J. P. CARRARA & SONS, INC

ERNEST J. BROD
12-13-13

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SUBMITTED
JAN 30 2014
J. P. CARRARA & SONS, INC.
MIDDLEBURY, VT 05753

MIDDLEBURY BRIDGE
JPC # 23411-013

LIFTING DESIGN CALCULATIONS
(WORK W/ CARRARA SHOP DWNGS)

ABUTMENTS

ABUTMENT 1 (PIECE MARKS SLAB-1 & SLAB)

WT 64500 LB, MAX

STATE OF VERMONT
ERNEST J. BROD
No. 2014
REGISTERED
PROFESSIONAL ENGINEER

STRIPPING

THERE ARE (4) LIFT POINTS IN FACE OF PANEL

~ 1/5 POINTS E.W.

STRESSES O.K. BY INSPECTION
ASSUME 60° MIN SLING ANGLE W/ HORIZONTAL
LOAD/LIFT POINT = 64500/ ↑×0.866 ≈ 18620 LB

FROM ATTACHED PRODUCT LITERATURE

USE 20T × 19 3/4" S.L. SWL (4:1 S.F.) = 40000 LB
O.K.

ERECTION

USE (2) LIFTERS IN TOP EDGE OF PANEL

& (2) BOTTOM FACE LIFTERS

STRESSES O.K. BY INSPECTION

BOTTOM LIFTERS

TENSION LOAD/LIFTER = $\frac{0.63 \times 64500}{2}$ = 20318 LB

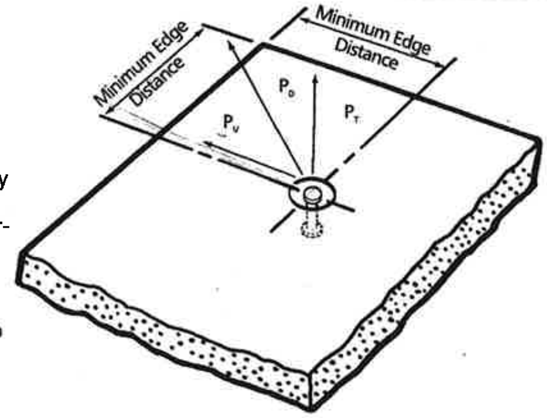
USE 20T × 19 3/4 S.L. SWL (4:1 S.F.) = 40000 LB
O.K.

TOPS

2

P-52 Swift Lift® Anchor Tensile and Shear Capacity

When anchors are used in the face of thin concrete elements



The following table lists the P-52 Swift Lift Anchors that are currently manufactured. Other sizes and lengths are available on special order. However, the sizes and lengths of anchors shown will handle the majority of flat precast concrete elements.

When the P-52 Swift Lift Anchor is properly embedded in normal weight concrete, the tabulated working loads are applicable for any direction of load. This applies even if the direction of load is parallel to the axis of the anchor, perpendicular to it or at any other angle.

Minimum distance between anchors is twice the minimum edge distance.

It is critical to remember that in order to obtain the safe working loads listed in the table below, the normal weight concrete must have obtained the minimum concrete strength shown, prior to initial load application.

Swift Lift Anchor Ton x Length	Safe Working Load	Minimum Concrete Strength	Minimum Edge Distance
1 ton x 2-5/8"	1,700 lbs.	3,500 psi	8"
1 ton x 3-3/8"	2,000 lbs.	2,200 psi	10"
1 ton x 4-3/8"	2,000 lbs.	1,600 psi	10"
1 ton x 8"	2,000 lbs.	1,600 psi	10"
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4 ton x 4-3/4"	5,800 lbs.	3,500 psi	14"
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4 ton x 14"	8,000 lbs.	1,600 psi	18"
4 ton x 19"	8,000 lbs.	1,600 psi	20"
8 ton x 4-3/4"	6,400 lbs.	3,500 psi	16"
8 ton x 6-3/4"	11,200 lbs.	3,500 psi	21"
8 ton x 10"	16,000 lbs.	3,500 psi	19"
8 ton x 13-3/8"	16,000 lbs.	1,600 psi	23"
8 ton x 26-3/4"	16,000 lbs.	1,600 psi	27"
20 ton x 10"	25,000 lbs.	3,500 psi	24"
20 ton x 19-3/4"	40,000 lbs.	3,500 psi	31"

Safe Working Loads provide a factor of safety of approximately 4 to1 in normal weight concrete. Safe Working Load is based on anchor setback from face of concrete "X" dimension, as shown on page 26.

TOP EDGE LIFTERS

SHEAR LOAD/LIFTER $= \frac{0.37 \times 64500}{2} = 11932$ LB

FOR $20^T \times 19\frac{3}{4}"$ S.L. DIAMETER OF SHAFT IS $1\frac{1}{2}"$

LIFTER SHEAR STRENGTH IS GOVERNED BY CONCRETE STRENGTH, SEE PCI DESIGN EQUATIONS 6.5.6 E.C.&7 ATTACHED

$$\phi V_c = \frac{\phi\, 800 A_b \sqrt{f'_c}}{1000} = \frac{0.85 \times 800 \times \pi \times 0.75^2 \sqrt{5000}}{1000}$$

$$= 84.9^k$$

OR $$\phi V_c = \frac{\phi\, 2\pi d_e^2 \sqrt{f'_c}}{1000} = \frac{0.85 \times 2 \times 3.14 \times \left(\frac{24}{2}\right)^2 \sqrt{5000}}{1000}$$

$$= 54.3^k \text{, GOVERNS}$$

FOR S.F. 4:1 SWL $= 54.3/4 = 13.6^k > 11.9^k$, O.K.

TENSION LOAD/LIFTER $= 64500/2 = 32500$ LB

FOR $20^T \times 19\frac{3}{4}"$ S.L

LIFTER TENSILE STRENGTH IS GOVERNED BY CONCRETE STRENGTH, SEE PCI FIGURE 6.15.7A ATTACHED

$$\phi P_c = \frac{\phi\, 2.67 \sqrt{f'_c}\, x_1 (y_1 + 2l_e)}{1000}$$

$$= \frac{0.85 \times 2.67 \sqrt{5000} \times 24 (0 + 2 \times 19.75)}{1000}$$

$$= 152.1^k$$

FOR S.F 4:1 SWL $= 152.1/4 = 38.0^k > 32.5^k$, O.K.

USE $20^T \times 19\frac{3}{4}"$ S.L., O.K.

4

where d_e is the distance measured from the stud axis to the free edge. If a stud is located in the corner of a concrete member, Eq. 6.5.3 should be applied twice, once for each edge distance.

For a group of studs, the concrete failure surface may be along a truncated pyramid rather than separate shear cones, as shown in Fig. 6.5.3.

For this case, the design tensile strength is:

$$\phi P_c = \phi\lambda\sqrt{f'_c}\,(2.8\,A_{slope} + 4\,A_{flat}) \qquad \text{(Eq. 6.5.4)}$$

where:

A_{slope} = area of the sloping sides

A_{flat} = area of the flat bottom of the truncated pyramid

For stud groups in thin members, the failure surface may penetrate the thickness of the member as shown in Fig. 6.5.4. The strengths based on this type of failure are P_{c2} values given in Fig. 6.5.3. For design, select the least of P_{c1}, P_{c2} or the sum of the individual capacities. Tables 6.20.8 through 6.20.12 are provided to calculate these values.

The design tensile strength per stud as governed by steel failure is:

$$\phi P_s = 0.9 A_b f_y = 54{,}000\,A_b \qquad \text{(Eq. 6.5.5)}$$

where ϕ = 1.0 and f_y = 60,000 psi. Table 6.20.6 tabulates the maximum design strengths from the above equations.

Shear

The design shear strength governed by concrete failure should be taken as the least of the values given by the following equations:

$$\phi V_c = \phi 800\,A_b\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.6)}$$

$$\phi V_c = \phi 2\pi d_e^2\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.7)}$$

where ϕ = 0.85

For groups of studs, the design shear strength, based on concrete strength, should be taken as the least of:

1. Strength of the weakest stud, based on the above equations, times the number of studs,
2. Strength based on the d_e of the weakest row of studs times the number of rows, or
3. Strength based on the d_e of the row of studs farthest from the free edge.

Note: These are based on "normal" arrangement of studs. For arrangements which are very unsymmetrical or unusual, a separate analysis, which considers the "zipper" effect, should be made.

Example 6.5.1 Shear strength of stud groups

Given:

A stud group in a column subject to the shear force shown.

f'_c = 5000 psi (normal weight)

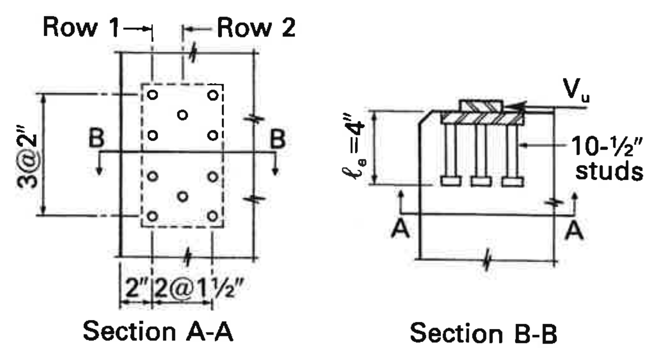


Fig. 6.5.4 Pullout surface areas for stud groups in thin sections

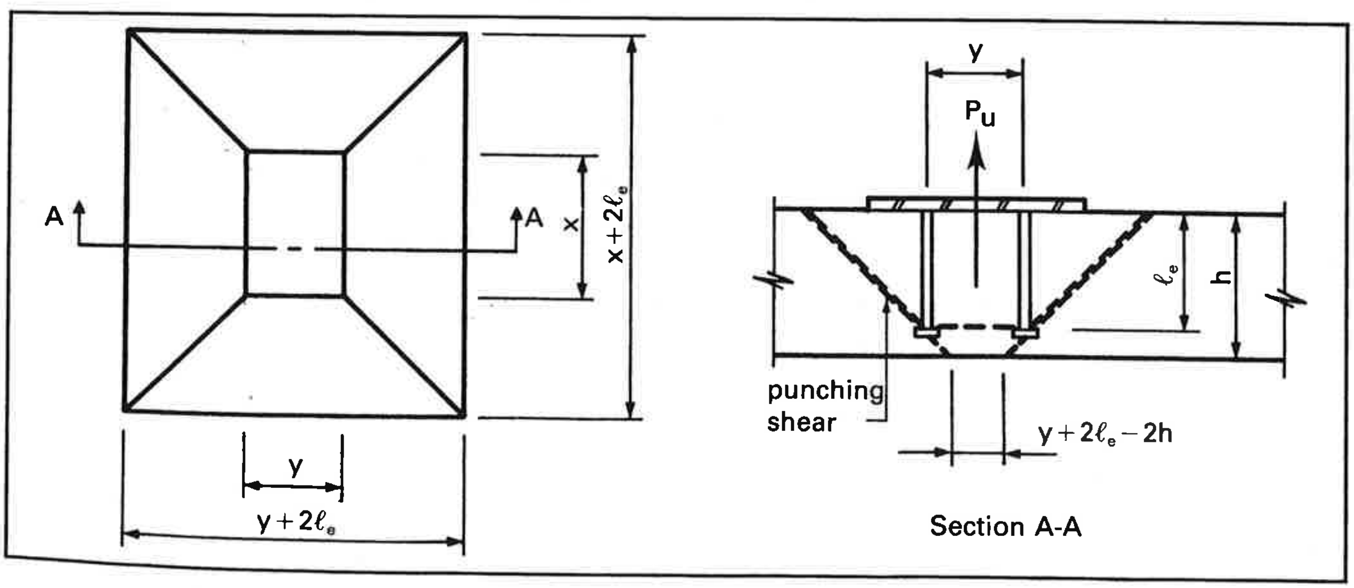
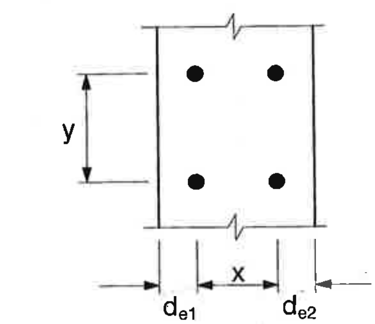


Figure 6.15.7A (continued) Design tensile strength for $h \geq h_{min}$, ϕP_{c1}—Case 3



x and y are the overall dimensions (width and length) of the stud group.

Case 3: Free edges on two opposite sides

$$\phi P_{c1} = \phi 2.67 \lambda \sqrt{f'_c}\,(x_1)(y_1 + 2\ell_e)$$

$$\phi = 0.85$$

where: x_1 and y_1 are the dimensions of the flat bottom of the part of the truncated pyramid.

For Case 3: $x_1 = x + d_{e1} + d_{e2}$ $\quad y_1 = y$

Note: Table values are based on $\lambda = 1.0$ and $f'_c = 5000$ psi; for different material properties, multiply table values by $\lambda \sqrt{f'_c / 5000}$

	x_1, in.	Design tensile strength, ϕP_{c1} (kips)														
ℓ_e in.	**y_1, in.**	**2**	**4**	**6**	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**24**	**26**	**28**	**30**
3	**0**	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	2	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	4	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	6	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	8	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	10	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	12	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	14	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	16	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
4	**0**	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	2	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	4	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	6	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	8	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	10	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	12	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	14	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
	16	8	15	23	31	39	46	54	61	69	77	85	92	100	108	115
6	**0**	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	2	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	4	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	6	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	8	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	10	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
	12	8	15	23	31	39	46	54	61	69	77	85	92	100	108	115
	14	9	17	25	33	42	50	59	67	75	83	92	100	109	117	125
	16	9	18	27	36	45	54	63	72	81	90	99	108	117	125	135
8	**0**	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	2	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	4	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	6	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
	8	8	15	23	31	39	46	54	61	69	77	85	92	100	108	115
	10	9	17	25	33	42	50	59	67	75	83	92	100	109	117	125
	12	9	18	27	36	45	54	63	72	81	90	99	108	117	125	135
	14	9	19	29	39	48	58	67	77	87	96	106	115	125	135	144
	16	10	21	31	41	51	61	72	82	92	103	113	123	133	143	154

ABUTMENT 2 (PIECE MARKS SHAB-3, SHAB-4 & SHAB-5)

WT 68600 LB, MAX

STRIPPING

THERE ARE (4) LIFT POINTS IN FACE OF PANEL
W/ 1/4 POINTS E.W.

STRESSES O.K. BY INSPECTION
ASSUME 60° MIN SLING ANGLE W/ HORIZONTAL
LOAD/LIFT POINT $= 68600 / (4 \times 0.866) = 19804$ LB

FROM ATTACHED PRODUCT LITERATURE

USE $20^T \times 19\frac{3}{4}''$ S.L SWL (4:1 S.F) = 40000 LB, O.K.

ERECTION

USE (2) LIFTERS IN TOP EDGE OF PANEL

& (2) BOTTOM FACE LIFTERS

CHECK BENDING STRESS (CONSERVATIVE)

$$M = wl^2/8 = 1.3 \times 300 \times 20.5^2 / 8 = 20487 \text{ LB-FT/FT}$$

$$S = bh^2/6 = 12 \times 24^2 / 6 = 1152 \text{ in}^3/\text{FT}$$

$$f = M/S = 20487 \times 12 / 1152 = 213 \text{ psi}$$

$$< 5\sqrt{5000} = 354 \text{ psi o.k.}$$

BOTTOM LIFTERS

TENSION LOAD/LIFTER $= \dfrac{0.63 \times 68600}{2} = 21609$ LB

USE $20^T \times 19\frac{3}{4}$ S.L SWL (4:1 S.F) = 40000 LB, O.K.

TOP EDGE LIFTER

SHEAR LOAD / LIFTER = $\frac{0.37 \times 68600}{2}$ = 12691 LB

BY PREVIOUS CALCULATIONS

FOR 20^T x 19 3/4 S.L. SHEAR SWL (4:1 S.F.) = 13.6^k > 12.7^k, OK

TENSION LOAD / LIFTER = 68600/2 = 34300 LB

BY PREVIOUS CALCULATION

FOR 20^T x 19 3/4 S.L. TENSION SWL (4:1 S.F.) = 38.0^k > 34.3^k, OK

USE 20^T X 19 3/4" S.L., OK

APPROACH SLABS

(PIECE MARKS SB-SB1 THRU SH-SB4)

WT 37820 LB, MAX

STRIPPING & ERECTION

THERE ARE (4) LIFT POINTS IN TOP FACE OF PANEL ~ 1/5 POINTS E.W.

STRESSES O.K. BY INSPECTION

ASSUME MAX SLING ANGLE W/HORIZONTAL = 60°

$$\text{LOAD/LIFT POINT} = \frac{37820}{4 \times 0.866} = 10918 \text{ LB}$$

FROM ATTACHED PRODUCT LITERATURE

USE 8T X 10" S.L. SWL (4:1 S.F.) = 16000 LB. OK

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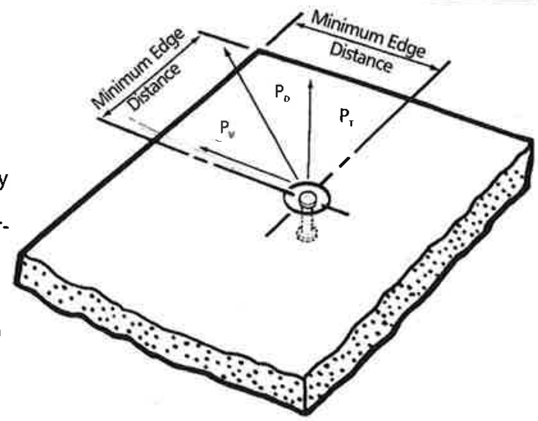
Technical Information | Swift Lift System | Fleet-Lift System | Lifting System | and Accessories | Connectors | Swift Lift® System

P-52 Swift Lift® Anchor Tensile and Shear Capacity

When anchors are used in the face of thin concrete elements



The following table lists the P-52 Swift Lift Anchors that are currently manufactured. Other sizes and lengths are available on special order. However, the sizes and lengths of anchors shown will handle the majority of flat precast concrete elements.

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Minimum distance between anchors is twice the minimum edge distance.

It is critical to remember that in order to obtain the safe working loads listed in the table below, the normal weight concrete must have obtained the minimum concrete strength shown, prior to initial load application.

Swift Lift Anchor Ton x Length	Safe Working Load	Minimum Concrete Strength	Minimum Edge Distance
1 ton x 2-5/8"	1,700 lbs.	3,500 psi	8"
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Safe Working Loads provide a factor of safety of approximately 4 to1 in normal weight concrete. Safe Working Load is based on anchor setback from face of concrete "X" dimension, as shown on page 26.

FOOTINGS

(PIECE MARKS SHF-1, SHF-2, SHF-4, SHF-5, SHF-6 & SHF-7)

WT 81890 LB, MAX
STRIPPING & ERECTION
THERE ARE (4) LIFT POINTS IN TOP FACE OF PANEL
W 1/5 POINTS E.W.

STRESSES O.K. BY INSPECTION

ASSUME MAX SLING ANGLE W/ ~~HORIZONTAL~~ = 60°

$$\text{LOAD/LIFT POINT} = \frac{81890}{4 \times 0.866} = 23640\ \text{LB}$$

FROM ATTACHED PRODUCT LITERATURE

USE $20^T \times 19\frac{3}{4}$ S.L SWL (4:1 S.F.) = 40000 LB, OK

NOTE: FOR LIFTERS LOCATED CLOSER THAN $19\frac{3}{4}$" FROM
A FREE EDGE MULTIPLY LIFTER CAPACITY
BY d_e/l_e = EDGE DISTANCE / LENGTH OF LIFTER

SHF-2 WT = 20.77^T & $\text{LOAD/LIFTER} = \frac{20.77 \times 2}{0.866 \times 4} = 12.0^k$

$$SWL(4:1\ S.F.) = \frac{9}{19\frac{3}{4}} \times \frac{40000}{1000} = 18.3^k > 12.0^k, \text{OK}$$

SHF-4 WT = 20.38^T & $\text{LOAD/LIFTER} = \frac{20.38 \times 2}{0.866 \times 4} = 11.8^k$

$$SWL(4:1\ S.F.) = \frac{9}{19\frac{3}{4}} \times \frac{40000}{1000} = 18.3^k > 11.8^k, \text{OK}$$

SHF-5 WT = 28.81^T & $\text{LOAD/LIFTER} = \frac{28.81 \times 2}{0.866 \times 4} = 16.6^k$

$$SWL(4:1\ S.F.) = \frac{14}{19\frac{3}{4}} \times \frac{40000}{1000} = 28.4^k > 16.6^k, \text{OK}$$

TOPS. 35500

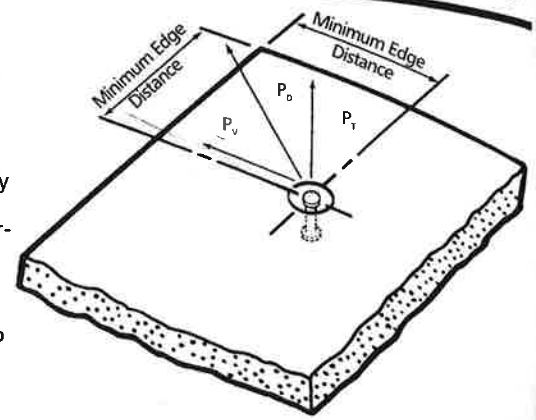
Swift Lift® System

11

DAYTON® SUPERIOR

P-52 Swift Lift® Anchor Tensile and Shear Capacity

When anchors are used in the face of thin concrete elements



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1 ton x 8"	2,000 lbs.	1,600 psi	10"
1 ton x 9-1/2"	2,000 lbs.	1,600 psi	10"
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WING WALLS

WINGWALL-1 & WINGWALL-2 (PIECE MARK SH-WW1 & SH-WW2)

WT 79000 LB, MAX

STRIPPING

THERE ARE (4) LIFT POINTS IN FACE OF PANEL

@ 1/5 POINTS E.W.

STRESSES OK BY INSPECTION
ASSUME 60° SLING ANGLE W/ HORIZONTAL
LOAD/LIFT POINT = 79000 / (4 x 0.866) = 22806 LB

FROM ATTACHED PRODUCT LITERATURE

USE 20^T x 10" S.L. SWL (4:1 S.F.) = 25000 LB, OK

ERECTION

USE (4) LIFTERS IN TOP EDGE OF PANEL

& (2) BOTTOM FACE LIFTERS

STRESSES OK BY INSPECTION

BOTTOM LIFTER

TENSION LOAD/LIFTER = (0.63 x 79000) / 2 = 24885 LB

USE 20^T x 10" S.L. SWL (4:1 S.F.) = 25000 LB, OK.

TOP EDGE LIFTER

SHEAR LOAD/LIFTER = (0.37 x 79000) / 4 = 7308 LB

TOPS 35500

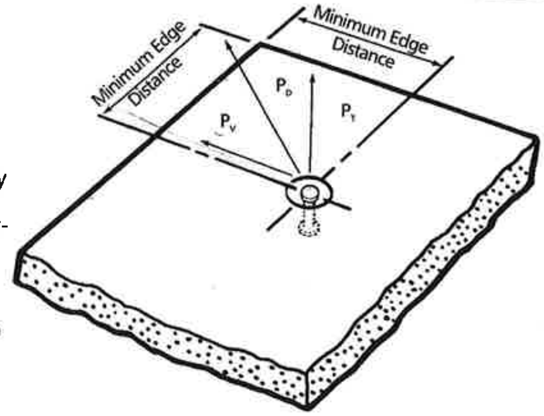
Swift Lift® System

13

DAYTON® SUPERIOR

P-52 Swift Lift® Anchor Tensile and Shear Capacity

When anchors are used in the face of thin concrete elements



The following table lists the P-52 Swift Lift Anchors that are currently manufactured. Other sizes and lengths are available on special order. However, the sizes and lengths of anchors shown will handle the majority of flat precast concrete elements.

When the P-52 Swift Lift Anchor is properly embedded in normal weight concrete, the tabulated working loads are applicable for any direction of load. This applies even if the direction of load is parallel to the axis of the anchor, perpendicular to it or at any other angle.

Minimum distance between anchors is twice the minimum edge distance.

It is critical to remember that in order to obtain the safe working loads listed in the table below, the normal weight concrete must have obtained the minimum concrete strength shown, prior to initial load application.

Swift Lift Anchor Ton x Length	Safe Working Load	Minimum Concrete Strength	Minimum Edge Distance
1 ton x 2-5/8"	1,700 lbs.	3,500 psi	8"
1 ton x 3-3/8"	2,000 lbs.	2,200 psi	10"
1 ton x 4-3/8"	2,000 lbs.	1,600 psi	10"
1 ton x 8"	2,000 lbs.	1,600 psi	10"
1 ton x 9-1/2"	2,000 lbs.	1,600 psi	10"
2 ton x 2-3/4"	2,100 lbs.	3,500 psi	8"
2 ton x 3-3/8"	2,900 lbs.	3,500 psi	10"
2 ton x 5-1/2"	4,000 lbs.	1,600 psi	13"
2 ton x 6"	4,000 lbs.	1,600 psi	13"
2 ton x 6-3/4"	4,000 lbs.	1,600 psi	13"
2 ton x 11"	4,000 lbs.	1,600 psi	14"
4 ton x 3-3/4"	4,000 lbs.	3,500 psi	12"
4 ton x 4-1/4"	4,900 lbs.	3,500 psi	13"
4 ton x 4-3/4"	5,800 lbs.	3,500 psi	14"
4 ton x 5-1/2"	7,400 lbs.	3,500 psi	17"
4 ton x 5-3/4"	7,900 lbs.	3,500 psi	17"
4 ton x 7-1/8"	8,000 lbs.	1,800 psi	20"
4 ton x 9-1/2"	8,000 lbs.	1,600 psi	17"
4 ton x 14"	8,000 lbs.	1,600 psi	18"
4 ton x 19"	8,000 lbs.	1,600 psi	20"
8 ton x 4-3/4"	6,400 lbs.	3,500 psi	16"
8 ton x 6-3/4"	11,200 lbs.	3,500 psi	21"
8 ton x 10"	16,000 lbs.	3,500 psi	19"
8 ton x 13-3/8"	16,000 lbs.	1,600 psi	23"
8 ton x 26-3/4"	16,000 lbs.	1,600 psi	27"
20 ton x 10"	25,000 lbs.	3,500 psi	24"
20 ton x 19-3/4"	40,000 lbs.	3,500 psi	31"

Safe Working Loads provide a factor of safety of approximately 4 to1 in normal weight concrete. Safe Working Load is based on anchor setback from face of concrete "X" dimension, as shown on page 26.

FOR $20^T \times 19\frac{3}{4}$" S.L., DIAMETER OF SHAFT IS $1\frac{1}{2}$"

LIFTER SHEAR STRENGTH IS GOVERNED BY CONCRETE STRENGTH. SEE PCI DESIGN EQUATIONS 6.58 & 6.57, ATTACHED

$$\phi V_c = \frac{0.85 \times 800 \times \pi \times 0.75^2 \sqrt{5000}}{1000} = 84.9^k$$

$$\text{or } \phi V_c = \frac{0.85 \times 2 \times 3.14 \times \left(\frac{19}{2}\right)^2 \sqrt{5000}}{1000} = 34.1^k, \text{ GOVERNS}$$

FOR S.F. 4:1 $\quad SWL = 34.1/4 = 8.52^k > 7.31^k$, OK

TENSION LOAD/LIFTER $= 79000/4 = 19750$ LBS

FOR $20^T \times 19\frac{3}{4}$" S.L.

LIFTER TENSILE STRENGTH IS GOVERNED BY CONCRETE STRENGTH, SEE PCI FIGURE 6.15.7A ATTACHED

$$\phi P_c = \frac{0.85 \times 2.67 \sqrt{5000} \times 19 \left(0 + 2 \times 19.75\right)}{1000}$$

$$= 120.5^k$$

FOR S.F 4:1 $\quad SWL = 120.5/4 = 30.1^k > 19.8^k$, O.K

USE $20^T \times 19\frac{3}{4}$" S.L., OK

WINGWALL-3 & WINGWALL-4 (PIECE MARK SH-WW3 & SH-WW4)

WT 78400 LB, MAX

STRIPPING

THERE ARE (4) LIFT POINTS IN FACE OF PANEL

@ 1/5 POINTS E.W

STRESSES O.K. BY INSPECTION
ASSUME 60° SLING ANGLE W/HORIZONTAL
LOAD/LIFTER = 78400/(4x0.866) = 22632 LB
FROM ATTACHMENT PRODUCT LITERATURE

USE 20T x 10" S.L. SWL (4:1 S.F.) = 25000, LB

ERECTION

USE (4) LIFTERS IN TOP EDGE OF PANEL

& (2) BOTTOM FACE LIFTERS @ N 1/5 POINT

CHECK STRESSES

$M = 0.067\,w\ell^2$

$w = 78400/(13.0 \times 24.42) = 247$ PSF

$M = 0.067 \times 1.3 \times 247 \times 24.42^2 = 12829$ LB-FT/FT

$S = bh^2/6 = 12 \times 18^2/6 = 648\ \text{in}^3/\text{ft}$

$f = 12829 \times 12/648 = 238$ PSI $< 5\sqrt{5000}$
$= 354$ PSI, O.K.

BOTTOM LIFTERS

TENSION LOAD/LIFTER $= \dfrac{0.63 \times 78400}{2}$

$= 24696$ LB

FROM ATTACHMENT PRODUCT LITERATURE

USE 20T x 10" S.L. SWL = 25000 LB, OK

TOP EDGE LIFTERS

SHEAR LOAD/LIFTER = $\frac{0.37 \times 78400}{4}$ = 7252 LB

TENSION LOAD/LIFTER = 78400/4 = 19600 LB

BY PREVIOUS PIECEMARK SH-WW1 & SH-WW2

CALCULATIONS,

USE $20^T \times 19\frac{3}{4}''$ S.L., OK where d_e is the distance measured from the stud axis to the free edge. If a stud is located in the corner of a concrete member, Eq. 6.5.3 should be applied twice, once for each edge distance.

For a group of studs, the concrete failure surface may be along a truncated pyramid rather than separate shear cones, as shown in Fig. 6.5.3.

For this case, the design tensile strength is:

$$\phi P_c = \phi\lambda\sqrt{f'_c}\,(2.8\,A_{slope} + 4\,A_{flat}) \qquad \text{(Eq. 6.5.4)}$$

where:

A_{slope} = area of the sloping sides

A_{flat} = area of the flat bottom of the truncated pyramid

For stud groups in thin members, the failure surface may penetrate the thickness of the member as shown in Fig. 6.5.4. The strengths based on this type of failure are P_{c2} values given in Fig. 6.5.3. For design, select the least of P_{c1}, P_{c2} or the sum of the individual capacities. Tables 6.20.8 through 6.20.12 are provided to calculate these values.

The design tensile strength per stud as governed by steel failure is:

$$\phi P_s = 0.9 A_b f_y = 54{,}000\,A_b \qquad \text{(Eq. 6.5.5)}$$

where ϕ = 1.0 and f_y = 60,000 psi. Table 6.20.6 tabulates the maximum design strengths from the above equations.

Shear

The design shear strength governed by concrete failure should be taken as the least of the values given by the following equations:

$$\phi V_c = \phi 800\,A_b\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.6)}$$

$$\phi V_c = \phi 2\pi d_e^2\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.7)}$$

where ϕ = 0.85

For groups of studs, the design shear strength, based on concrete strength, should be taken as the least of:

1. Strength of the weakest stud, based on the above equations, times the number of studs,
2. Strength based on the d_e of the weakest row of studs times the number of rows, or
3. Strength based on the d_e of the row of studs farthest from the free edge.

Note: These are based on "normal" arrangement of studs. For arrangements which are very unsymmetrical or unusual, a separate analysis, which considers the "zipper" effect, should be made.

Example 6.5.1 Shear strength of stud groups

Given:

A stud group in a column subject to the shear force shown.

f'_c = 5000 psi (normal weight)

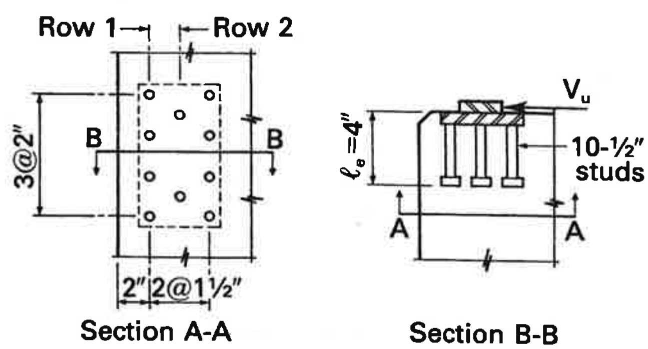


Fig. 6.5.4 Pullout surface areas for stud groups in thin sections

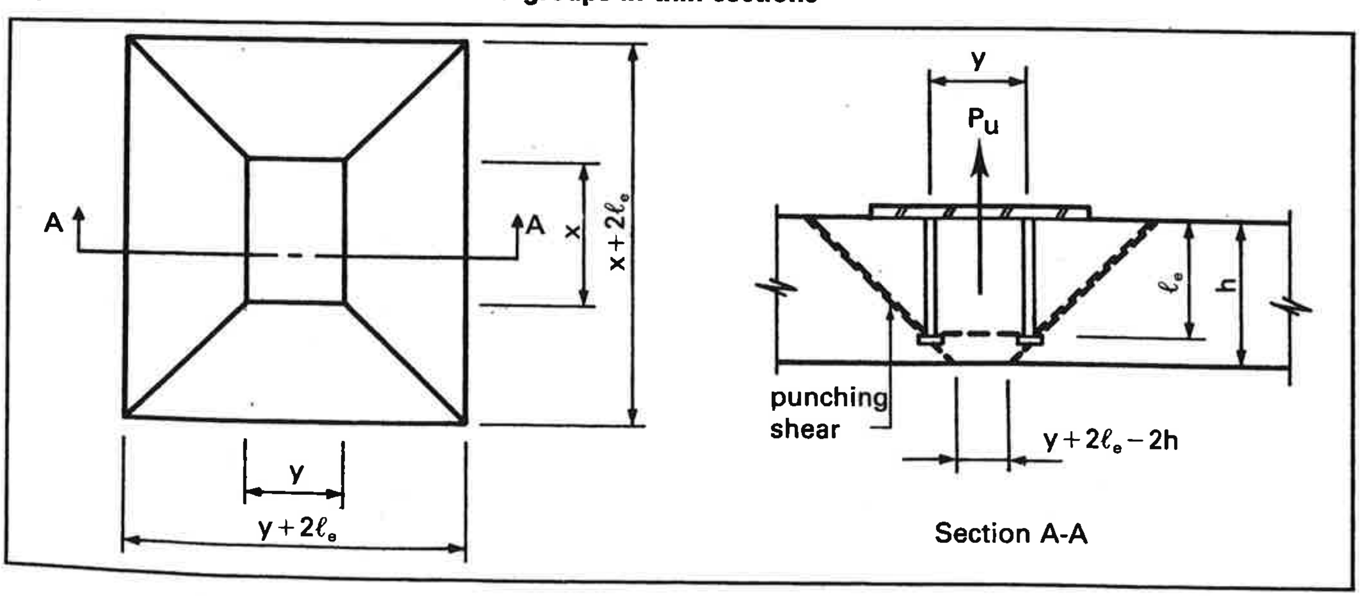


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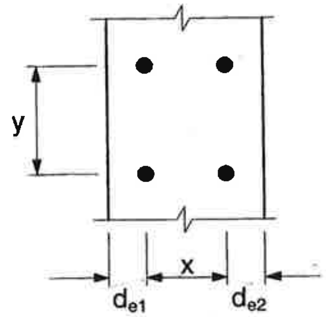
Figure 6.15.7A (continued) Design tensile strength for $h \geq h_{min}$, ϕP_{c1}—Case 3



x and y are the overall dimensions (width and length) of the stud group.

Case 3: Free edges on two opposite sides

$$\phi P_{c1} = \phi 2.67 \lambda \sqrt{f'_c}\,(x_1)(y_1 + 2\ell_e)$$

$$\phi = 0.85$$

where: x_1 and y_1 are the dimensions of the flat bottom of the part of the truncated pyramid.

For Case 3: $x_1 = x + d_{e1} + d_{e2}$ $\quad y_1 = y$

Note: Table values are based on $\lambda = 1.0$ and $f'_c = 5000$ psi; for different material properties, multiply table values by $\lambda\sqrt{f'_c/5000}$

ℓ_e in.	y_1, in. \ x_1, in.	Design tensile strength, ϕP_{c1} (kips)														
		2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
3	0	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	2	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	4	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	6	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	8	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	10	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	12	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	14	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	16	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
4	0	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	2	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	4	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	6	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	8	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	10	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	12	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	14	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
	16	8	15	23	31	39	46	54	61	69	77	85	92	100	108	115
6	0	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	2	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	4	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	6	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	8	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	10	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
	12	8	15	23	31	39	46	54	61	69	77	85	92	100	108	115
	14	9	17	25	33	42	50	59	67	75	83	92	100	109	117	125
	16	9	18	27	36	45	54	63	72	81	90	99	108	117	125	135
8	0	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	2	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	4	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
	6	7	14	21	28	35	42	49	57	63	71	77	85	92	99	106
	8	8	15	23	31	39	46	54	61	69	77	85	92	100	108	115
	10	9	17	25	33	42	50	59	67	75	83	92	100	109	117	125
	12	9	18	27	36	45	54	63	72	81	90	99	108	117	125	135
	14	9	19	29	39	48	58	67	77	87	96	106	115	125	135	144
	16	10	21	31	41	51	61	72	82	92	103	113	123	133	143	154

JOSEPH P. CARRARA & SONS, INC. LETTER OF TRANSMITTAL

2464 CASE STREET
MIDDLEBURY, VERMONT 05753

TEL (802) 388-6363
FAX (802) 388-9010

DATE : **January 30, 2014**	JOB NO. : **23411-013**
ATTENTION : **Mr. Brian Emmons** ***bemmons@tbuckcon.net***	
RE : **Sand Hill Bridge No. 13**	
VT 125 over Middlebury River	
Middlebury, VT	
Project No. RS 0174 (8)	
Transmittal #11	**Precast Concrete**

TO: **T. Buck Construction**
249 Merrow Road
Auburn, Maine 04210

207-783-6223 x205

WE ARE SENDING YOU [x] Attached [ ] Under separate cover via **Email** the following items:

[x] Shop Drawings [ ] Prints [ ] Plans [ ] Samples [ ] Specifications

[ ] Copy of Letter [ ] Change Order [ ] Other:

COPIES	DATE	NO.	DESCRIPTION
1	**1/30/14**	**as noted**	**JPC Lifting Design Calculations for Footings, Abutments, Wingwalls, & Approach Slabs**

THESE ARE TRANSMITTED as checked below :

[x] For approval [ ] Approved as submitted [ ] Resubmit ____ copies for approval

[ ] For your use [ ] Approved as noted [ ] Submit ____ copies for distribution

[ ] As requested [ ] Returned for corrections [ ] Return ____ corrected prints

[ ] For review and comment [ ] ____

[ ] FOR BIDS DUE ____ 20 ____ [ ] PRINTS RETURNED AFTER LOAN TO US

REMARKS **Brian,**

The attached design calculations are submitted for the lifting of precast footings, abutments, wingwalls, and approach slabs.

Please contact us at 802-388-6363 with any questions.

COPY TO :

SIGNED : Michael Weigand

If enclosures are not as noted, please notify us at once.