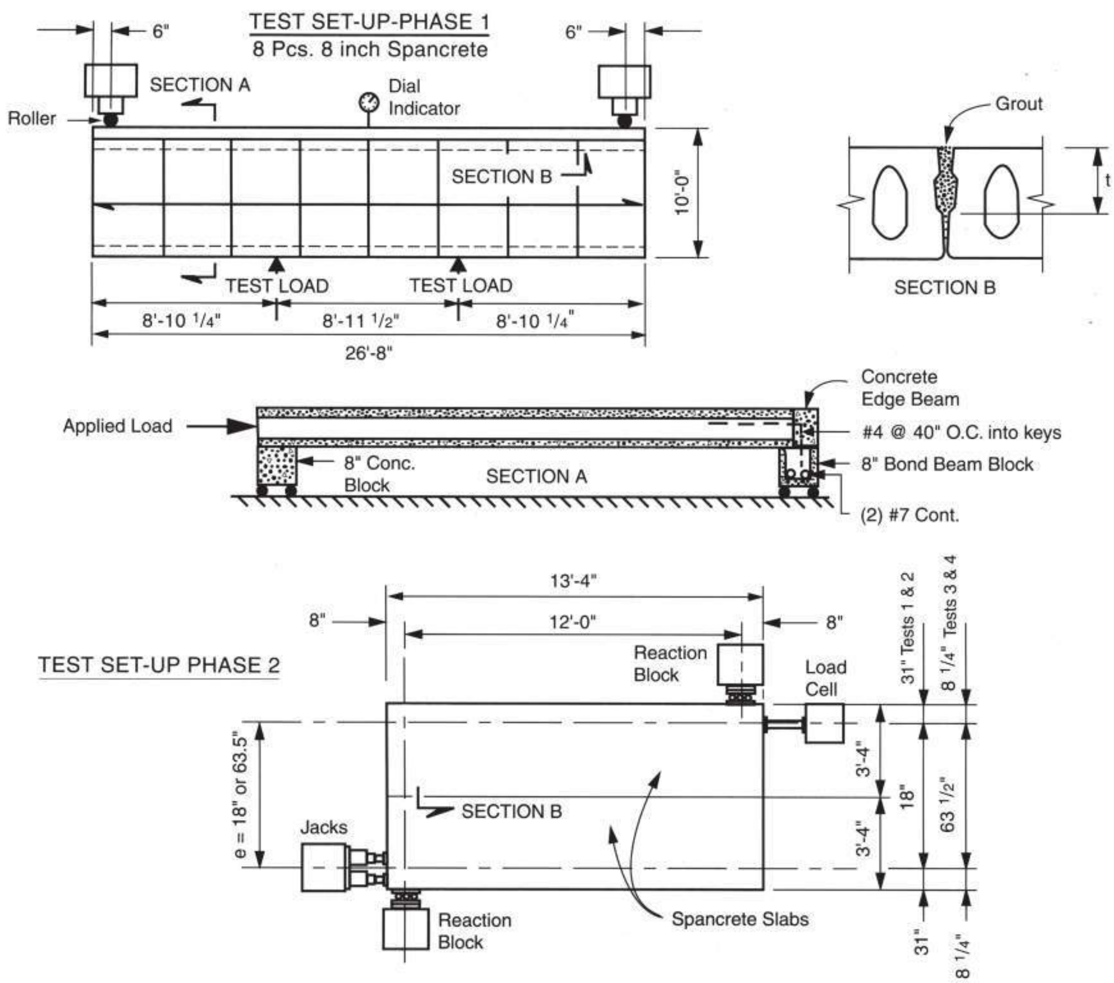
1. An untopped Spancrete deck will function satisfactorily as a diaphragm.

2. The keyway longitudinal shear capacity may be taken as $V_n = 0.04\, f'_{cg}\, ht \leq 120\, ht$
   where f'_{cg} = grout strength
   h = depth of diaphragm
   t = effective depth of grout

3. For detailed assistance in diaphragm analysis and design, consult the “PCI Design Handbook for Precast and Prestressed Concrete” or the “PCI Manual for the Design of Hollow Core Slabs.”

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

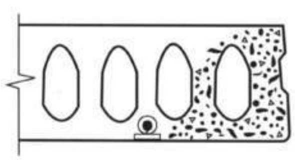


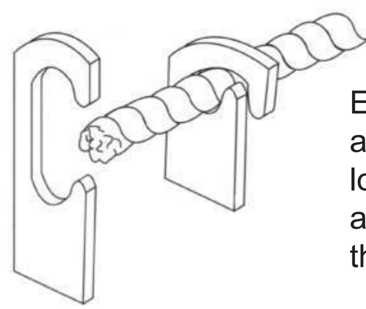
Note: This research was done using 40" wide, 8" thick Standard Spancrete® hollowcore. However, this concept applies to all Spancrete cross sections.

BOTTOM WELD PLATES

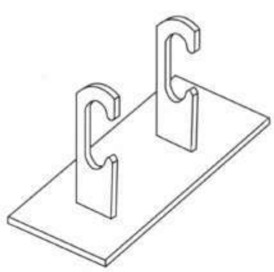
WITH STUD ANCHORS

As part of a program for evaluating various inserts used in Spancrete® hollowcore plank, weld plates, as shown below, were tested in the bottom of some plank. These plates were located on the casting bed prior to placing concrete, and centered on one of the bottom prestressing strands. Bending the stud tightly over the strand anchors the plate in its predetermined location.

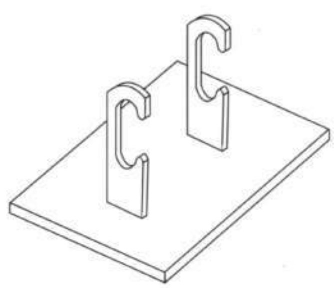




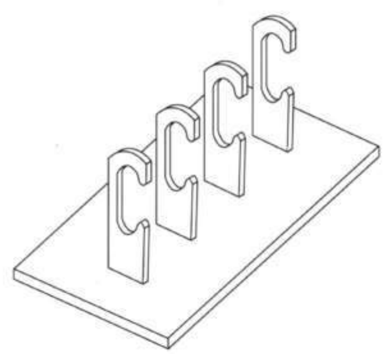
Each stud is crimped to a strand to initially fix its location and ultimately anchor the plate into the concrete.



BWA-3
Plate 3/16" x 2" x 6" with
Two 1/8" x 5/8" x 7/8"
Rectangular Studs



BWA-2
Plate 3/8" x 4" x 6" with
Two 1/8" x 5/8" x 7/8"
Rectangular Studs



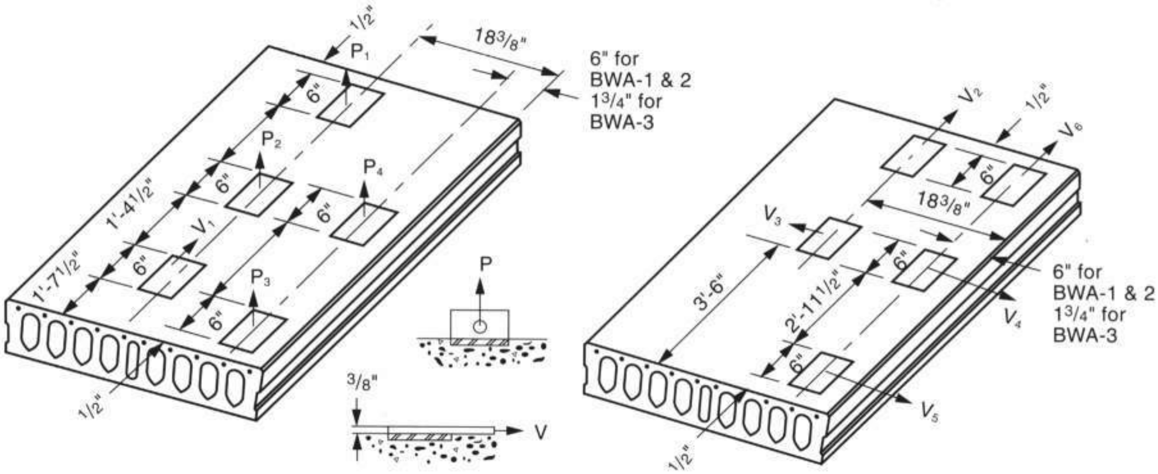
BWA-1
Plate 3/8" x 4" x 6" with
Four 1/8" x 5/8" x 7/8"
Rectangular Studs

Test setup and results are given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

BOTTOM WELD PLATE TEST SETUP (STUD ANCHORS)



SUMMARY OF TEST RESULTS (in kips)

First value given is average test capacity. Second value given is lowest test capacity.

INSERT	P_1	P_2	P_3	P_4	V_1	V_2	V_3	V_4	V_5	V_6
BWA-3	2.8	3.1	1.9	2.9	8.6	5.4	9.7	1.9	.95	3.0
	2.5	2.9	1.3	2.7	8.6	4.7	9.3	1.8	.70	2.6
BWA-2	3.6	4.1	3.1	2.9	14.3	5.3	12.7	6.2	2.7	3.2
	3.3	4.0	2.7	2.8	10.6	4.2	9.2	3.7	2.5	3.2
BWA-1	4.1	5.8	2.5	4.0	20.3	8.2	19.6	5.9	2.9	4.9
	3.3	5.2	2.4	3.1	19.6	5.6	15.2	5.0	2.6	4.2

Note: Actual test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

This research was done using 40" wide, 8" thick Standard Spancrete® hollowcore. However, this concept applies to all Spancrete cross sections.

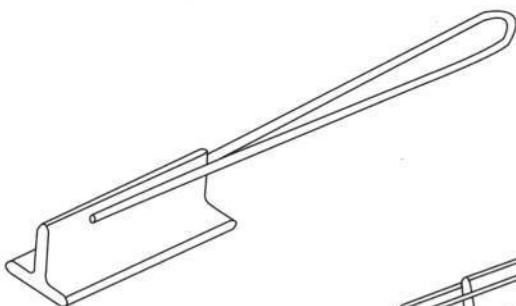
SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

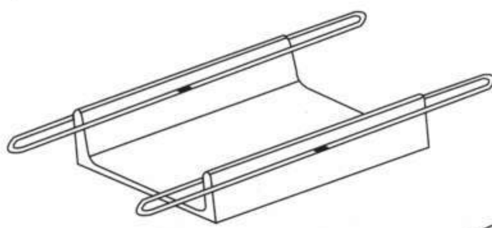
BOTTOM WELD PLATES

WITH WIRE LOOPS

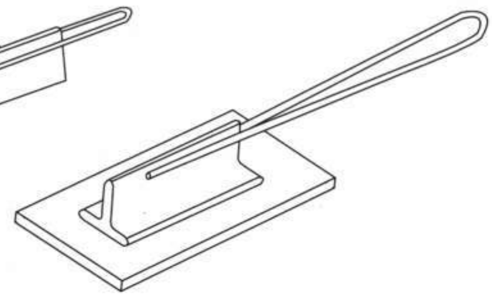
As part of a program for evaluating various inserts used in Spancrete® hollowcore plank, weld plates as shown below were tested in the bottom of some plank. The plates were located on the casting bed at a strand location and tied securely to the strand prior to placing concrete.



BWA-5
Tee 1 1/2" x 1/4" x 4"
with 1/4"" smooth bar
x 18" long



BWA-6
Channel 4" x 5.4# x 6"
with 1/4" x 10" loops



BWA-8
BWA-5 anchor welded
to plate 3/8" x 4" x 8"

SUMMARY OF TEST RESULTS (IN KIPS)

First value given is average test capacity. Second value given is lowest test capacity.

INSERT	P_1	P_2	P_3	P_4	V_1	V_2	V_3	V_4	V_5	V_6
BWA-5	3.2	5.9	1.5	3.2	13.3	6.3	14.8	2.2	1.1	2.7
	2.7	5.5	1.3	3.0	8.2	5.6	11.2	1.9	.72	1.7
BWA-6	7.8	12.2	5.0	7.2	32.7	26.6	32.0	12.4	5.5	11.0
	7.5	12.0	3.4	7.0	31.0	24.7	31.0	8.3	4.9	9.4
BWA-8	6.3	5.4	-	-	-	-	-	-	-	-
	5.8	4.5	-	-	-	-	-	-	-	-

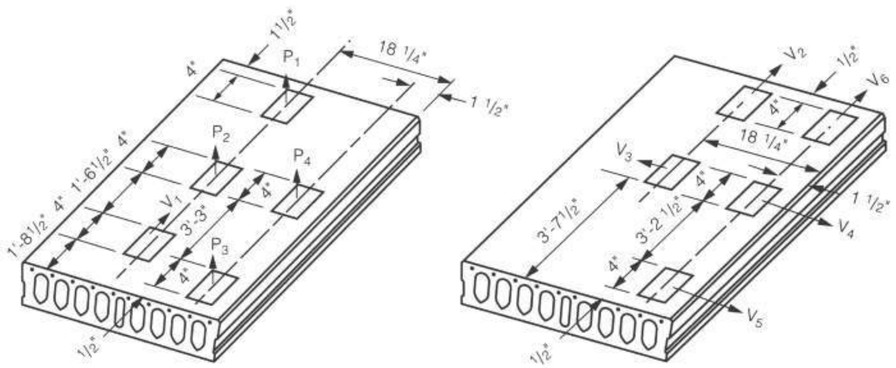
Note: Values given are test results. Appropriate strength reduction factors and safety factors must be applied to these values. Consideration should be given to the number of tests and the inconsistencies in results of the concrete failures when determining these factors.

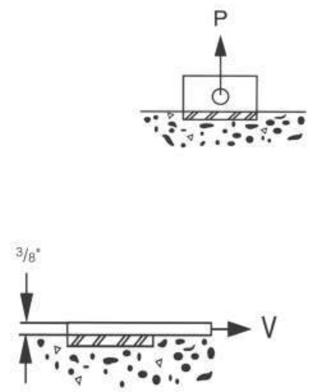
This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE
MANUFACTURERS ASSOCIATION

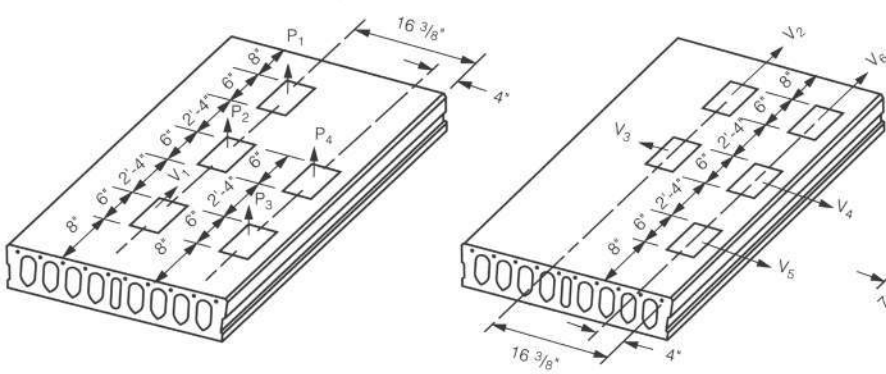
DESIGN EXAMPLE

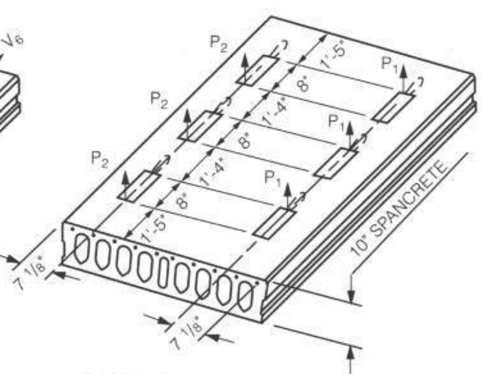
BOTTOM WELD PLATE TEST SETUP (WIRE LOOPS)





BWA-5





BWA-6

BWA-8

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia · *China* · *Denmark* · *Guatemala* · *Hungary* · *Ireland* · *Japan* · *Russia* · *South Korea* · *Switzerland*

Spancrete® hollowcore is a registered trademark

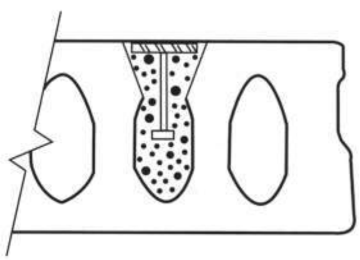
SPANCRETE MANUFACTURERS ASSOCIATION MEMBERS

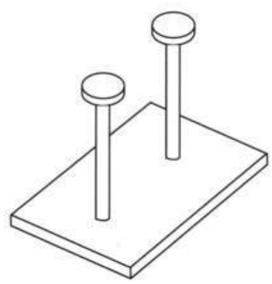
SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

TOP WELD ANCHORS

As part of a program for evaluating various inserts used in Spancrete® hollowcore plank, weld anchors were tested in the tops of some plank as shown below. These anchors were installed by cutting holes over cores in fully cured slabs, and then installing the insert in a fairly stiff sand and cement grout.





TOP WELD ANCHOR TESTED

CONCLUSIONS:

1. Holding capacity is reduced when anchor is located near any plank edge.

2. Weld plates grouted into a finished plank are an acceptable and economical way to locate a weldment in the Spancrete plank.

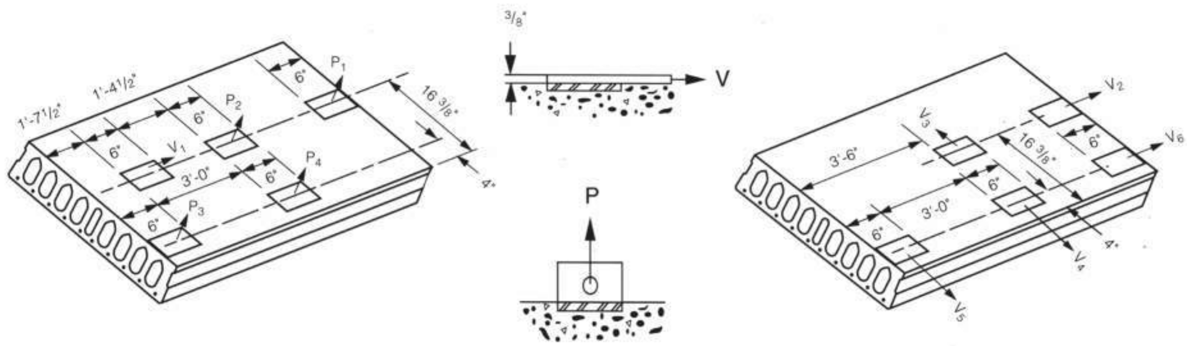
3. Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

Test results are given on the reverse side.

SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

TOP WELD ANCHOR TEST SETUP



Load	Test #1	Mode*	Test #2	Mode*	Avg.	Low
P_1	10.05^k	Conc.	8.31^k	Conc.	9.2^k	8.31^k
P_2	11.45	Conc.	10.16	Conc.	10.8	10.16
P_3	8.13	Conc.	7.13	Conc.	7.6	7.13
P_4	10.38	Conc.	6.90	Conc.	8.6	6.90
V_1	11.94	Conc.	16.61	Steel	14.3	11.94
V_2	5.71	Steel	8.04	Conc.	6.9	5.71
V_3	16.61	Conc.	24.47	Conc.	20.5	16.61
V_4	6.96	Conc.	4.32	Conc.	5.6	4.32
V_5	-	Conc.	3.39	Conc.	3.39	3.39
V_6	1.98	Conc.	6.21	Conc.	4.10	1.98

TEST RESULTS (kips)

*Failure mode

Note: Values given are test results. Appropriate strength reduction factors and safety factors must be applied to these values. Consideration should be given to the number of tests and the inconsistencies in results of the concrete failures when determining these factors.

This research was done using 40" wide, 8" thick Standard Spancrete® hollowcore. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

DOWEL CONNECTIONS

INTRODUCTION:

Tests were conducted on connections between Spancrete® hollowcore plank and masonry walls using a rebar driven into a hole drilled through the plank and into a bond beam. This is an economical connection that is occasionally used at lap and bearing conditions to resist relatively light loads. The connection is used for inward or outward pressures or to transfer diaphragm shear forces along the length of the wall.

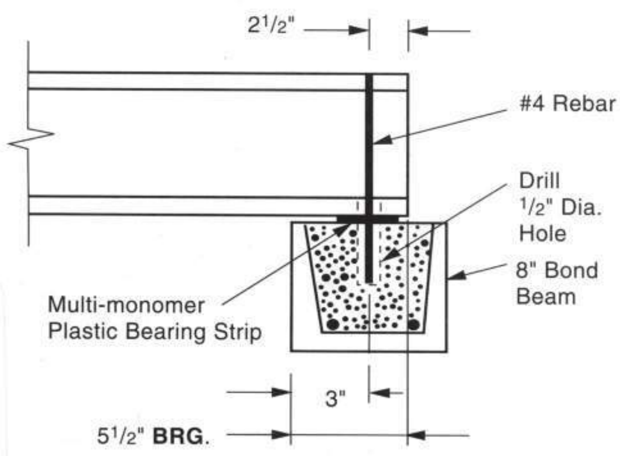


Figure 1 End Bearing

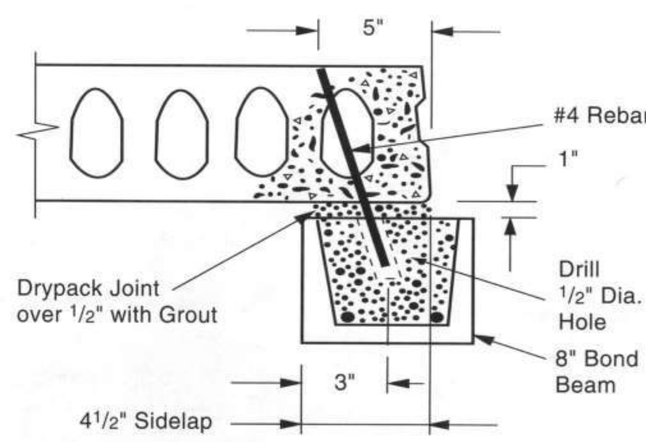


Figure 2 Sidelap

CONCLUSIONS:

1. Shear force can be transferred from Spancrete to a masonry wall with this connection.
2. Plank thickness is not a factor in the capacity of the connection. The results are applicable to all plank thicknesses.
3. The thickness of concrete below the cores in the tested plank represents the minimum currently in use. The results may therefore be conservatively applied to other plank cross sections.
4. The results presented are actual test data from a small number of samples. It is suggested that the lowest test value for each detail be used with appropriate ACI and PCI load factors and a strength reduction factor of at least .70.

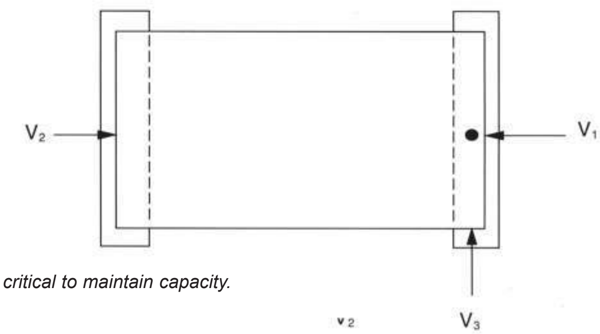
SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

The data listed are minimum test values obtained for shear in the direction noted. Appropriate load factors and strength reduction factors must be applied.

END BEARING DATA

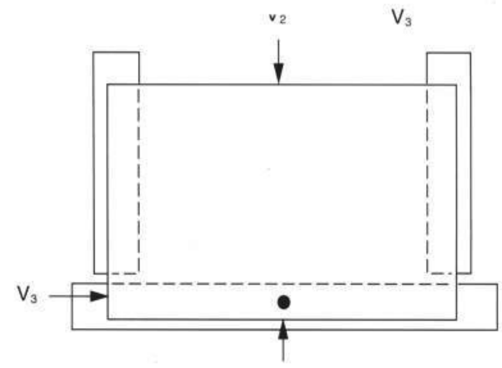
Load Direction	Minimum* V_{test} (KIPS)	Failure Mode
V_1	2.78	Concrete Cone
V_2	3.40	Concrete Cone
V_3	4.50	Rebar Yield



Note: *Dowel location in bond beam is critical to maintain capacity.*

SIDELAP DATA

Load Direction	Minimum* V_{test} (KIPS)	Failure Mode
V_1	1.87	Spalling bond beam face
V_2	2.96	Spalling bond beam face
V_3	2.66	Spalling bond beam face



Note: Actual test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values. The research was done using 48" wide, 8" thick Standard Spancrete® hollowcore. However, this concept applies to all Spancrete cross sections.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia, China, Denmark, Guatemala, Hungary, Ireland, Japan, Russia, South Korea, Switzerland

Spancrete® hollowcore is a registered trademark

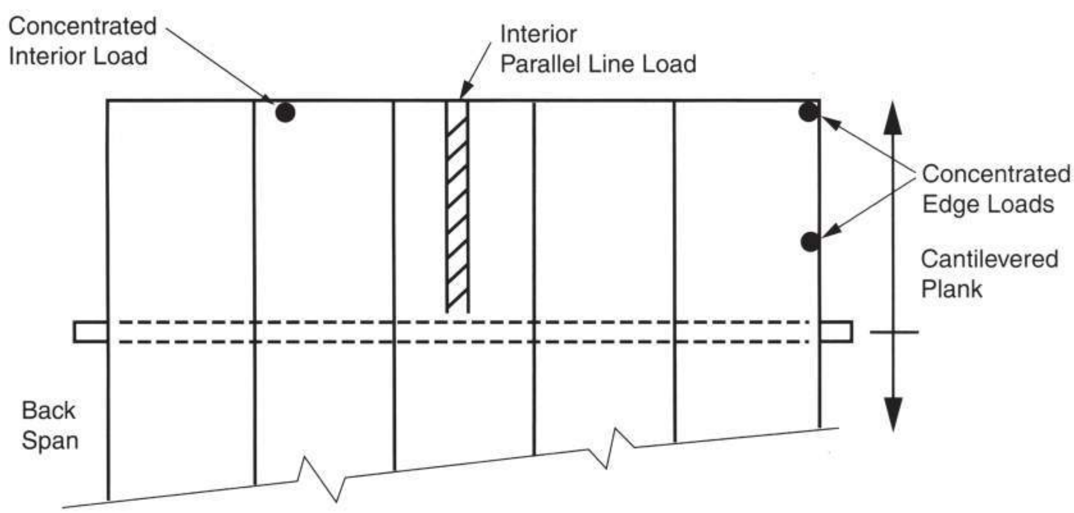
SPANCRETE MANUFACTURERS ASSOCIATION MEMBERS

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

CANTILEVER LOAD DISTRIBUTION

The Spancrete Manufacturers Association has been conducting research and testing since the 1960's to further the technical development of Spancrete® hollowcore design and enhance its use throughout construction. Using the considerable data accumulated from full scale testing on simple spans, we developed a three-dimensional computer analysis technique to model recommended cantilever load distribution widths for non-uniform loads.



CANTILEVER LOAD TYPES

CONCLUSIONS:

Non-uniform loads applied to Spancrete cantilevers may be distributed over effective widths that are a funciton of the load location and the cantilever length. The recommended distribution widths for parallel line loads and point loads are:

1.Edge loads	for moment	DW = 1 + 0.45 L ft
	for shear	DW = 1.5 ft
2.Interior loads	for moment	DW = 1.6 + 0.8 L ft
	for shear	DW = 1.5 + 0.2 L ft

where L = distance from support to point load or to end of parallel line load.

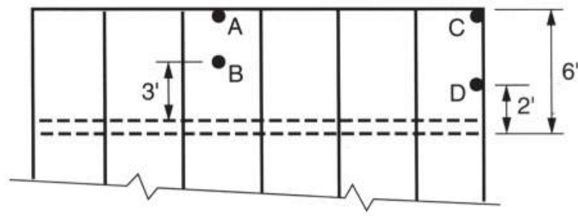
SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

CANTILEVER LOAD DISTRIBUTION

GIVEN:

An untopped 10" Spancrete® hollowcore layout as shown.
Assume plank weight = 75 psf,
superimposed dead load = 10 psf and
live load = 80 psf. Concentrated loads:
(A) P_d = 0.7 k P_L = 0.5 k
(B) P_d = 1.0 k P_L = 0.5 k
(C) and (D) P_d = 0.25 k P_l = 1.4 k



PROBLEM:

Determine the unit design shears and moments for the interior and edge conditions.

SOLUTION:

INTERIOR LOAD CONDITION		EDGE LOAD CONDITION
Point load at end of cantilever		Point load at end of cantilever
1.2(0.7) + 1.6(0.5) = 1.64 k	P_u	1.2(0.25) + 1.6(1.4) = 2.54 k
1.6 + 0.8(6) = 6.4 ft	DW_{mom}	1 + 0.45(6) = 3.7 ft
1.5 + 0.2(6) = 2.7 ft	DW_{shear}	1.5 ft
Intermediate point load		Intermediate point load
1.2(1.0) + 1.6(0.5) = 2.00 k	P_u	1.2(0.25) + 1.6(1.4) = 2.54 k
1.6 + 0.8(3) = 4 ft	DW_{mom}	1 + 0.45(2) = 1.90 ft
1.5 + 0.2(3) = 2.1 ft	DW_{shear}	1.5 ft
1.2(10 + 75) + 1.6(80) = 230 psf	W_u	1.2(10 + 75) + 1.6(80) = 230 psf
1.64(6)/6.4 + 2.00(3)/4 + .230 (6^2)/2 = 7.18 $^{ft\text{-}k/ft}$	M_u	2.54(6)/3.7 + 2.54(2)/1.9 + .230(6^2)/2 = 10.93 $^{ft\text{-}k}$/ft
1.64/2.7 + 2.00/2.1 + .230(6) = 2.94 ^{k}/ft	V_u	2.54/1.5 + 2.54/1.5 + .230 (6) = 4.77 ^{k}/ft

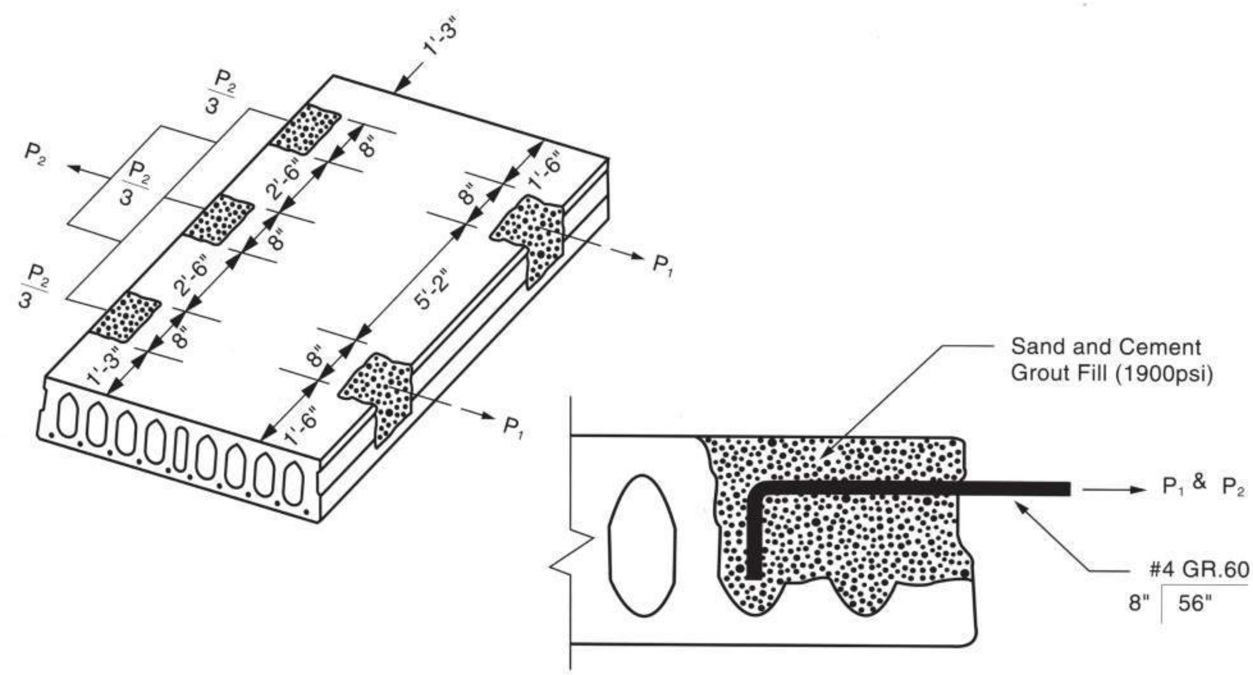
Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SIDE ANCHORS

As part of a program for evaluating various inserts used in Spancrete® hollowcore plank, anchors were tested in the sides of plank as shown below and on the reverse side. The type of anchor, installation, and test conditions are presented in each table. Interpretation for job applications, load interactions, appropriate strength reduction factors, and safety factors are left to the user.



TEST RESULTS (KIPS)

Load	Test #1	Mode*	Test #2	Mode*	Test #3	Mode*	Test #4	Mode*	Avg.	Low
P_1	9.23	Conc.	11.57	Conc.	12.23	Conc.	10.43	Conc.	10.87	9.2
P_2	38.5	Conc.	27.5	Conc.	-	-	-	-	33.0	27.5

*Failure Mode

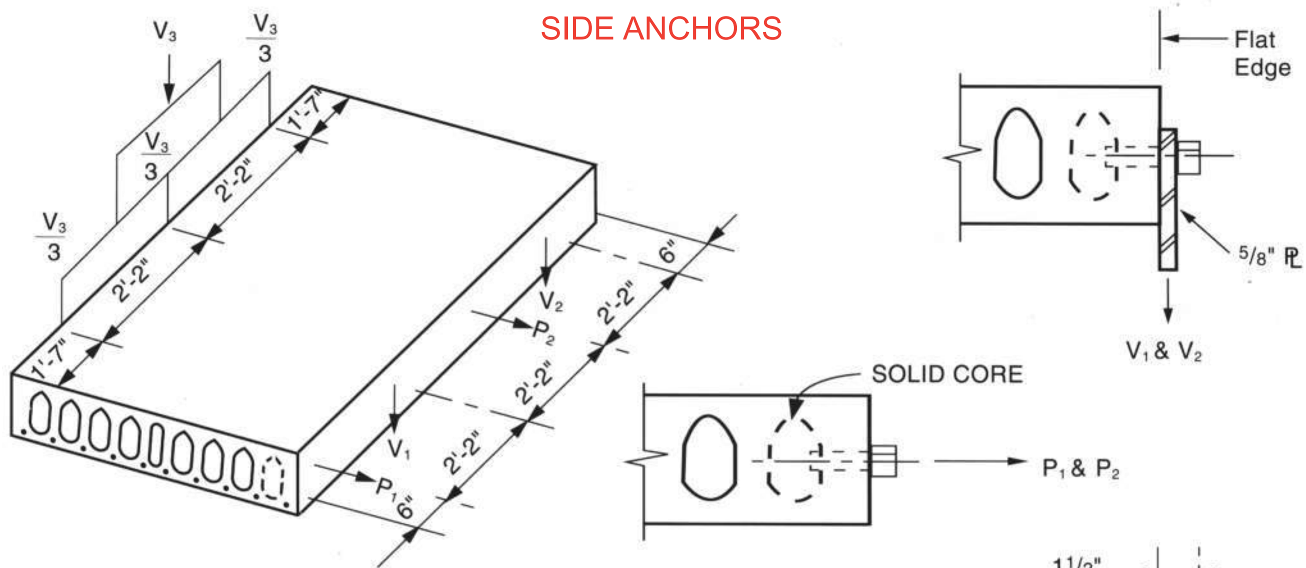
Values given are test results. Appropriate strength reduction factors and safety factors must be applied.

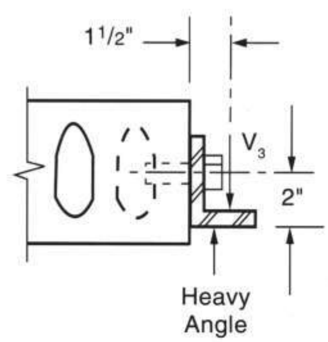
Additional test data is presented on the other side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

SIDE ANCHORS





3/4" dia. Sleeve Anchor
(contact your local producer for specific anchor information)

Test Results (KIPS)

Load	Test #1	Mode*	Test #2	Mode*	Avg.	Low
P_1	11.5	Conc.	11.68	Conc.	11.59	11.5
P_2	13.7	Conc.	11.25	Conc.	12.48	11.25
V_1	10.88	Conc.	11.32	Conc.	11.10	10.88
V_2	8.21	Conc.	8.29	Conc.	8.25	8.21
V_3	20.0	Conc.	17.10	Conc.	18.55	17.10

*Failure Mode

Values given are test results. Appropriate strength reduction factors and safety factors must be applied.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia *Ireland*
China *Japan*
Denmark *Russia*
Guatemala *South Korea*
Hungary *Switzerland*

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

CONCENTRATED LOADS ON UNTOPPED SPANCRETE® HOLLOWCORE DECKS

When Spancrete hollowcore plank shear keys are grouted, the resulting system has many of the characteristics of a monolithic plate. One such characteristic is the development of bending moments transverse to the span resulting from concentrations of load; since Spancrete is unreinforced in the transverse direction, we must place limits on this situation.

A series of tests were conducted to study how concentrated loads apply to a Spancrete system. Load location and bearing plate size were used as variables; transverse spacing of concentrated loads was not considered in this test series.

CONCLUSIONS:

1. There is no significant difference between placing loads over a grout key compared with placing loads within the center of a Spancrete unit.
2. The bearing plate size has little effect on the load capacity.
3. When two concentrated loads are placed in a line parallel to the span, a reduction in individual load magnitude is necessitated.

RECOMMENDED ONE OR TWO POINT CONCENTRATED LOAD LIMITS ON UNTOPPED SPANCERETE
(Working Loads)

SPANCRETE THICKNESS	4"	6"	8"	10"	12"	16"
Each Single Point Load	3.4	7.5	10.1	13.5	16.8	25.8
Each Double Point Load Spaced ≥ 0.5L	2.3	5.0	6.8	9.0	11.3	17.3
Each Double Point Load Spaced < 1'	1.7	3.7	5.0	6.7	8.4	12.9

Note:

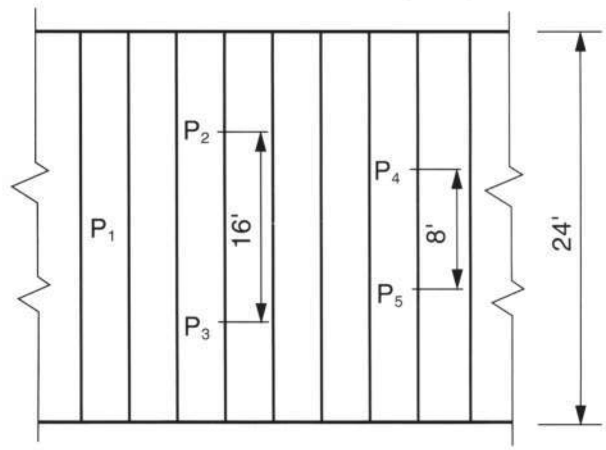
1. Values in each case are maximum recommended working loads using Ultralight Spancrete. Check with your local Spancrete manufacturer, as higher capacities than those shown may be available.
2. Values are based on a factor of safety of 2 and a ϕ factor of 0.9.
3. Values for 4", 6", 10", 12" and 16" plank are extrapolated and not verified by test.
4. Interpolation is allowed for double Point loads spaced between 1'-0" and 0.5L apart.

A design example is given on the reverse side.

SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

CONCENTRATED LOADS (UNTOPPED)



GIVEN:

8" Ultralight Spancrete® hollowcore system shown
$P_1 = 3.5^k$ DL and 2.5^k LL (working loads)
$P_2 = P_3 = 3.7^k$ DL and 2.0^k LL (working loads)
$P_4 = P_5 = 3.6^k$ DL and 2.2^k LL (working loads)

PROBLEM:

Evaluate the concentrated loads.

SOLUTION:

Case 1

$P_1 = 3.5^k + 2.5^k = 6.0^k$. This is less than the recommended concentrated load limit of 10.1^k for 8" Spancrete, and is acceptable. (See table on front side.)

Case 2

$P_2 = P_3 = 3.7^k + 2.0^k = 5.7^k$ The spacing of these loads is greater than 0.5L. Since each load is less than the 6.8^k recommended concentrated load limit for 8" Spancrete under this category, this case is acceptable.

Case 3

$P_4 = P_5 = 3.6^k + 2.2^k = 5.8^k$ The spacing of these loads is less than 0.5L. Interpolate between 5.0^k for spacing less than 1' and 6.8^k for spacing $\geq$ 0.5L.

Recommended load limit $= 6.8 - \frac{(12 - 8)}{12}(6.8 - 5.0) = 6.2^k$ for each load.

Since each load is less than the 6.2^k recommended concenrated load limit for 8" Spancrete under this category, this case is acceptable.

(See Research Note entitled **"LOAD DISTRIBUTION"** for effective distribution width).

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia
China
Denmark
Guatemala
Hungary
Ireland
Japan
Russia
South Korea
Switzerland

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

CONCENTRATED LOADS ON TOPPED SPANCRETE® HOLLOWCORE DECKS

In tests conducted on untopped Spancrete assemblies, concentrated loads were found to induce transverse bending moments in addition to moments in the direction of plank span. Limits were established for concentrated loads so that transverse bending would not control a design. This information is contained in the Research Note entitled CONCENTRATED LOADS ON UNTOPPED SPANCRETE DECKS.

One of the parameters affecting the capacity to resist splitting due to concentrated loads was the transverse section modulus. Addition of a composite structural topping increases the transverse section modulus, and therefore should increase the resistance to concentrated loads. Load tests were conducted using 48" wide* Ultralight Spancrete to verify this anticipated increase in strength.

CONCLUSIONS:

1. Concentrated load capacity was increased with the addition of a structural topping in proportion to the increase in transverse section modulus.

2. Concentrated load capacity was not significantly affected by the amount or type of reinforcing used in the topping.

RECOMMENDED CONCENTRATED WORKING LOAD LIMITS FOR SPANCRETE WITH STRUCTURAL COMPOSITE TOPPING

Deck Thickness:	4"+2"	6"+2"	8"+2"	10"+2"	12"+2"	16"+2"
Single Point Load	5.9	10.3	12.7	15.9	19.2	28.8
Each Double Point Load Spaced ≥ 0.5L	4.0	6.8	8.5	10.6	12.9	19.3
Each Double Point Load Spaced < 1'	3.0	5.1	6.3	7.9	9.6	14.4

Note:

1. Values are based on a factor of safety of 2 and a ϕ factor of 0.9.
2. Values for 4", 6", 10", 12" and 16" plank are extrapolated and not verified by test.
3. Interpolation is allowed for double point loads spaced between 1' and 0.5L apart.
4. Topping is 4000 psi at 28 days with E = 3000 ksi.
5. *Slab width will affect load limits; consult your local producer regarding other sections.
6. Minimum recommended bearing area under a concentrated load is 4" x 4".

A design example is given on the reverse side.

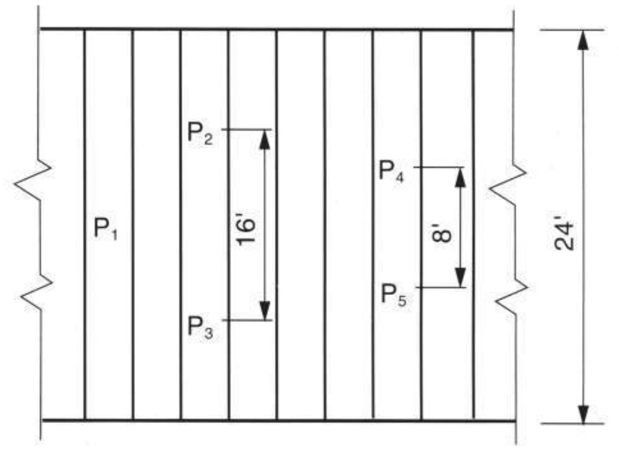
SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

CONCENTRATED LOADS (TOPPED)

GIVEN:

8" Topped Ultralight Spancrete® hollowcore system shown
P_1 = 3.5^k DL and 2.5^k LL
P_2 = P_3 = 4.7^k DL and 3.0^k LL
P_4 = P_5 = 4.6^k DL and 3.1^k LL



PROBLEM:

Evaluate the concentrated loads for application on 8" Spancrete with a 2" structural topping.

SOLUTION:

Case 1

P_1 = 3.5^k + 2.5^k = 6.0^k. This is OK, since recommended concentrated load limit is 12.7^k.
(See table on front side.)

Case 2

Spacing of P_2 and P_3 at 16' is greater than 0.5L. P_2 = P_3 = 4.7^k + 3.0^k = 7.7^k This is OK, since recommended concentrated load limit is 8.5^k.

Case 3

Spacing of P_4 and P_5 at 8' is less than 0.5L. Interpolate between 6.3^k for < 1' spacing and 8.5^k for ≥ 0.5L.

$$\text{Recommended load limit} = 8.5 - \frac{(12 - 8)}{12}(8.5 - 6.3) = 7.8^k$$

P_4 = P_5 = 4.6^k + 3.1^k = 7.7^k. This is also OK, since it is less than the introplated load limit.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

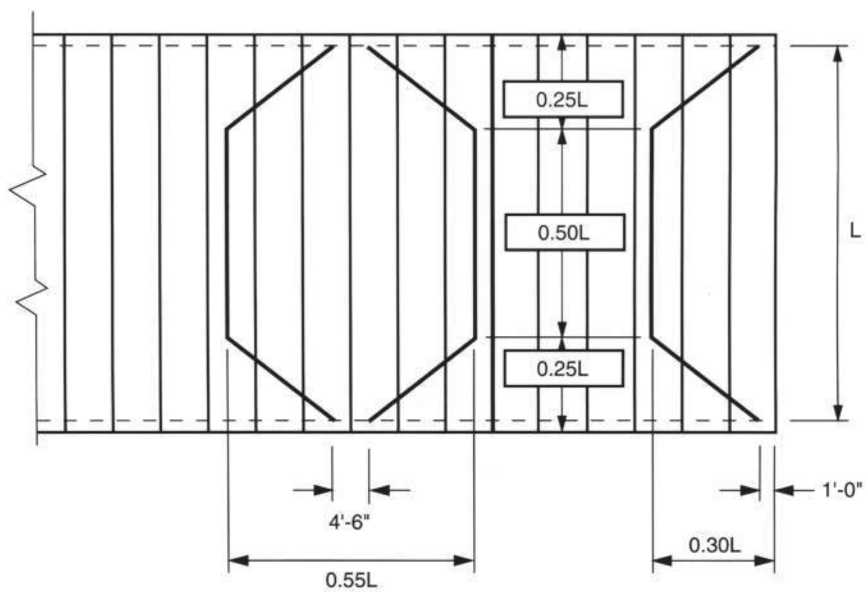
SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

LOAD DISTRIBUTION

The Spancrete Manufacturers Association sponsored extensive research on the behavior of Spancrete® hollowcore plank subjected to non-uniform loads in the form of line loads and point loads, and the effect of openings.

It was determined from testing that non-uniform loads are resisted by multiple slabs and can be simply represented as a varying width of section, as shown below. This effective resisting section is used to determine peak moments and shears for design. The design method is similar to that presented in the PCI Design Handbook and the PCI Manual for the Design of Hollowcore Plank, except that testing on Spancrete plank established greater effective distribution widths as shown below. The relationship shown is applicable when the width to span ratio of the plank assembly is greater than 1.0.



Effective Distribution Area

A design example is given on the reverse side.

SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

LOAD DISTRIBUTION

GIVEN:

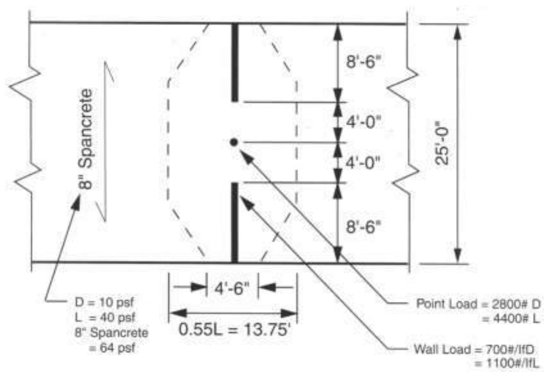
8" Spancrete® hollowcore floor with loads as shown.

PROBLEM:

Determine the design loads, and check shear and flexure, for the plank example shown.

SOLUTION:

Flexural design is critical at midspan; use the maximum distribution width to find an equivalent uniform load



DW = 0.55L = 13.75 ft

Uniform: w = 10 + 40 = 50 psf   Wall: w = (700 + 1100) ÷ 13.75 = 131 psf

$M_w = (131 \times 8.5^2) \div 2 = 4732$ ft#/ft $w_w = (8 \times 4732) \div 25^2 = 60.6$ psf

Point Load: $w_p = 2\,(2800 + 4400) \div (25 \times 13.75) = 42$ psf Total Equivalent Uniform Load = 50 + 61 + 42 = 153 psf

Use Spancrete series 8610 (3/4" clear cover, 10-3/8" 250 KSI strands)

Shear design is normally first evaluated at h/2 from the support.

DW = 4.5 + 0.333 (0.55L - 4.5) ÷ 6.25 = 4.99 ft. Use this width to distribute loads

Uniform: $W_D = 10$ psf $w_L = 40$ psf Wall: $W_D = 700 \div 4.99 = 140$ psf $w_L = 1100 \div 4.99 = 220$ psf

Point: $P_D = 2800 \div 4.99 = 561$ #/ft $P_L = 4400 \div 4.99 = 882$ #/ft

Checking shear across the span using these distributed loads, we find that V_u is slightly greater than ϕV_c at h/2 ($\Delta V_u = 1.10$ k). The web shear capacity at this location can be increased by grouting cores (See Research Note 1007, "Shear Strength With Filled Cores").

At x = 2.38, the shear capacity is also exceeded, but the loads can be recalculated using the wider distribution width at this location: DW = 4.5 + 2.38(13.75 - 4.5)/6.25 = 8.02 ft. Using this width, the revised loadings are:

Wall:	$w_{dl} = 700/8.02 = 87$ PSF	$w_{ll} = 1100/8.02 = 137$ PSF
Point Load:	$P_{dl} = 2800/8.02 = 349$ PLF	$P_{ll} = 4400/8.02 = 549$ PLF

Recheck shear and find that $V_u < \phi V_c$ at 2.38 ft. and at all points in the span beyond.

Additional information for Shear Design is provided in Research Note titled, "SHEAR STRENGTH".

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete. This research was done using 40" wide, 8" thick Standard Spancrete. However, this concept applies to all Spancrete cross sections.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia	Ireland
China	Japan
Denmark	Russia
Guatemala	South Korea
Hungary	Switzerland

Spancrete® hollowcore is a registered trademark

SPANCRETE MANUFACTURERS ASSOCIATION MEMBERS

MKSMARN-2010

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SPAN LIMITATIONS

Floor Vibrations - Rhythmic Activity

Earlier Research Notes resulted from tests conducted on Spancrete® hollowcore sections. This issue is presented as an empirical guide to analyze floor vibrations caused by a specific type of activity.

Floor vibrations due to rhythmic activities, such as aerobics or dancing, have long been a problem in steel construction. There have been few problems in precast concrete construction, but as spans get longer, questions occasionally arise. Here are the required steps to evaluate a Spancrete floor system for vibration performance.

Step 1. Determine the activity causing the vibration and the activity of the occupants affected by the vibration.

Step 2. Calculate the natural frequency of the floor plank.

$$f_o = \frac{0.74}{L^2}\sqrt{\frac{EI}{wb}}$$

(for simple spans only)

where
- L = span, ft.
- I = slab moment of inertia, $in.^4$
- w = uniform slab weight, psf
- E = modulus of elasticity, psi
- b = slab width, ft.

Step 3. Establish the acceptable acceleration based on the activity of the affected occupants. A sample table is given with a design example on the reverse side.

Step 4. Determine the factors required based on the activity causing the vibrations. A sample table of functions is given with a design example on the reverse side.

Step 5. Calculate the minimum required natural frequency.

$$f_{omin} = f_i\sqrt{\left(1 + \frac{k}{a_o/g}\,DLF\,\frac{w_p}{w}\right)}$$

where
- f_{omin} = minimum floor natural frequency
- f_i = forcing frequency from activity
- k = factor for modes of vibration
- DLF = dynamic load factor
- w = weight of participants plus floor
- a_o/g = acceptable acceleration
- w_p = weight of participants

Step 6. Compare results. The floor plank will be acceptable if the natural frequency provided as determined in Step 2 is greater than that required from Step 5.

Note: The support system can have a significant influence on performance, and may require a separate analysis. This presentation assumes a rigid support, such as a wall bearing structure. See RESEARCH NOTE “Floor Vibrations - Spancrete on Flexible Supports” for additional information.

A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

SPAN LIMITATIONS

Floor Vibrations - Rhythmic Activity

GIVEN:

Acceptable Acceleration

Affected Occupancy	a_o/g
Office & Residential	0.004 to 0.007
Dining	0.015 to 0.025
Weightlifting	0.015 to 0.025
Rhythmic Activity	0.040 to 0.070

Forcing Functions

Activity	w_p	f_i	k	DLF
Dancing	12.5 psf	1.5 to 3.0	1.3	0.2 to 0.5
Aerobics	4 psf	2 to 2.75	2.0	1.5 (1st harmonic)
		4 to 5.5	2.0	0.6 (2nd harmonic)
		6 to 8.25	2.0	0.1 (3rd harmonic)
Stadium	30 psf	3	1.3	0.25 (1st harmonic)
		5.5	1.3	0.05 (2nd harmonic)

PROBLEM:

Determine if 8" Spancrete® hollowcore with a unit weight of 60 psf will result in acceptable vibrations on a 22 ft. span with aerobics as both the forcing function and the response occupancy.

SOLUTION:

Step 1. Aerobics, in this case, is both the causal and affected activity.

Step 2. The natural frequency provided is

$$f_o = \frac{0.74}{L^2}\sqrt{\frac{EI}{wb}} = \frac{0.74}{(22^2)}\sqrt{\frac{4300000(1730)}{60(4)}} = 8.51 \text{ Hz}$$

Step 3. Use a_o/g = 0.055 as acceptable vibration (midrange in table for this activity)

Step 4. From the Forcing Functions table use: f_i = 2.5, 5, 7 for the 1st, 2nd, and 3rd harmonics
k = 2.0 DLF = 1.5, 0.6, 0.1 for the 1st, 2nd, and 3rd harmonics w_p = 4 psf (thus w = 64 psf)

$$f_{omin} = f_i\sqrt{\left(1 + \frac{k}{a_o/g}\,DLF\,\frac{w_p}{w}\right)}$$

1st harmonic $f_{omin} = 2.5\sqrt{\left(1 + \frac{2.0}{0.055}(1.5)\frac{4}{64}\right)} = 5.25\text{Hz}$

Similarly, calculate

2nd harmonic f_{omin} = 7.69Hz
3rd harmonic f_{omin} = 7.75Hz

Conclusion: The natural frequency provided, 8.5 Hz, is greater than that required in all three harmonics. Therefore, 8" Spancrete will provide an acceptable floor.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SPAN LIMITATIONS

Floor Vibrations - Heel Drop Response

As the use of longer spans and larger open areas has become more prevalent in office construction, awareness of floor vibrations has increased. The effect that one person walking on a floor has on others sitting at desks has been studied primarily in the steel construction industry. While few problems have been reported in the precast concrete industry, methods of analysis are available for evaluating Spancrete® hollowcore. For this type of floor vibration, the minimum required damping can be determined for specific conditions and compared to a probable expected damping.

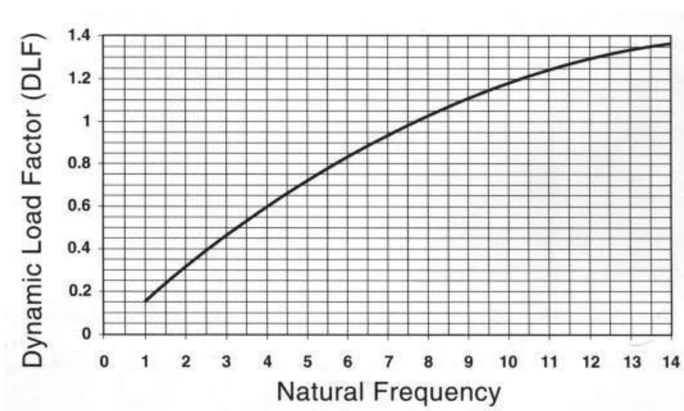
Expectations of actual damping are generally a funciton of non-structural items such as ceilings and partitions. While bare Spancrete has approximately 3% damping, a suspended ceiling could add 1 to 3% damping and partitions could add 5 to 10% damping. The design procedure follows.

Calculate the required minimum damping.

$$D_{min} = 35\,A_o f_o + 2.5 \quad \text{where} \quad A_o = DLF\left(\frac{0.6\,l^3}{48\,EI}\right) \text{and } f_o = \frac{0.74}{L^2}\sqrt{\frac{EI}{wb}}$$

f_o = system natural frequency, Hz
E = modulus of elasticity, ksi
l = span, in.
DLF = dynamic load factor
I = moment of inertia, in.^{4}
L = span, ft.
w = slab weight, psf
b = slab width, ft.

(For A_o, the moment of inertia to be used should be that available in a design strip such as 0.55L.)
(For f_o, the moment of inertia to be used is that of the plank.)



When the required damping is less than about 3%, a bare floor will normally be found acceptable. When the required damping is in excess of 4.5%, particular care should be taken in identifying sources of damping.

Note: The support system can have a significant influence on performance, and may require a separate analysis. This presentation assumes a rigid support, such as a wall bearing structure. See RESEARCH NOTE "Floor Vibrations - Spancrete on Flexible Supports" for additional information.

A design example is given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

SPAN LIMITATIONS

Floor Vibrations - Heel Drop Response

PROBLEM:
Check the minimum damping required for 8" Spancrete® hollowcore with a unit weight of 60 psf on a 28 ft. span in an office floor application.

GIVEN:
The help drop dynamic load factor can be determined from the figure on the other side.

SOLUTION:

The plank natural frequency is: $f_o = \frac{0.74}{28^2}\sqrt{\frac{4300000(1730)}{60(4)}} = 5.25$ Hz

From the figure on the other side, heel drop DLF = 0.75

Using a 0.55L distribution width, the total moment of inertia is:

$I = 0.55(28)(1730/4 \text{ ft.}) = 6660.5 \text{ in.}^4$, and $Ao = 0.75\left(\frac{0.6(28x12)^3}{48(4300)(6660.5)}\right) = 0.012$ in.

Minimum damping $D_{min} = 35(0.012)5.25 + 2.5 = 4.7\%$

Conclusion: In this example, a suspended ceiling and regularly spaced partitions would be recommended to increase the damping provided.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SPAN LIMITATIONS

Floor Vibrations - Flexible Supports

Calculation of floor vibration characteristics for Spancrete® hollowcore plank on rigid supports was addressed in RESEARCH NOTES 1021 and 1022. When Spancrete is supported on flexible supports, those supporting members will reduce the natural frequency of the system as compared to the Spancrete alone. Therefore, it is appropriate to include the effects of flexible support members when the system is to be evaluated for vibration criteria.

For floor plank combined with suport beams, the natural frequency of the system can be approximated by:

$$\frac{1}{f^2_n} = \frac{1}{f^2_p} + \frac{1}{f^2_b}$$

where f_n = system natural frequency, Hz
f_p = Spancrete plank natural frequency, Hz
f_b = support beam natural frequency, Hz

Where a bay of plank is being investigated and the supporting beams at each end are different, the beam with the lower natural frequency should be used to determine the system natural frequency.

The individual natural frequencies are calculated as:

$$f_p = \frac{0.74}{L^2}\sqrt{\frac{EI}{wb}} \quad \text{for the plank, and} \quad f_b = \frac{0.74}{L^2_b}\sqrt{\frac{E_bI_b}{w_b}} \quad \text{for the beam}$$

where

L, L_b = respective span, ft
b = plank width, ft
w_b = uniform beam load, plf
E, E_b = respective modulus of elasticity, psi
I, I_b = respective moment of inertia, in^4
w = uniform plank weight, psf

The system natural frequency can then be used instead of the slab natural frequency for the evaluation procedures presented in RESEARCH NOTES 1021 and 1022.

A design example is given on the reverse side.

SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

SPAN LIMITATIONS

Floor Vibrations - Flexible Supports

GIVEN:
Two bays of 22' long 8" Spancrete® hollowcore weighing 60 psf supported by a beam with spans 20'.

PROBLEM:
Determine the natural frequency of the system when the support beam is:
(1) a steel beam, w18 x 50 (2) a precast concrete beam, IT 30 x 20

SOLUTION:

(1) For plank $f_p = \frac{0.74}{L^2}\sqrt{\frac{EI}{wb}} = \frac{0.74}{(22^2)}\sqrt{\frac{4{,}300{,}000\ (1730)}{60(4)}} = 8.51 \text{ Hz}$

For steel beam $f_b = \frac{0.74}{L_b^2}\sqrt{\frac{E_b I_b}{w_b}} = \frac{0.74}{(20^2)}\sqrt{\frac{29{,}000{,}000\ (800)}{22\ (60) + 50}} = 7.61 \text{ Hz}$

For system $\frac{1}{f_n^2} = \frac{1}{f_p^2} + \frac{1}{f_b^2} = \frac{1}{8.51^2} + \frac{1}{7.61^2} = 0.0311$ and $f_n = \sqrt{\frac{1}{0.0311}} = 5.67 \text{ Hz}$

(2) For precast beam $f_b = \frac{0.74}{(20^2)}\sqrt{\frac{4{,}700{,}000\ (1650)}{20\ (60) + 542}} = 11.84 \text{ Hz}$

For system $\frac{1}{f_n^2} = \frac{1}{8.51^2} + \frac{1}{11.84^2} = .0209$ and $f_n = 6.91 \text{ Hz}$

Conclusion: Use results for the total system frequencies in place of simple plank frequencies in the evaluation techniques in RESEARCH NOTES 1021 and 1022.

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

LOAD-BEARING SPANCRETE® HOLLOWCORE WALL PANEL DESIGN

A series of tests were conducted to evaluate the Load-Bearing capacity of vertical Spancrete hollowcore wall panels with particular attention to slenderness effects. Slenderness ratios of 93.5 and 151 were tested. The tests were set up to apply a variable axial load with eccentricity at one end. A constant lateral load was applied at midheight to simulate wind load on the panel and establish an initial panel deflection.

The results were evaluated using ACI moment magnifier provisions, a modified moment magnifier approah from the PCI Design Handbook, and a second order, or P-delta, analysis.

CONCLUSIONS:

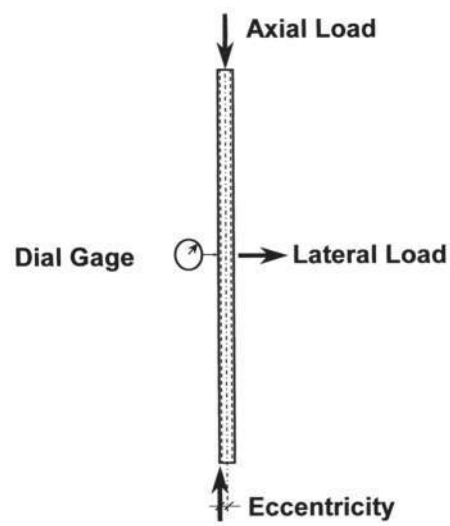
1. Test capacities exceeded those predicted by the ACI moment magnifer approach, even though the ACI slenderness limit of 100 was exceeded.

2. The modifications to EI in the PCI Handbook were found to result in very erratic strength predictions and are not recommended for use.

3. A prediciton of capacity based on a second order analysis was found to be in good agreement with the test results.

4. In comparison to a test condition, design applications must account for reduction in stiffness due to sustained loads. Also, a strength reduction factor, ⌀, must be applied.

5. A second order analysis should be used to evaluate slenderness effects when designing Load-Bearing Spancrete wall panels.



Wall Panel Test Set-up

6. The ultimate tensile stress in the concrete should be maintained at a level below cracking for slender panels. While the effects of cracking can be evaluated, second order deflections tend to increase dramaticaally after cracking.

A design example is given on the reverse side.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

LOAD-BEARING SPANCRETE® HOLLOWCORE WALL PANEL DESIGN

GIVEN:

An 8" Spancrete hollowcore wall panel, 8' wide and 30' high, carrying axial loads of 16 k per panel dead load and 8 k per panel roof live load, both at an eccentricity of 3" from the panel centroid. The wind load is 25 psf. Assume the panel has an initial bow = 1". The panel moment of inertia = 3629 in^4.

PROBLEM:

Design the panel for this Load-Bearing condition.

SOLUTION:

For the load combination 1.2D + 1.6W + .5Lr, the design forces are:

P_u = 23.2 k and w_u = 0.320 k/ft. Calculate first order deflections:

Due to axial load: $\Delta_1 = \dfrac{Ml^2}{16EI} = \dfrac{23.2(3)(30*12)^2}{16EI}$ Due to wind load: $\Delta_1 = \dfrac{5wl^4}{384EI} = \dfrac{5(0.320)(30)^4 1728}{384EI}$

Modify EI, per ACI for sustained loads and stiffness reduction:

$ßd = \dfrac{1.2*16}{(1.2*16 + .5*8)}$ = 0.83 for axial load and B_d = 0 for wind load Then $EI = \dfrac{0.7(4300)(3629)}{(1 + 0.83)}$ = 5.97 x 10^6 for axial load

and $EI = \dfrac{0.7(4300)(3629)}{(1 + 0)}$ = 10.92 x 10^6 for wind load, where 0.7 = ACI stiffness reduction for second order analysis.

Thus the first order deflection Δ_1 = 0.09" axial + 0.53" wind + 1.0" bow = 1.62"
The second order deflection is due to the axial load acting on a panel that is deflected. The instantaneous and sustained deflections are separated. With dead load and bowing considered sustained.

Dead load portion = $\dfrac{1.2(16)}{1.2(16)+.5(8)}$ = 0.83 Live Load portion = 1 - 0.83 = 0.17

Δ_1 sustained = 0.83 (0.09) +1 = 1.07" Δ_1 instantaneous = 0.17 (0.09) + .53 = .55"
For the simple case of a panel braced at the top and bottom only:
Final $M_u = M_{u1} + [P\Delta_1 / (1 - Pl^2 / \pi^2 EI)]$
First order M_u = [23.2(3)/2(12)] + $0.32(30)^2/8$] = 38.9 ft-k/panel

$$\text{Final } M_u = 38.9 + (23.2)(1.07/12) / \{1 - [23.2(30x12)^2 / (\pi^2)(5.97)(10^6)]\}$$
$$+ (23.2)(.55/12) / \{1 - [23.2(30x12)^2 / (\pi^2)(10.92)(10^6)]\} = 42.2 \text{ ft-k/panel}$$

Since the gross moment of inertia was used to calculate deflections, the tension stress at factored and magnified moment must be checked. M_u/S = 42.2(12)(3.98)/3629 = 0.555 ksi. The panel will be uncracked with a minimum prestress of 225 psi. Therefore the original assumption is validated. The panel design is completed by selecting a prestress pattern which provides the axial and flexural strength required.

Sample calculations are intended only to illustrate the concept presented and do not represent all considerations necessary for a complete design.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

LOAD-BEARING TOP CONNECTION DESIGN FOR SPANCRETE® HOLLOWCORE WALL PANELS

When Spancrete hollowcore wall panels are used in a Load-Bearing situation, the roof load is typically applied to the inside face of the panels. Connections are required to transfer the load into the panel. A series of tests was conducted to determine the load transfer capacity of a channel insert cast into the inside surface of a panel. Variables in the test program included load eccentricity, channel length, channel anchorage and proximity to the top end of the panels.

The channels were C4 x 5.4 standard rolled sections. The anchorage used allows the inserts to be tied to the strands on the casting bed so the Spancrete machine can run over the channels without mechanical interference. The anchorage consisted of 2 - #3 rebars extending 8 inches beyond each end of the channel. The bars were welded to the tips of the channel flanges as shown in the figure below.

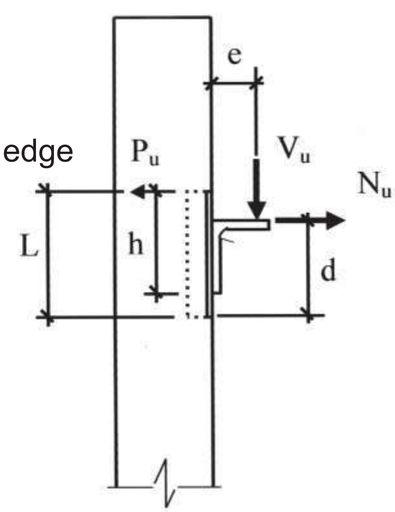
CONCLUSIONS:

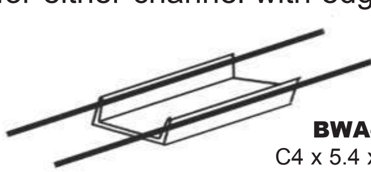
There is not a significant interaction of shear and tension with eccentric shear loads. Therefore, the following recommendations are made for channel capacity:

$V_u \leq \varnothing V_n$ and $P_u \leq \varnothing P_n$

where $P_u = \dfrac{V_u e}{h} + \dfrac{N_u d}{L}$ using the figure below

where V_n = 34k for 12" or 18" channel loaded away from free edge
and P_n = 3.75k for 12" channel with 2" to 8" edge distance
= 4.75k for 18" channel with 2" to 8" edge distance
= 5.10k for either channel with edge distance > 8"





BWA-6
C4 x 5.4 x 1'-0"
w/2 - #3 Bars

A design example is given on the reverse side.

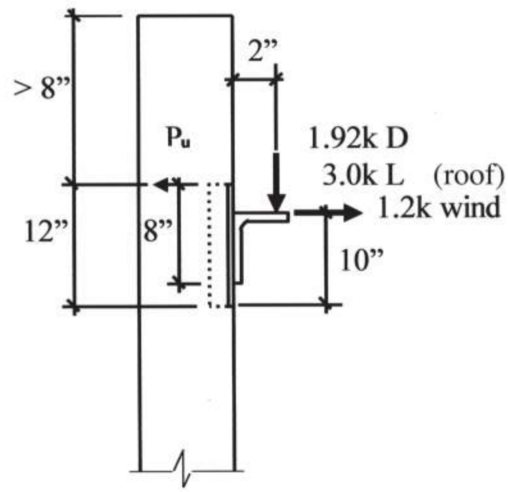
Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

LOAD-BEARING TOP CONNECTION DESIGN FOR SPANCRETE® HOLLOWCORE WALL PANELS

GIVEN:
Details and loading conditions as shown



PROBLEM:
Check capacity of 12" long channel anchor

SOLUTION:
For dead and live loads only.

$V_u = 1.2(1.92) + 1.6(3.0) = 7.10k$

$P_u = 7.10 (2) / 8 = 1.78k$

Applying the recommended PCI connection factor,

$V_u = 1.3(7.10) = 9.23k$ [$< \varnothing V_n = 0.70(34) = 23.8k$, OK]

$P_u = 1.3(1.78) = 2.31k$ [$< \varnothing P_n = 0.70(5.1) = 3.57k$, OK]

For dead plus live plus wind loads,

$V_u = (1.2)(1.92) + (.5)(3.0) = 3.80k$

$$P_u = \frac{3.80(2)}{8} + \frac{(1.6)(1.2)(10)}{12} = 2.55k$$

With the connection factor,

$V_u = 1.3(3.80) = 4.94k$ [$< \varnothing V_n = 0.70(34) = 23.8k$, OK]

$P_u = 1.3(2.55) = 3.32k$ [$< \varnothing P_n = 0.70(5.1) = 3.57k$, OK]

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

NON-BEARING TOP CONNECTION DESIGN FOR SPANCRETE® HOLLOWCORE WALL PANELS

Vertical Spancrete hollowcore wall panels in a non-Load-Bearing condition must be connected to a roof structure to tie the panels to the building. Additionally, the panels may be required to act as shear walls for the building. The top non-Load-Bearing connections may then be subjected to either normal forces or in-plane forces. At the same time, the roof structure will move vertically due to gravity loads or thermal gradients. The top, non-Load-Bearing connection should allow vertical slip so unintended vertical forces are not transferred into the panels.

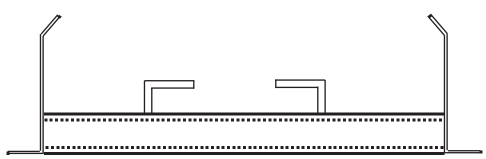
Tests were conducted on a specific insert to determine load capacities for both direct tension and in-plane shear. The insert selected was a Unistrut P3300 with special end caps and additional stud anchorage as shown in the figure below.

CONCLUSIONS:

1. When used with the notched steel strap shown, the insert will allow vertical movement, yet can also transfer both in-plane and out of plane forces.

2. The test capacity achieved was 2.75k in tension and 9k in shear.

3. A strength reduction factor of 0.70 and an extra conneciton factor of 1.3 are recommended for use with this insert.



P3300 Unistrut x 8" Long
with Special End Caps and
2-3/8" x 2-1/2" Studs

Notched Strap
3/8" x 1-3/8"
(2" wide strap sim.)

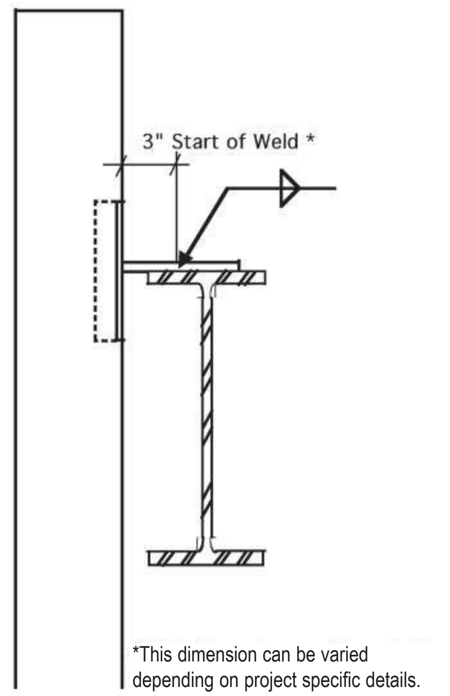
A design example is given on the reverse side.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

NON-BEARING TOP CONNECTION DESIGN FOR SPANCRETE® HOLLOWCORE WALL PANELS



GIVEN:

Wind Normal = .9 k
Wind Parallel = 1.4 k

PROBLEM:

Design the non-Load-Bearing connection.

SOLUTION:

For wind normal to the connection, $T_u = 1.6(.9) = 1.44$ k; applying the extra connection factor as recommended,
$T_u = 1.3(1.44) = 1.87\ k < \phi T_n = 0.7(2.75) = 1.93\ k$ OK

Check capacity of the strap in tension at the notch:
$\phi T_n = 0.9(36)(0.75)(0.375) = 9.11\ k > 1.44\ k$ OK

Check strap compressive capacity for 3" unbraced length:
$\frac{Kl}{r} = \frac{2(3)}{0.3(0.375)} = 53$; from AISC LRFD, $F_{au} = 26.39$ ksi;
Therefore, $\phi C_u = 0.375(1.375)(26.39) = 13.6\ k > 1.44\ k$ OK

For wind parallel to the connection, $V_u = 1.6(1.4) = 2.24$k; applying the extra connection factor as recommended,
$V_u = 1.3(2.24) = 2.91\ k < \phi V_n = 0.7(9) = 6.3\ k$ OK

At strap notch, $\phi V_n = 0.9(0.6)(36)(0.375)(0.75) = 5.47\ k > 2.24\ k$ OK

Check bending of strap for 3" moment arm to start of weld:
$M_u = 2.24(3) = 6.72$ in-k. Then req'd $Z = 6.72 / 0.9(36) = 0.207$. Therefore the width required = $\sqrt{0.207(4)/0.375} = 1.49$ in. Use a 2" wide strap.

Design the amount of weld to beam: try 3/16" fillet weld 2" long. $f_{vux} = 2.24/2(2) = 0.56$ k/in and $f_{vuy} = 2.24(3+1) / 2(2) = 2.24$ k/in $R_{wu} = \sqrt{(0.56^2 + 2.24^2)} = 2.32$ k/in < 4.18 k/in OK

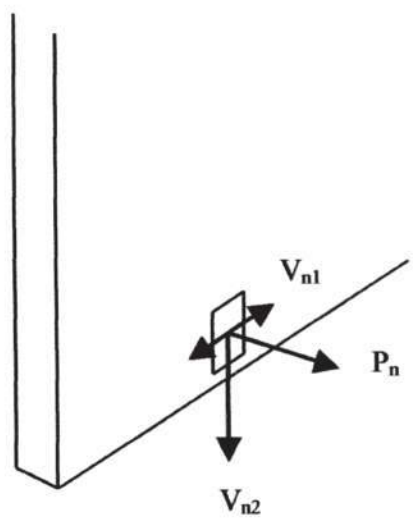
Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

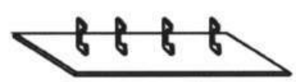
BASE CONNECTION DESIGN FOR SPANCRETE® HOLLOWCORE WALL PANELS

Spancrete hollowcore wall panels used in a vertical orientation require connections to the foundation. These connections may be subjected to normal forces due to wind load and eccentric axial load, or uplift and in-plane forces if acting as a shear wall. Where applicable, the base connections may also have to satisfy the structural integrity provisions of AC1 318.

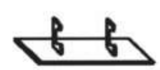
Spancrete wall panels are typically welded to a continuous or spaced clip angle that has been welded or properly bolted to the foundation. The SMA has tested a number of weld anchor assemblies for use as the panel weld anchor in such a base detail. Three are shown below, with their test capacities.

Insert	V_{n1}	V_{n2}	P_n
BWA-1	15.17k	5.62k	3.30k
BWA-3	9.31k	4.66k	2.52k
BWA-6	31.04k	13.0k	5.34k

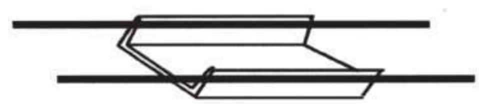




BWA-1
PL 3/8" x 4" x 6"
w/ 4 Erico R6 Studs



BWA-3
PL 3/16" x 2" x 6"
w/ 2 Erico R6 Studs



BWA-6
C4 x 5.4 x 0'-6"
w/ 2 - #3 Bars

It is recommended that a strength reduction factor of 0.70 and an additional connection load factor of 1.3 be used with these test values.

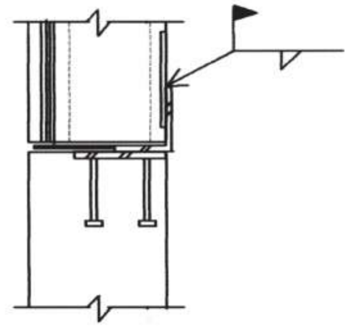
A design example is given on the reverse side.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

BASE CONNECTION DESIGN FOR SPANCRETE® HOLLOWCORE WALL PANELS



GIVEN:
Wind normal to panel = .9 k per connection
Uplift due to in-plane wind = 4.2 k per connection
Shear due to in-plane wind = 1.31 k per connection

PROBLEM:
Select a panel anchor for the base detail shown to resist the loads given.

SOLUTION:
For in-plane wind uplift, the factored force is V_u = 1.6(4.2) = 6.72 k
Applying the PCI recommended connection factor, V_u = 1.3(6.72) = 8.74 k
From the table on the other side, select a BWA-6, $\varnothing V_{n2}$ = 0.7(13) = 9.1 k
Check in-plane shear capacity for BWA-6: V_u = 1.6(1.31) = 2.10 k
Applying the recommended connection factor

V_u = 1.3(2.10) = 2.73 k [< $\varnothing V_n$ = 0.7(31.04) = 21.73 k OK]
Check normal wind on BWA-6: P_u = 1.6(.9) = 1.44 k
With the connection factor:
P_u = 1.3(1.44) = 1.87 k [< $\varnothing P_n$ = 0.7(5.34) = 3.74 k OK]

Conclusion:
Use BWA-6 for the panel weld anchor.
(Note that the BWA-6 anchor has sufficient capacity for the ACI 10 kip structural integrity connection, if required.)

Note: Sample calculations are intended to illustrate the concept presented and do not represent all considerations necessary for the complete design.

SPANCRETE
MANUFACTURERS ASSOCIATION

RESEARCH NOTES

SOUND TRANSMISSION CLASS (STC) RATING FOR SPANCRETE® HOLLOWCORE WALL PANELS

When Spancrete hollowcore wall panels are exposed to airborne sounds, vibrations are produced in the wall and radiate to the opposite side with reduced intensity. The airborne sound transmission loss of the wall is a function of its weight, stiffness and vibration damping characteristics. Weight is Spancrete's greatest asset when used as a sound insulator.

The Sound Transmission Class (STC) is a rating system used to estimate the sound insulation properties of a wall. For Spancrete insulated wall panels, adding incremental STC adjustments for add-on layers to the STC of the base wall panel can approximate the composite STC of the overall assembly.

Base Wall Panel	STC
6" Spancrete	50
8" Spancrete	56

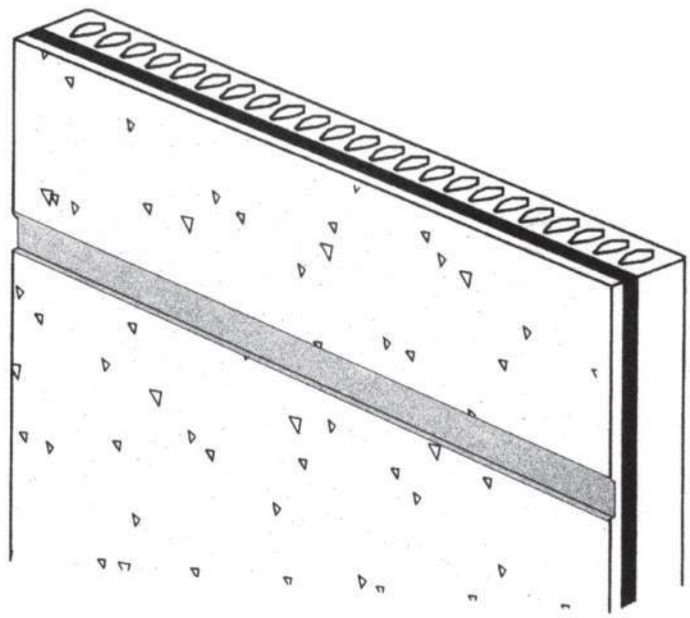
Add-On Layer	STC Increment
2-3" Insulation	Negligible
2" Exterior Wythe*	5
3" Exterior Wythe*	7

*Connected only by metal ties approximately 4' o/c

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE



The approximate STC for an 8" Spancrete® hollowcore wall panel with 2" insulation and a 2" exterior wythe:

STC = 56 + 0 + 5 = 61

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

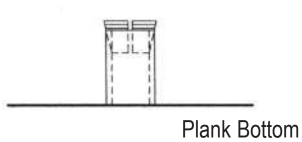
Armenia	*Ireland*
China	*Japan*
Denmark	*Russia*
Guatemala	*South Korea*
Hungary	*Switzerland*

SPANCRETE MANUFACTURERS ASSOCIATION

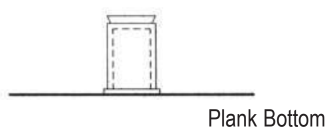
RESEARCH NOTES

BOTTOM INSERTS - POWERS FASTENERS

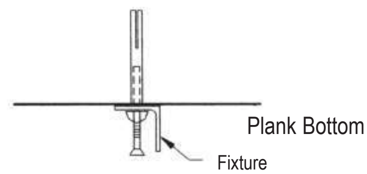
As part of a program for evaluating inserts used in Spancrete® hollowcore plank, five types of inserts manufactured by Powers Fasteners, Inc. were installed in the test plank bottom, directly under cores where the bottom thickness is 1-1/8".



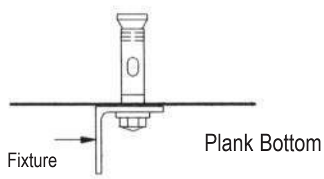
3/8" Hollow Set™ Drop—In
CAT NO. 9340



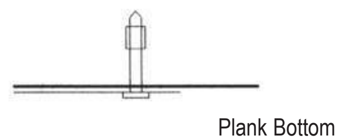
3/8" Mini Drop—In
CAT NO. 6322



Zamac Hammer Screw
1/4" x 1-1/4"
CAT NO. 2842



Acorn Nut Lok/Bolt™
1/4" x 1-3/8"
CAT NO. 5150



Powder Actuated
Threaded Stud
3/8" - 16
CAT NO. 50340

CONCLUSIONS:

1. Drilled sleeve type inserts are an economical and acceptable way of fastening to a Spancrete plank.

2. It is critical that the type of anchor does not extend into the core, that the embedment be at least ¾", and that the manufacturer's specified installation procedures be followed.

3. Holding power is reduced when insert is located near any plank edge and in thin concrete sections.

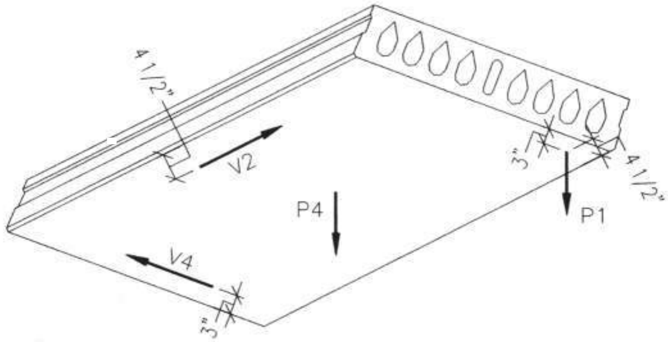
4. If the concrete spalls into the core during drilling, the insert should not be used at that location.

5. Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

Test results are given on the reverse side.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE



BOTTOM INSERT
TEST SETUP

CAT NO.	PRODUCT DESCRIPTION	DRILL BIT DIA. (in.)	EMBED DEPTH (in.)	ULTIMATE TEST LOADS (lbs)	P1	P4	V2	V4
9340	3/8" Hollow Set Drop-In	5/8"	3/4"	AVG.	1967	2523	3772	2723
				LOW	1470	2020	2530	2080
6322	3/8" Mini Drop-In	1/2"	3/4"	AVG.	2284	2346	2372	2088
				LOW	1590	1910	1790	760
2842	Zamac Hammer Screw 1/4" x 1 1/4"	1/4"	3/4"	AVG.	754	875	925	981
				LOW	670	670	710	600
5150	Acorn Nut Lok/Bolt 1/4" x 1-3/8"	1/4"	3/4"	AVG.	1338	879	1252	885
				LOW	1040	710	730	620
50340	3/8"—16 Powder Actuated Threaded Stud	-	3/4"	AVG.	N/A	1129	1028	942
				LOW	N/A	970	790	700

NOTE:

1. Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values. For more information on anchor design and installation, please refer to either the latest edition of Power's Fastening Systems Design Manual or www.powers.com.

2. All tests were performed at Power Fastener's Technical Center in New Rochelle, NY: and in accordance with ASTM E—488, Standard Test Methods For Strength of Anchors in Concrete and Masonry Materials. Five tests were done for each load case.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia, *China*, *Denmark*, *Guatemala*, *Hungary*, *Ireland*, *Japan*, *Russia*, *South Korea*, *Switzerland*

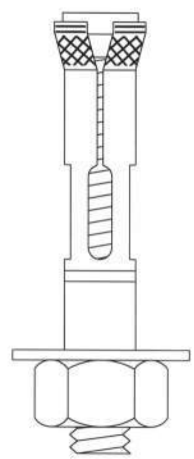
Spancrete® hollowcore is a registered trademark

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

BOTTOM INSERTS - REDHEAD

As part of a program for evaluating various inserts used in Spancrete® hollowcore plank, a common brand of ½" diameter sleeve anchors were installed in the test plank bottom, directly under cores where the bottom thickness is 1-½ inches.



½" ø x 3" REDHEAD
CAT. NO. HN 1230

SLEEVE ANCHORS TESTED

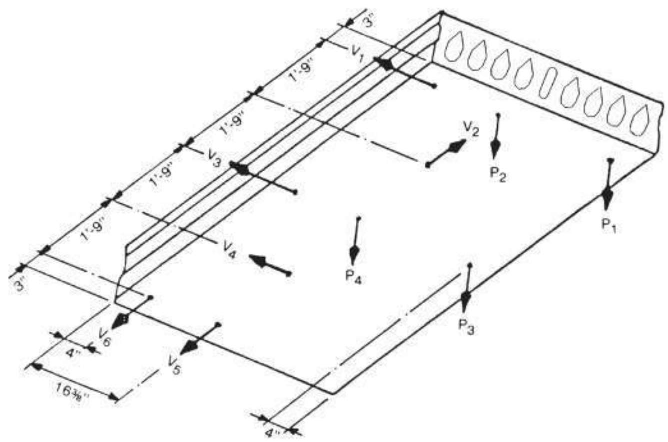
CONCLUSIONS:

1. Drilled sleeve type inserts are an economical and acceptable way of fastening to a Spancrete plank.
2. It is critical that this type of anchor does not extend into the core, that the embedment be at least 1-½", and that the manufacturer's specified torques be used.
3. Holding power is reduced when insert is located near any plank edge and in thin concrete sections.
4. Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE



BOTTOM EXPANSION BOLT TEST SETUP

½" ø x 3" REDHEAD ANCHOR

Load	Test #1	Mode*	Test #2	Mode*	Test #3	Mode	Avg.	Low
P1	5.053k	Conc.	4.012k	Conc.	5.480k	Conc.	4.848k	4.012k
P2	5.505	Steel	5.061	Conc.	5.133	Conc.	5.233	5.061
P3	1.536	Conc.	6.337	Conc.	4.888	Conc.	4.254	1.536
P4	5.285	Steel	6.092	Steel	5.251	Conc.	5.543	5.251
V1	3.157	Conc.	-	-	2.776	Conc.	2.967	2.776
V2	7.141	Steel	6.502	Steel	3.667	Steel	5.770	3.667
V3	6.125	Steel	5.963	Steel	3.730	Conc.	5.273	3.730
V4	6.850	Steel	6.750	Steel	5.881	Steel	6.494	5.881
V5	5.819	Conc.	5.332	Conc.	4.302	Steel	5.151	4.302
V6	2.336	Conc.	4.046	Conc.	2.700	Conc.	3.027	2.336

*Failure mode

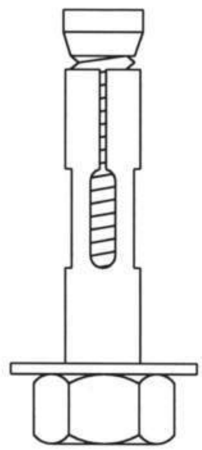
NOTE: Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

BOTTOM INSERTS - Hilti

As part of a program for evaluating various inserts used in Spancrete® hollowcore plank, a common brand of ½" diameter sleeve anchors were installed in the test plank bottom, directly under cores where the bottom thickness is 1-½ inches.



½" ø x 3" HILTI
CAT. NO. 336256

SLEEVE ANCHORS TESTED

CONCLUSIONS:

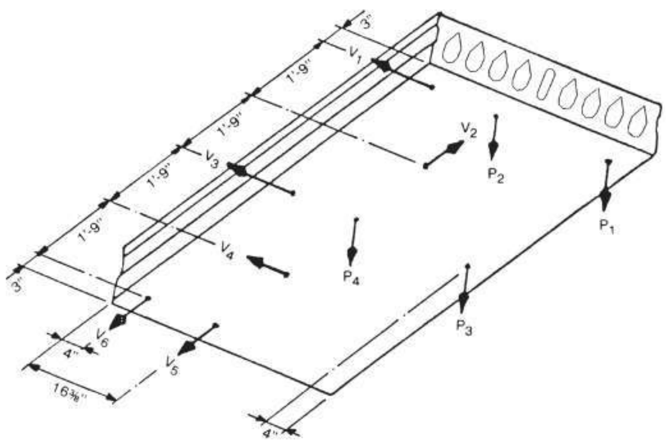
1. Drilled sleeve type inserts are an economical and acceptable way of fastening to a Spancrete plank.
2. It is critical that this type of anchor does not extend into the core, that the embedment be at least 1-½", and that the manufacturer's specified torques be used.
3. Holding power is reduced when insert is located near any plank edge and in thin concrete sections.
4. Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE
MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

BOTTOM EXPANSION BOLT TEST SETUP



½" ø 3" HILTI ANCHOR

Load	Test #1	Mode*	Test #2	Mode*	Avg.	Low
P1	4.23^k	Conc.	4.04^k	Conc.	4.14^k	4.04^k
P2	4.19	Conc.	4.73	Conc.	4.46	4.19
P3	4.73	Conc.	5.53	Conc.	5.13	4.73
P4	-	-	4.63	Conc.	4.63	4.63
V1	2.92	Conc.	2.69	Conc.	2.81	2.69
V2	4.43	Steel	4.40	Steel	4.42	4.40
V3	4.39	Steel	4.47	Steel	4.43	4.39
V4	3.08	Conc.	4.51	Steel	3.80	3.08
V5	5.65	Conc.	3.96	Conc.	4.81	3.96
V6	2.25	Conc.	2.17	Conc.	2.21	2.17

*Failure mode

NOTE: Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Seismic Design for Spancrete® Hollowcore Wall Panels

Special Base Connectors have been developed for Spancrete Wall Panels used as shear walls in moderate to high seismic design category structures. These Special Base Connectors provide a load limiting, ductile connection that protects the wall panels from overload in the event of an earthquake.

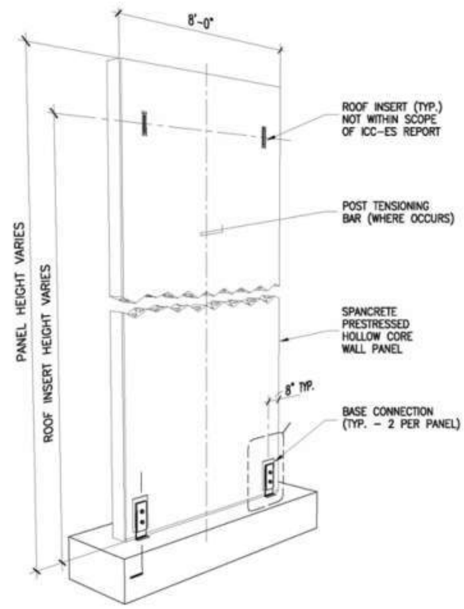


Figure 1 Wall Panel Perspective

The components of the system are shown in Figure 1.

The wall panel is a standard 8'-0" wide Spancrete Panel, with slight modifications to the core size in the region of the inserts.

When a large earthquake strikes, the wall panel rocks about the base, providing moment resistance through a tension/compression couple consisting of a tension force in the Special Base Connector and a compression force generated as the panel bears on the grout between the panel and the foundation. The Special Base Connector has a ductile load limiting mechanism that keeps the base moment capacity from exceeding a predetermined amount. This limits the amount of shear force that can be generated in the panel while the Special Base Connector allows the panel to continue to rock to larger displacements without losing strength.

The Special Base Connection components are shown in Figure 2.

This entire assembly consists of three parts. Part 1 is an assembly that is embedded in the Spancrete Wall Panel as it is fabricated in the precast plant. Part 2 is an assembly that is shipped loose and installed in the field. Part 3 is a traditional embedded steel foundation plate and is embedded in the foundation concrete.

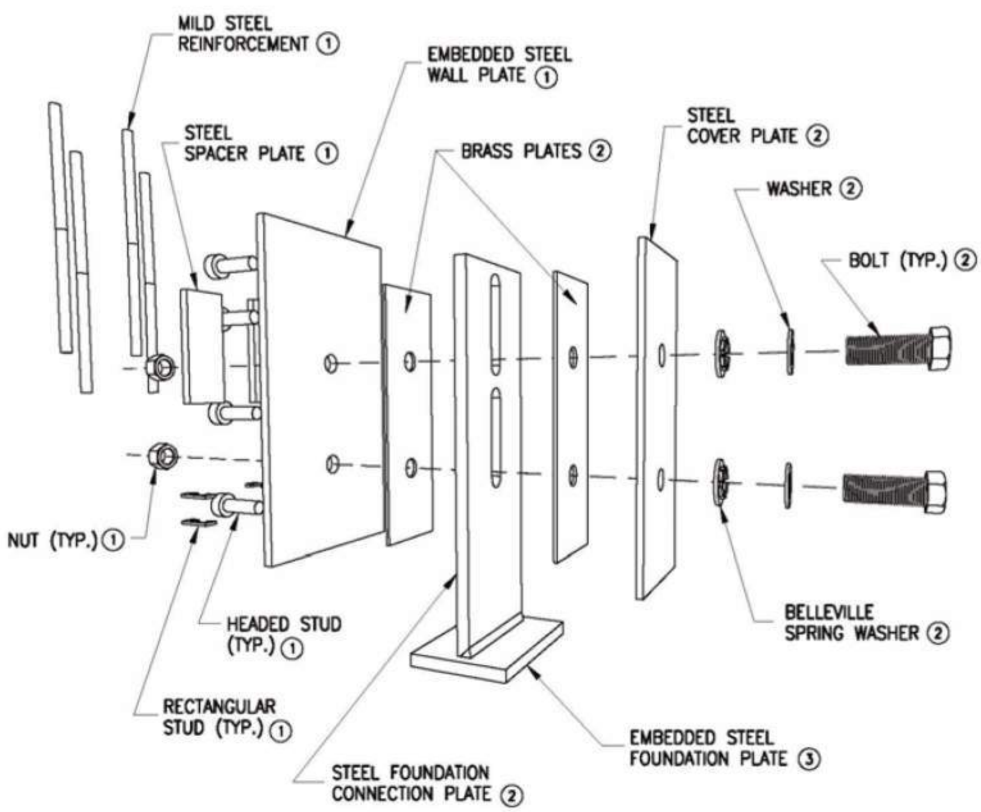
In an earthquake, the Spancrete Wall System with Special Base Connector responds elastically until the capacity of the Special Base Connector is reached. Once this happens, the panel continues to rock and the "sandwich" consisting of the embedded steel plate, the two brass plates and the steel cover plate slides back and forth across the steel foundation connection plate which is rigidly attached to the foundation. The sliding of the brass plates dissipates energy, the amount of which is controlled by a constant clamping force provided by the Belleville Spring Washer.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

The Spancrete® Hollowcore Wall system with Special Base Connector has an ICC Evaluation Report which allows it to be used in regions of high seismic risk. ICC Evaluation Report 5902 considers the system to be equivalent to a special reinforced concrete shear wall as defined in both the UBC and IBC. Available references that can be used to assist in the design and construction of the Spancrete Wall System with Special Base Connector are listed below.



① INDICATES WALL BASE EMBED COMPONENT – EMBEDDED IN PRECAST CONCRETE WALL PANEL.

② INDICATES SBC/FRICTION ASSEMBLY COMPONENT – SHIPPED LOOSE and INSTALLED IN FIELD.

③ INDICATES FOUNDATION EMBED PLATE – INSTALLED IN FIELD.

Figure 2 – Base Connection Component Diagram

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Design Flowchart

Step 1
Develop Design Seismic Forces

Design forces are developed in accordance with the governing building code for the project site. Since the system is considered a Special Reinforced Concrete Shear Wall, the R value for 2003 IBC is 5 for a bearing wall and 6 for a non-bearing wall. Distribute the design shear forces to each wall panel as appropriate.

Step 2
Develop Design Wind Loads

Depending on the building site, wind loads may govern over seismic forces. Regardless of which force is larger, different components of the panel design may be governed by wind or seismic. Specifically, out-of-plane design capacities of the base connections are very different depending on whether seismic or wind forces are being considered.

Step 3
Calculate Wall Panel Interaction Diagram

For a given Spancrete® Hollowcore Wall Panel Section, a unique interaction diagram is developed depending on the design concrete strength and the number of strands. Note that the core size shown at the embed locations must not be made any larger. The interaction diagram also depends on the tension capacity of the Special Base Connector, which is given in the ICC Evaluation Report. Detailed calculations for developing an interaction diagram for this system are given in Reference 2.

Step 4
Determine the axial load on the wall

The axial load on a given wall panel can vary significantly, depending on whether the wall is a bearing wall or not, and what kind of roof system it is supporting. Include the weight of the entire wall panel, as the section that is being checked is at the base of the wall.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE
MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Step 5 Determine the moment strength of each wall panel	The moment strength at the base of each wall panel is determined using the interaction diagram from Step 3 and the axial load from Step 4. See Reference 4 for detailed calculations.
Step 6 Calculate the shear capacity of each wall panel	The shear capacity of each wall panel is equal to the moment capacity at the base divided by the height to the roof (point of load application.)
Step 7 Compare the shear capacity of each wall panel to the demand on the wall panel	If the shear capacity of the wall panels exceed the design demand on the wall, then the design is acceptable. If not, vertical post tensioning can be added to the wall panels to increase their axial load. If this is done, go back to Step 4 and revise the total axial load to include the PT force. See Reference 4 for detailed calculations.
Step 8 Check deformation capacity of the wall panels	Code level deflections must be checked and compared to the allowable drift. Also, the system must be able to deform through the Maximum Inelastic Response Displacement Demands without reaching the deflection limits of the Special Base Connector.
Step 9 Check out-of-plane connections	Out-of-plane loads must be checked against the out-of-plane capacity of the Special Base Connectors. See Reference 3 for out-of-plane connector capacities for both wind and seismic forces.

Flexural Design and Deformation Check

Figure 3 shows the elevation of the panel that will be designed for overturning resistance. Figure 4 shows the panel cross section. The roof is 28'-0" above the top of the footing and there is no parapet. The Spancrete® Hollowcore Wall Panel is 8'-0" wide. Special Base Connectors resist the overturning forces caused by in-plane earthquake (or wind) loading and are located 8" in from the panel edge.

The design seismic force (applied at the roof elevation) is 6.2 kips and is determined from the governing building code. In this case, the wall panel resists no nominal gravity loads beyond the weight of the panel itself. The panel weight is 21.2 kips.

The Spancrete Wall Panel is 8" thick, with a effective thickness of 6.12". The design concrete strength is 5,000 psi, and there are (20) 3/8" diameter strands that prestress the wall panel. There are no other panel connections besides the two Special Base connectors and the Roof Inserts.

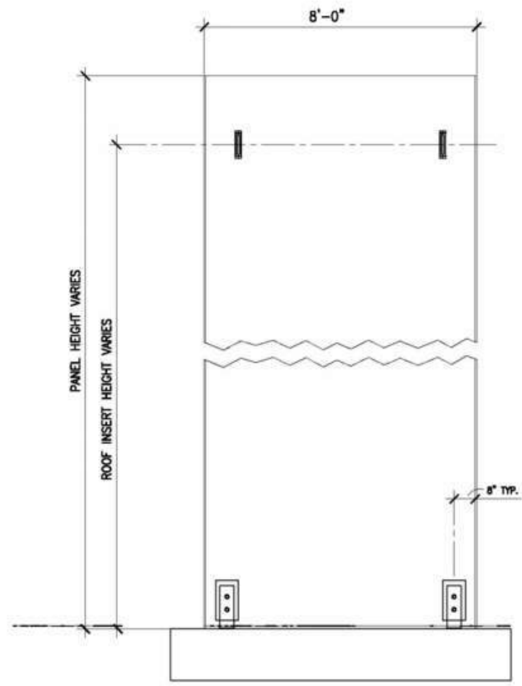


Figure 3 Wall Panel Elevation

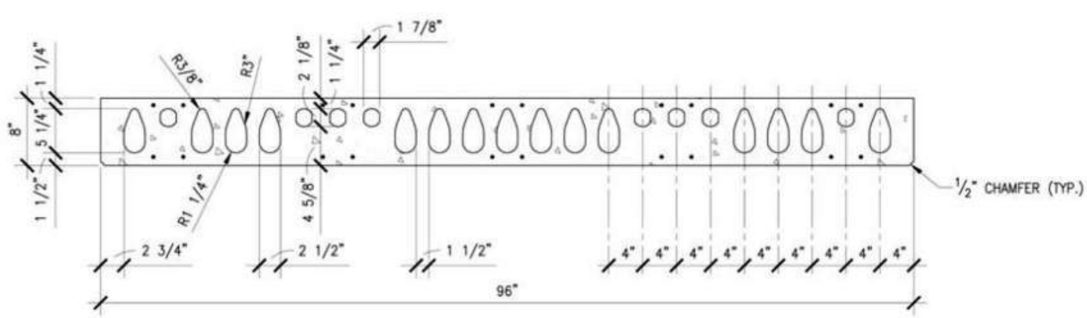


Figure 4 Wall Panel Section

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

Axial Load/Moment Design

The design gravity load on the panel is P_u = 0.9 x 21.2 kips = 19.1 kips

The overturning moment on the panel is M_U = 6.2 kips x 28 ft = 174k – ft

Using the interaction diagram shown in Figure 5, the moment capacity of this Spancrete® Hollowcore Wall Panel at an axial load of 19.1 kips is 175 k-ft. Therefore, it provides sufficient moment strength to resist the applied loads.

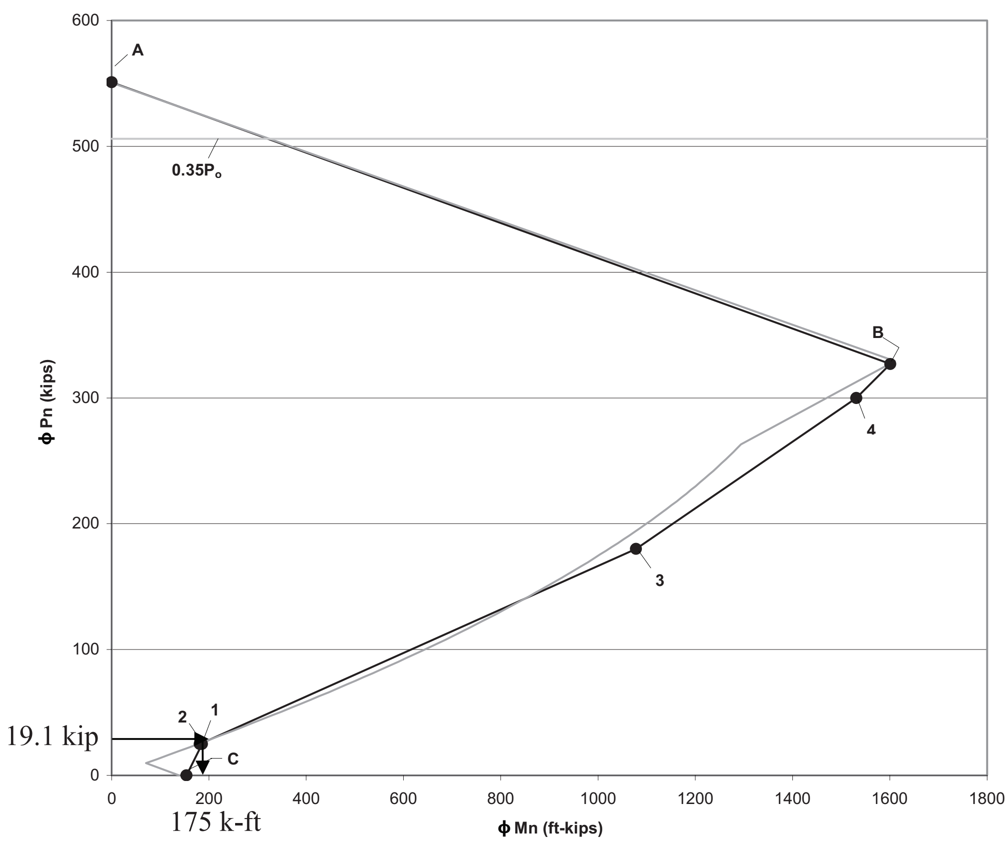


Figure 5 Interaction Diagram

f'_c = 5000 psi

$t_{effective}$ = 6.12 in

(20) 3/8" ϕ strands (f_{pu} = 250 ksi)

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Deformation Check

In addition to the panel design for forces, system deformations must be checked for both code level loads and Maximum Inelastic Response Displacement Demand.

The deformation consists of three basic parts. First is the deformation of the panel itself which is calculated using traditional stiffness relationships, assuming a fixed base. Second is the panel deformation that is caused by the elongation of the Special Base Connector. This component models the flexibility of the base between the panel and the foundation. Third is the post-elastic component of the panel deformation that is accommodated by the slip of the base connection. This allows the panel to undergo post-elastic deformation without degradation in the panel or connections. Foundation rocking is not considered.

Since the panel is prestressed, it does not crack under the design forces and solid section properties can be used to calculate the panel stiffness. For shear, only the solid face shell of the panel is used to define the shear area.

A_v = (1.25" + 1.5") x 8'-0" x 12 in/ft
= 264 in^2

For flexure, the effective thickness is used for the flexural moment of inertia.

I = 6.12 in x (8'-0")^3/12 x 1728 in^3/ft^3= 451,215 in^4

The flexural deflection is $$\Delta_f = \frac{V_u h_w^3}{3EI} = \frac{6.2\text{k x } (28\text{ft})^3 \text{ x } 1728\text{in}^3/\text{ft}^3}{3 \text{ x} 4031\text{ksi x } 451{,}215\text{in}^4} = 0.0431\text{in}$$

The shear deflection is $$\Delta_v = \frac{1.2V_u h_w}{0.4EA_v} = \frac{1.2 \times 6.2\text{k} \times 28\text{ft} \times 12\text{in/ft}}{0.4 \times 4031\text{ksi} \times 264\text{in}^2} = 0.0059 \text{ in}$$

At the moment demand on the interaction diagram, the force in the foundation plate is equal to 33.3 kips. Therefore, the concentrated deflection at the roof, Δ_{pl}, due to the foundation plate elongation, δ_{pl}, is

T_{pl} = 33.3k $$\delta_{pl} = \frac{T_{pl} L_{pl}}{A_{pl} E_s} = \frac{33.3\text{k x 9in}}{\frac{1}{2}\text{in x 5in x 29,000ksi}} = 0.0041\text{in}$$

$$\Delta_{pl} = \delta_{pl} \frac{L_{panel}}{d} \frac{h}{L_{panel}} = 0.0041 \frac{96\text{in}}{(96-8)\text{ in}} \frac{28\text{ft}}{8\text{ft}} = 0.016\text{ in}$$

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

DESIGN EXAMPLE

The total deflection is

$$\delta_{le} = \Delta_f + \Delta_v + \Delta_{pl} = 0\ .0431 \text{ in} + 0.0059 \text{ in} + 0.016 \text{ in} = 0.065 \text{ in}$$

The Maximum Inelastic Response Displacement Demand is then

$$\delta_1 = \frac{C_d \delta_{le}}{I_E} = \frac{C_d(\Delta_f + \Delta_v + \Delta_{pl})}{I_E} = 5\frac{(0.065\,\text{in})}{1.0} = 0.325\,\text{in}$$

For a one story building, the first story drift is then

$$\Delta = \frac{\delta_1}{h_w} = \frac{0.325\text{in}}{28\,\text{ft}}\frac{\text{ft}}{12\,\text{in}} = 0.0009$$

This is less than the allowable story drift, Δ_a, of 0.01 for the Spancrete® Hollowcore Wall Panel System [Ref. Spancrete Machinery Corp., (2004)].

All of the additional post elastic deflection is accommodated by the slip of the friction connection.

The additional post elastic roof deflection is

$$\Delta_{slip} = \delta - \delta_{le} = 0.325 \text{ in} - 0.065 \text{ in} = 0.26 \text{ in}$$

Since the Special Base Connector is located 8" from each end of the panel, when calculating the slip on the bolt, it is conservative to estimate d = L_{panel} – 8". The roof deflection requires slip at the base equal to

$$\delta_{slip} = \Delta_{slip}\frac{L_{panel}}{h}\frac{d}{L_{panel}}$$

$$= 0.26\,\text{in}\frac{8\,\text{ft}}{28\,\text{ft}}\frac{(96-8)\text{in}}{96\,\text{in}} = 0.068\,\text{in} < 1\tfrac{5}{8}\text{ in}$$

This slip deformation is less than the available slip in the vertical slot. Therefore, the friction connection has sufficient post elastic deformation capacity to accommodate the required story drifts.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Shear Design of the Spancrete® Wall Panel

Using the force and geometry criterion previously defined, the shear design can be done. Since the Spancrete Hollowcore Wall Panels do not have any horizontal shear reinforcing, in order to meet the requirements for a special reinforced concrete wall, the design shear stress must be kept less than $1\sqrt{f'_c}$.

Using just the face shell, the shear area of the panel is

$$A_v = (1.25" + 1.5") \times 8'\text{-}0" \times 12 \text{ in/ft} = 264 \text{ in}^2$$

The resulting shear stress on the wall panel is

$$v_u = \frac{6.2\text{k} \times 1000 \text{ lb/k}}{264 \text{ in}^2} = 23.5 \text{ psi} < 1\sqrt{5000 \text{ psi}} = 70.7 \text{ psi}$$

Shear Design at the Panel Base

At the panel base, the shear force in the panel must be transferred to the foundation. There are two loading conditions that need to be considered.

First, the design seismic force is applied to the panel, and resistance is provided by shear friction at the foundation interface. The normal force that creates the shear friction is provided by the compression caused by the overturning couple. Note that the effect of any applied gravity load or post tensioning is already included in the increased moment capacity.

Second, the maximum shear force that can be generated in the panel is limited by the flexural strength at the base connection. Sliding shear at the foundation interface is again resisted by compression caused by the overturning couple. Since the shear force is proportional to the compression caused by the overturning couple, one calculation suffices for both situations.

From the interaction diagram shown in Figure 5, the moment capacity, ϕM_n, of the panel is 175 k-ft. In the tension controlled region of the interaction diagram, the ϕ factor is 0.9. Since the flexural strength of the panel is controlled by the Special Base Connector, which has extremely flat hysteresis behavior (see Figure 6), the flexural strength at the base can be approximated by

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

$$M_p = \frac{\phi M_n}{\phi} = \frac{175\,k-ft}{0.9} = 194\,k-ft$$

The shear force associated with M_p is

$$V_p = \frac{M_p}{h} = \frac{194\,k-ft}{28\,ft} = 6.9\ kips$$

The normal force acting at the compression region of the Spancrete® Hollowcore Wall Panel is

$$N = \frac{M_p}{d}$$

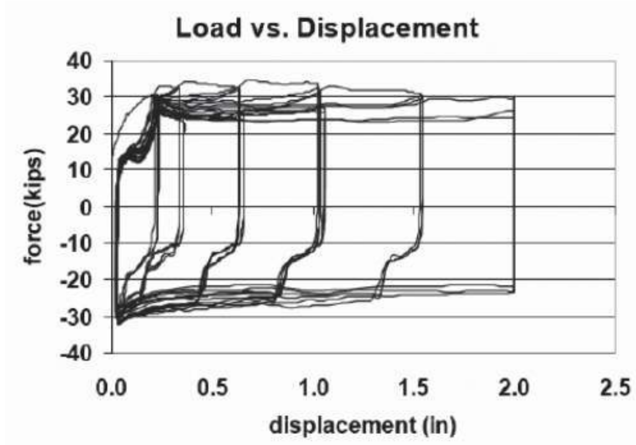


Figure 6 – Hysteresis Loop for Special Base Connector

where

d = distance between the Special Base Connector acting in tension and the center of the compression force.

Since the Special Base Connector is located 8" from the end of the panel for purposes of calculating the normal force on the panel, the distance, d, can be conservatively approximated by

$$d = L_{panel} - 8"$$

where

L_{panel} = Wall Panel Length (=8'-0")

Therefore,

$$N = \frac{M_p}{d} = \frac{194\,k-ft}{(8\,ft - 8\,in/12)} = 26.5\,kips$$

Since the base of the Spancrete Wall Panel is sawcut, the friction coefficient, μ is 0.6. Therefore, the shear friction resistance provided by this normal force is

$$\phi V_n = \phi \mu N = 0.75 \times 0.6 \times 26.5\ kips = 11.9\ kips > V_p = 6.9 kips$$

which is larger than the maximum shear force the panel can generate. Since the shear strength of the interface is greater than the shear associated with the formation of a plastic hinge, then ϕ = 0.75 is used [Ref. 6].

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Vertical Post-Tensioning

In some cases, the overturning capacity that can be achieved by the Spancrete® Hollowcore Wall Panel with Special Base Connectors is insufficient to resist the design seismic forces on the wall. In these cases, it is possible to increase the flexural capacity of the wall by adding vertical post-tensioning in the field. The post-tensioning effectively adds reliable vertical load to the panel, moving the design point up on the interaction diagram.

In the design example [Ref. 4], example calculations are provided for a Spancrete Wall Panel building located in San Francisco. In this case, additional reliable vertical load needs to be added to the panel to increase the flexural strength of the panel to foundation connection.

In this case, the design seismic force (applied at the roof elevation) is 17.6 kips per panel. However, in addition to the wall weight itself, vertical post-tensioning is provided to increase the vertical load on the panel, moving the design point up the interaction diagram. (See Figure 7.)

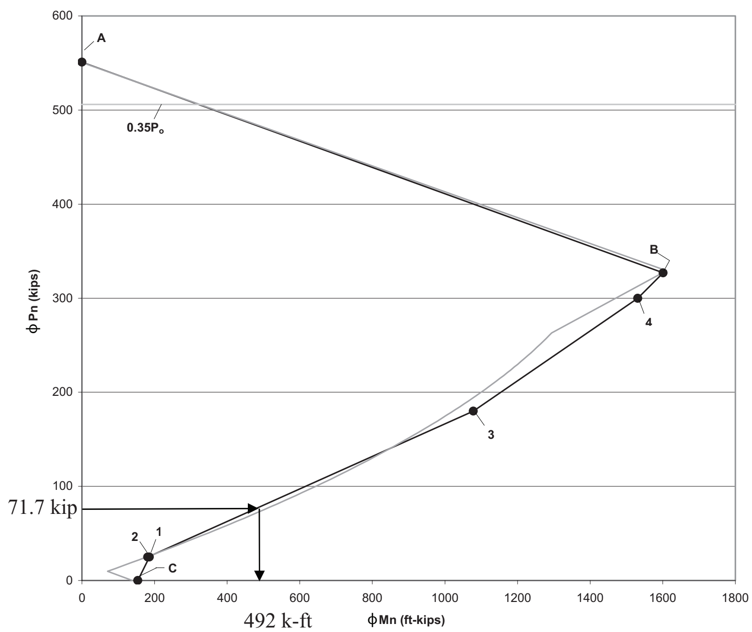


Figure 7 Interaction Diagram

f'_c = 5000 psi

$t_{effective}$ = 6.12 in

(20) 3/8" ϕ strands (f_u = 250 ksi)

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

The vertical post-tensioning is anchored in the foundation and coupled at the base of the panel. Since the post-tensioning is unbonded for the full panel height, the increase in PT stress as the panel rocks is small, but is calculated below.

Axial Load/Moment Design

The panel weight is 21.2 kips. Two (2) 5/8" ϕ Dywidag Threadbars will be stressed to a force of 0.6 F_{pu}, after losses. With this post-tension force,

$$P_u = 0.9\,P_{DL} + A_{ps}f_{fpt,i}$$
$$= 0.9 \times 21.2 \text{ kips} + 2 \times 0.274 \text{ in}^2 \times 0.6 \times 160 \text{ ksi} = 71.7 \text{ kips}$$

The overturning moment on the panel is

$$M_u = 17.6 \text{ kips} \times 28 \text{ ft} = 492\text{k} - \text{ft}$$

Using the interaction diagram shown in Figure 7, the moment capacity of this Spancrete® Hollowcore Wall Panel at an axial load of 71.7 kips is 492 k-ft.

Deformation Check

The deformation check and shear stress check should be performed similar to the calculations shown earlier.

Using those calculations, the Maximum Inelastic Response Displacement for this wall is

$$\delta_1 = 0.78"$$

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Post-Tensioning Stress Check

In order to ensure that the PT remains elastic through the required deformations, the strain in the PT bars is checked against their strain capacity. In the design guide [Ref. 2], the limit state that is checked is that corresponding to the displacement capacity of the wall system, Δ_{cap}. However, since this wall is so stiff, the Maximum Inelastic Response Displacement is much less than the displacement associated with the maximum available slip in the bolt. Therefore, the maximum PT stress will be checked for 50% more than the Maximum Inelastic Response Displacement.

The horizontal roof drift capacity is

$$\Delta = 1.5\,\delta_1 = 1.5 \times 0.78" = 1.17"$$

The additional strain the PT at this horizontal displacement is $\varepsilon_{pt,\Delta}$ where

$$\delta_{corner} = \Delta\left(\frac{L_w}{h_w}\right) = 1.17"\left(\frac{8ft}{28ft}\right) = 0.33"$$

$$\delta_{pt} = \frac{\delta_{corner}}{2} = \frac{0.33"}{2} = 0.17"$$ (with PT located at mid-length of wall)

$$\varepsilon_{pt,\Delta} = \left(\frac{\delta_{pt}}{h_w}\right) = \left(\frac{0.17"}{28ft \times 12in/ft}\right) = 0.00051$$

The additional stress in the PT at this horizontal displacement is

$$f_{pt,\Delta} = E_{pt}\varepsilon_{pt,\Delta} = 29{,}000 \text{ ksi} \times 0.00051 = 14.7 \text{ ksi}$$

and

$$f_{ptfinal} = f_{pt,i} + f_{pt,\Delta} = 0.6 \times 160 \text{ ksi} + 14.7 \text{ ksi} = 110.7 \text{ ksi} < f_{pt,y} = 0.9 \times 160 \text{ ksi} = 144 \text{ ksi}$$

Therefore, the PT will remain elastic even at 50% more than the Maximum Inelastic Response Displacement.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

Connection Design

Note that at the time the ICC Evaluation Report was obtained, the governing code was the IBC 2000 for purposes of that approval. Therefore, the following calculations are developed using IBC 2000, and can be compared with the published connection capacities in the Reference 3. However, similar calculations can be followed, modifying as appropriate, to determine the out-of-plane capacity of the Special Base Connector for other model codes.

In addition to considering in-plane loads, a Spancrete® Hollowcore Wall Panel with Special Base Connectors must also resist out of plane loads.

For a typical example in Boston [Ref. 4], the out-of-plane ultimate design force is as follows:

Loading Type	Design Force (kips)
Seismic	0.29
Wind	1.6 x 1.38 = 2.20

Table 1: Design Out-of-Plane Forces – Boston

Out of plane seismic loading conditions consider both loads on the body of the connector (a_p=1.0 and R_p=2.5) as well as loads on the fasteners (a_p=1.25 and R_p=1.0). Using these values, at the base connection the fasteners must be designed for a force 3.1 times the load on the body of the connector. In the following development, the connection capacity (as limited by the strength of the fasteners) is reduced by 3.1 to provide a "fastener based" strength of the connection that can be directly compared with the above design forces.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Out-of-Plane Capacity

The foundation plate is clamped to the wall panel by the cover plate and the two bolts that connect the cover plate to the embed plate, as shown in Figure 8. The foundation plate is welded to the foundation embed plate, providing fixity to the bottom of the plate.

Figure 9 shows the reactions on the ½" x 5" foundation plate when the panel moves out of plane, assuming that no additional fixity is provided by the cover plate. This is conservative for the design of the foundation plate and the foundation embed plate.

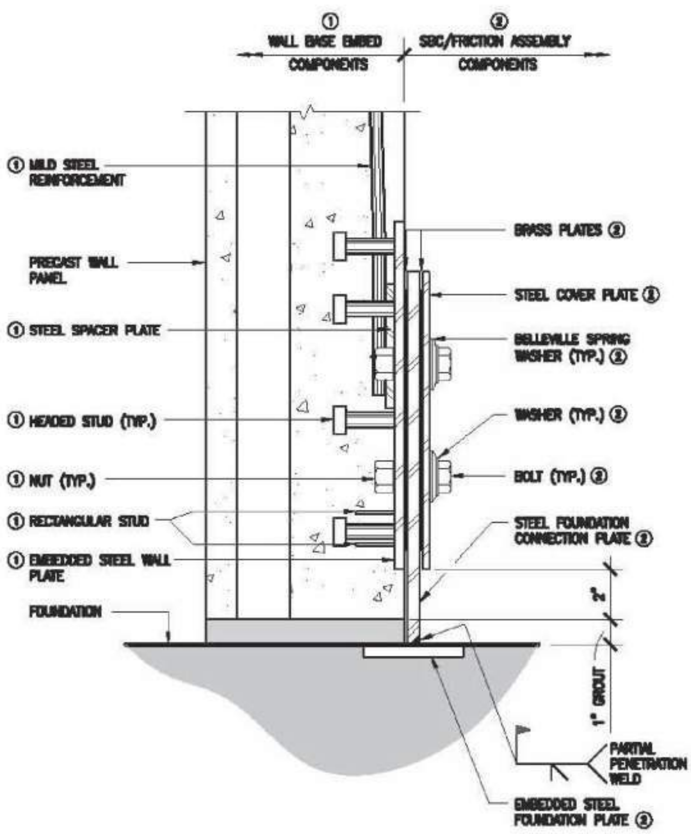


Figure 8 Base Connection

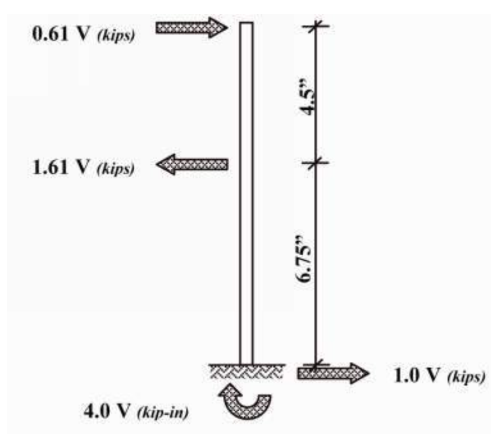


Figure 9 Reactions on Foundation Plate

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

Foundation Plate Design

Based on a finite element analysis of the foundation plate, the maximum moment on the foundation plate is 4.0V at the connection to the foundation embed plate. The plate section is ½" x 5", but is reduced by the bolt hole at the bolt locations. The effective plate width at these locations is 4.19".

Using this effective plate width, the foundation plate plastic modulus is

$$Z_{PL} = (5\text{-}13/16)\text{in} \times \frac{(0.5\text{in})^2}{4} = 0.26\text{in}^3$$

For 36 ksi material, the out-of-plane flexural strength of the plate is

$$\phi M_n = 0.9 \times 36 \text{ ksi} \times 0.26 \text{ in}^3 = 8.48\text{k-in}$$

From Figure 9, M_u(k-in)=4V_u(kips), therefore the out-of-plane shear capacity of the connection is

$$\phi V_n = \frac{\phi M_n}{4\text{in}} = \frac{8.48\text{k}-\text{in}}{4\text{in}} = 2.12\text{kips}$$

For 50 ksi material, the out-of-plane shear capacity of the connection is 2.94 kips.

Stud Group Design

Assuming that (2) studs resist the out-of-plane bottom bolt reaction,

$$P_{stud} = \frac{1.61V}{2} = 0.805V$$

With ½" diameter studs (f_y=60 ksi)

$$N_s = n\, A_{se}\, f_y = 1 \times 0.2\text{in}^2 \times 60 \text{ ksi} = 12 \text{ kips}$$

$$h_{ef} = 2\tfrac{1}{2} \text{ in} - \tfrac{1}{4} \text{ in} = 2.25"$$

$$N_b = k\sqrt{f'_c}\, h_{ef}^{1.5} = 24\sqrt{5000\,\text{psi}}\,(2.25\,\text{in})^{1.5} / 1000\,{}^{\text{lb}}\!/\!_{\text{k}} = 5.73\,\text{kips}$$

Therefore, N_n=5.73 kips.

Since $N_s > N_b$ ductile yielding of the connection must impose no more than 0.75 ϕN_n on the anchor. The force on a stud when the 36 ksi foundation plate yields is:

$$P_{stud}(\text{at PL yield}) = \frac{1.61}{2}\frac{(36\text{ksi x } 0.26\text{in}^3)}{4\text{in}} = 1.88\,\text{kips}$$

which is less than

$$0.75\phi N_n = 0.75 \times 0.75 \times 5.73 \text{ kips} = 3.22 \text{ kips}$$

For Grade 50 plate, the load on the stud at plate yield is 2.62 kips, which is still less than 3.22 kips.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Fastener Design

As discussed above, the load on the fasteners themselves must be amplified by a factor of 3.1, or alternately, the capacity reduced by a factor of 3.1. Therefore, the Special Base Connection capacity, as limited by the fasteners, is

$$\phi V_n = \frac{2}{1.61}\phi N_n = \frac{2}{1.61}0.75\left(\frac{5.73\text{kips}}{3.1}\right) = 1.72\text{kips}$$

This capacity governs for seismic design forces in Seismic Design Category C or above, For Seismic Design Categories A & B, as well as wind forces, the extra 3.1 factor does not apply, and the strength of the connector is governed by the flexural strength of the foundation plate.

Special Base Connection Strength

For designs governed by the IBC 2000, the following table identifies the out-of-plane capacity of the Special Base Connector:

	NOMINAL OUT-OF-PLANE CAPACITY, ϕV_n (kips)[1]	
	A36 Foundation Plate	**A572, Gr50 Foundation Plate**
Seismic	1.72	1.72
Wind[2]	2.12	2.94

[1] Out-of-plane capacity is the capacity of the connection at the top of the footing embed plate, with the geometry shown in Figure 8. The weld between the foundation plate and the footing embed plate, as well as the anchorage of the footing embed plate, must be designed by the panel designer.

[2] Includes Seismic Design Categories A & B.

Table 2: Special Base Connection Out-of-Plane Design Strength

For the sample design in Boston (see Table 1), the seismic capacity exceeds the seismic demand (1.72 kips > 0.29 kips) and the wind capacity exceeds the wind demand (2.94 kips>2.20kips) as long as Grade 50 foundation plate is used.

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE MANUFACTURERS ASSOCIATION

DESIGN EXAMPLE

References

1. Spancrete Machinery Corporation (2003), *Quality Control Manual - Spancrete® Wall Panels with Slotted Bolted Connections*, Waukesha, WI, July 21, 2003

2. Spancrete Machinery Corporation (2004), Design Procedure for the *Spancrete Prestressed Hollowcore Wall Panel*, Waukesha, WI, February 24, 2004.

3. International Code Council Evaluation Service (2004), *Spancrete Prestressed Hollowcore Wall Panels with Base Connections*, Whittier, CA, March 1, 2004.

4. Spancrete Manufacturers Association (2005), TECHNICAL MANUAL – *Section 4.4 Design Example for Spancrete Seismic Wall Panels with Special Base Connectors*, Waukesha, WI, January 4, 2005.

5. Bora, C., Oliva, M.G., Nakaki, S.D. and Becker, R. (2005), "Development of a Precast Concrete Shear-Wall System Requiring Special Code Acceptance", *PCI Journa*l, V. 52, No. 1, Jan-Feb, 2007, pp 122-135.

6. American Concrete Institute (2005), Building Code Requirements for Structural Concrete (ACI 318-05), Farmington Hills, MI, 2005.

Note: Research Notes provide a condensed version of a more complicated design procedure and do not represent all considerations necessary for a complete design. Consult the references for more information.

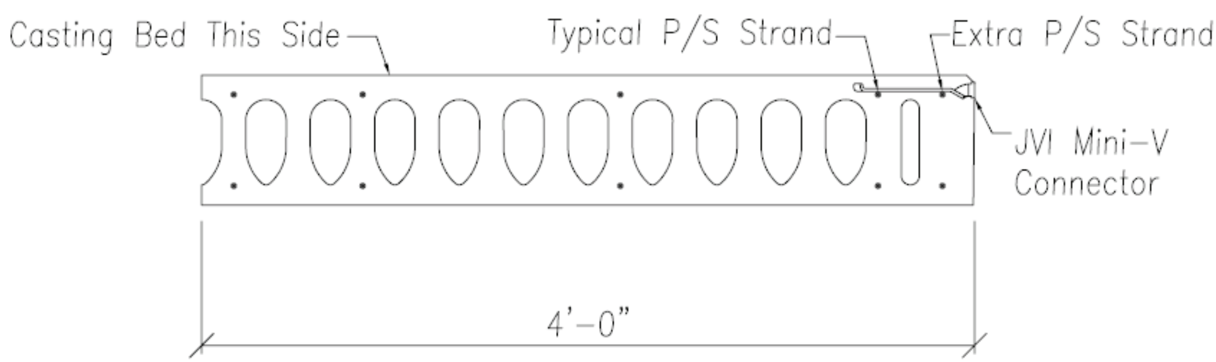
SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Combined Loading on JVI Mini-V Connectors Cast In Spancrete ® Hollowcore Wall Panels

When Spancrete Wall Panels are used as shear walls to resist lateral loads acting on a structure may be required that adjacent panels be connected together. For applications where the interior surfaces of the panels are exposed such as gymnasiums, it is desirable to conceal these connections in the wall panel joint. The JVI Mini-V connector is a readily available insert that will satisfy this condition.

The Spancrete Manufacturers' Association conducted tests of the JVI Mini-V connector to determine its capacity when cast in Spancrete Wall Panels. The inserts were cast in 4-foot wide test specimens (typical Spancrete Wall Panels are 8 foot wide), and tested in an inverted orientation as shown.



Test Specimen

Research Notes are produced periodically by the SMA Technical Committee. SMA Research Notes are based on testing done for the Spancrete Manufacturers Association. The information contained in these Research Notes should be used by those experienced in structural design and should not replace sound engineering judgment.

SPANCRETE
MANUFACTURERS ASSOCIATION

RECOMMENDATIONS

1. The values tabulated are raw test values to which no factor of safety has been applied.
2. The cracking load should be used as the design capacity. Due to the tight clustering of the data, using the average load for the unidirectional tests is acceptable.
3. A strength reduction factor ø equal to 0.65 should be applied to the test value.
4. The calculated design forces must be multiplied by standard ACI and IBC load factors to generate ultimate loads for comparison.
5. If these connectors are used to support gravity loads (e.g. supporting a panel above a door opening), an additional overload factor is recommended.
6. When the connectors are used to join multiple panels for a shear wall (VQ/I), the overload factor is not required.

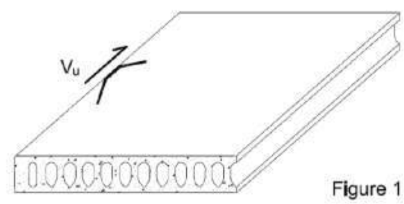


Figure 1

Table 1	Load Applied Parallel to the Panel Face				
Test Number	Cracking		Maximum		Failure Mode
	Load (lbs)	Displacement (in)	Load (lbs)	Displacement (in)	
H-1	11800	0.027	12000	0.692	yield tension leg buckle compression leg
H-2	10700	0.016	11450	0.886	yield tension leg buckle compression leg
H-3	11700	0.013	11700	0.013	fracture tension leg buckle compression leg
H-4	12300	0.072	12300	0.072	fracture tension leg buckle compression leg
Average	**11625**		**11862**		
Low	**10700**		**11450**		

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia, China, Denmark, Guatemala, Hungary, Ireland, Japan, Russia, South Korea, Switzerland

Spancrete® hollowcore is a registered trademark

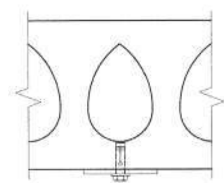
SPANCRETE MANUFACTURERS ASSOCIATION MEMBERS

SPANCRETE MANUFACTURERS ASSOCIATION

RESEARCH NOTES

Investigation of Inserts in Spancrete ® Hollowcore Plank – Bottom Expansion Bolts

As part of a program for testing inserts used in Spancrete® plank, a common brand of ½ inch diameter expansion anchors were installed in the test plank bottom, directly under the cores. The bottom thickness was either 1-1/8 inches or 1-1/2 inches depending on the Spancrete section used.

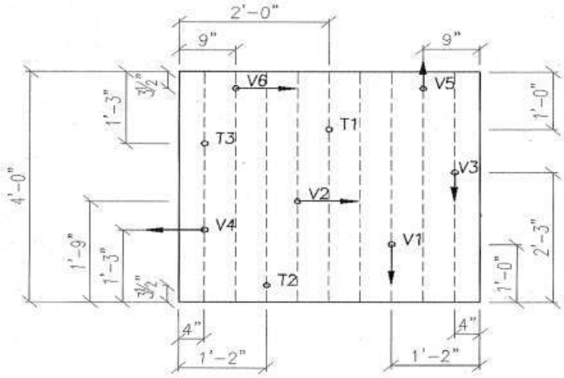


½" x 2-3/4" Power-Bolt Heavy-duty Expansion Anchor
CAT. NO. 6930

CONCLUSIONS:

1. Drilled expansion anchors are an economical and acceptable way of fastening to a Spancrete plank.
2. It is critical that the anchor does not extend into the core, that the embedment (1-1/8" or 1-1/2") agree with the proper load capacity chart, and that the manufacturer's specified installation procedures be followed.
3. Holding power is reduced when the insert is located near any plank edge and in thin concrete sections.
4. If the concrete spalls or cracks during the drilling and torqueing of the anchors, the insert should not be used at that location.
5. Anchors with 1-1/8" embedment were not able to be tested in shear.
6. Test values must be reduced by appropriate strength reduction factors and safety factors to obtain working load values.

SPANCRETE MANUFACTURERS ASSOCIATION



(dashed lines represent core centerlines)

TEST RESULTS

TENSION

	1-1/2 " Bottom Flange			1-1/8" Bottom Flange		
	T1	T2	T3	T1	T2	T3
Average	4670	4090	4280	2560	2565	3035
Low	3787	3708	3491	1884	1832	1699

SHEAR

	1-1/2 " Bottom Flange					
	V1	V2	V3	V4	V5	V6
Average	7515	8030	6830	4945	6310	6275
Low	4581	6371	4811	3297	5558	5005

NOTE: Values given are test values in pounds. Appropriate strength reduction factors and safety factors must be applied.

MIDWEST
Hanson Structural Precast Midwest, Inc.
Maple Grove, Minnesota

Spancrete, Inc.
Green Bay, Wisconsin

Spancrete Industries, Inc.
Waukesha, Wisconsin

Spancrete of Illinois, Inc.
Arlington Heights, Illinois

Wells Concrete
Wells, Minnesota

WEST
Hanson Structural Precast Pacific, Inc.
Irwindale, California

KIE-CON
Division of Kiewit Pacific Co.
Anitoch, California

Owell Precast
Sandy, Utah

SOUTHWEST
Manco Structures, Ltd.
Schertz, Texas

SOUTH
Cement Industries, Inc.
Fort Myers, Florida

Florida Precast Industries, Inc.
Sebring, Florida

EAST
Mid-Atlantic Precast, LLC.
King George, Virginia

EGYPT
Samcrete Eqypt
Ahram, Giza

MEXICO
ITISA
Mexico City, Mexico

Spancrete Noreste
Monterrey, Mexico

CROATIA
Mucić & Co
Dugopolje, Croatia

CARIBBEAN
Preconco Limited
Barbados, West Indies

TURKEY
Yapi-Merkezi
Camlica-Istanbul, Turkey

UAE
Hi-Tech Concrete Products LLC
Abu Dhabi, UAE

MACHINE MANUFACTURER
Spancrete Machinery Corporation
N16 W23415 Stoneridge Drive
Waukesha, WI 53188
Telephone: 414-290-9000
Fax: 414-290-9130
www.spancrete-machinery.com

SPANCRETE IS ALSO MANUFACTURED IN:

Armenia
China
Denmark
Guatemala
Hungary
Ireland
Japan
Russia
South Korea
Switzerland

SPANCRETE MANUFACTURERS ASSOCIATION MEMBERS