Waitsfield BRF 013-4(39)

Bridge 177, VT 100 over Mad River

Waitsfield, VT

Lift Loop / Lift Insert Design Calculations

For Precast Concrete Substructure Components

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VT AOT

Engineer:
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Date:
May 6, 2016

STATE OF VERMONT
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CIVIL
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PROFESSIONAL ENGINEER

SUBMITTED

MAY 10 2016

J. P. CARRARA & SONS, INC.
MIDDLEBURY, VT 05753

WAITSFIELD BRF 013-4(39)

BRIDGE 177, VT 100 OVER MAD RIVER

WAITSFIELD, VT

LIFT LOOP / LIFT INSERT DESIGN CALCULATIONS

FOR PRECAST CONCRETE COMPONENTS
(W/ORK WITH JPC SHOP DRAWINGS)

APPROACH SLABS - ABUTMENTS 1 & 2:

$f'_c = 5000$ psi $f'_{ci} = 3500$ psi 25'-0" LONG x 7'-1¾" WIDE x 16" THICK

$W_\ell = 17.9T = 35.85$ KIPS

THERE ARE (4) LIFT LOCATIONS & ASSUME 60° LIFT ∡ WITH HORIZONTAL:

DESIGN LOAD / LIFT LOCATION = $35.85^K / (4)(0.866) = 10.3K$

FROM ATTACHED PRODUCT LITERATURE, "P.24" FROM DAYTON:

USE 8TON x 6¾" SWIFT LIFT ANCHOR

SWL (4:1 S.F.) = 11.2K > 10.3K, OK.

CENTER LIFTING PATTERN ON C.G. OF APPROACH SLABS

FOOTINGS - ABUTMENT #1 (MK. FTG1 - FTG3)

$f'_c = 5000$ psi $f'_{ci} = 3500$ psi 30" THICK x 7'-6" WIDE

SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY / HEIGHT

$W_{T\cdot MAX} = 49K$ ("FTG2")

THERE ARE (4) LIFT LOCATIONS &
ASSUME A 60° LIFT ANGLE WITH THE HORIZONTAL

DESIGN LOAD / LIFT LOCATION $= 49^K / (4)(0.866) = 14.1^K$

FROM ATTACHED PCI LITERATURE,
USE (2) ½"φ 270KSI STRAND LIFT LOOPS WITH 25" MINIMUM EMBEDMENT AT EACH LOCATION - ALLOWS FOR 3" CLR AT BOTTOM & UP TO 2" RECESS AT TOP

FROM TABLE 5.2.3; EXTRAPOLATE CAPACITY FOR 25" EMBEDMENT & 60° SLING ANGLE BETWEEN 45° & VERTICAL LIFT:

$$\text{CAPACITY} = \left(13 + \frac{18-13}{2} + 19 + \frac{25.5-19}{2}\right)/2 = 18.9^K > 14.1^K \text{ OK.}$$

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 5

$$\phi P_c = \phi\, 2.67 \lambda \sqrt{f'_{ci}}\,(x_1)(y_1 + l_e)$$

$$= (0.85)(2.67)(1.0)(3500)^{0.5}(18" + 20")(12" + 25")/1000$$

$$= 188.8^K$$

FOR S.F. 4:1 SWL $= 188.8/4 = 47^K > 14.1^K$, O.K.

Rt 100, Bridge 177
Waitsfield, VT

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Footing CG Calculation

Waitsfield, VT Bridge **FTG1** Abutment #1

Area	x	y	z		x-bar	y-bar	z-bar	Volume	x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3			
1	188.25	30	90	rect	94.125	15	45	294.140625	27685.99	4412.109	13236.33
2	90	30	-90	tri	30	15	60	-70.3125 tri	-2109.375	-1054.688	-4218.75
3	88.5	8.0625	30		144	34.03125	15	12.3876953	1783.828	421.5688	185.8154
4	99.75	7.0625	30		49.875	33.53125	15	12.2306315	610.0027	410.1084	183.4595
5	30	7.0625	-30	tri	10	33.53125	20	-1.8391927 tri	-18.39193	-61.67043	-36.78385
									27952.05	4127.429	9350.069
								246.607259 ft^3			
									113.3464 in.	16.73685 in.	37.91482 in.
								36,991 lbs.	x-bar	y-bar	z-bar
								18.50 T	9.445535 ft.	1.394737 ft.	3.159568 ft.
									9' 5-3/8"	1' 4-3/4"	3' 1-15/16"

Waitsfield, VT Bridge **FTG2** Abutment #1

Area	x	y	z		x-bar	y-bar	z-bar	Volume	x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3			
1	192	30	90	rect	96	15	45	300	28800	4500	13500
3	192	8.0625	30		96	34.03125	15	26.875	2580	914.5898	403.125
									31380	5414.59	13903.13
								326.875 ft^3			
									96 in.	16.56471 in.	42.53346 in.
								49,031 lbs.	x-bar	y-bar	z-bar
								24.52 T	8 ft.	1.380393 ft.	3.544455 ft.
									8' 0"	1' 4-9/16"	3' 6-9/16"

Waitsfield, VT Bridge **FTG3** Abutment #1

Area	x	y	z		x-bar	y-bar	z-bar	Volume	x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3			
1	247.25	30	90	rect	123.625	15	45	386.328125	47759.81	5794.922	17384.77
2	90	30	-90	tri	217.25	15	30	-70.3125 tri	-15275.39	-1054.688	-2109.375
3	78.75	5.0625	30		39.375	32.53125	15	6.92138672	272.5296	225.1614	103.8208
4	108.5	2.0625	30		133	31.03125	15	3.88509115	516.7171	120.5592	58.27637
5	30	2.0625	-30	tri	177.25	31.03125	10	-0.5371094 tri	-95.20264	-16.66718	-5.371094
									33178.47	5069.288	15432.12
								326.284993 ft^3			
									101.6855 in.	15.53638 in.	47.29643 in.
								48,943 lbs.	x-bar	y-bar	z-bar
								24.47 T	8.473796 ft.	1.294698 ft.	3.94137 ft.
									8' 5-11/16"	1' 3-9/16"	3' 11-5/16"

WATTSFIELD BRF 013-4(39)

LEVELING JACK DESIGN CALCULATIONS

(WORK WITH JPC SHOP DRAWINGS)

THERE ARE (3) PRECAST FOOTINGS AT ABUTMENT #1 :

MK. "MD-FG1" $W_T = 17.92T = 35.8^k$ (7) LEVELING JACKS

LOAD / LEVELING JACK = $35.8/7 = 5.12^k$

MK. "MD-FG2" $W_T = 24.01T = 48^k$ (10) LEVELING JACKS

LOAD / LEVELING JACK = $48/10 = 4.8^k$

MK. "MD-FG3" $W_T = 23.9T = 47.8^k$ (9) LEVELING JACKS

LOAD / LEVELING JACK = $47.8/9 = 5.31^k$ ← CONTROLS

$P_u = 1.4 \times 5.31 = 7.44^k$

BEARING ON PL IN FOOTING.

$A = 6 \times 6 - \pi \cdot 2^2 = 23.43 \text{ in}^2$; 4" ϕ SLEEVE @ ℄ of PL

$\phi P_n = 0.70 \times 0.85 \times 5.0 \times 23.43 = 69.7^k \gg 7.44^k$, O.K.

SHEAR STRENGTH OF BEARING PLATE (3/4")

$P = 5.31^k$

PERIMETER OF 1½" HEAVY HEX NUT

$= 6 \times \frac{2\frac{3}{8}}{2} \times \frac{1}{0.866} = 8.23"$

$P_{allow} = 14.4 \times 8.23 \times 0.75 = 88.9^k \gg 5.31^k$, O.K.

(VERIFYING JACK DESIGN CALCULATIONS)

BOLT

$P = 5.31^k$

ALLOWABLE TENSION/COMPRESSION FOR

$1\frac{1}{2}''\phi$ BOLT $= 35.3^k >> 5.31^k$

A307

BEARING ON PC IN SUPPORT SLAB, $\frac{1}{2}''$ THICK

ASSUME $f'_c = 3500$ PSI CONCRETE IN MUD SLAB

AREA OF $1\frac{1}{2}''\phi$ BOLT $= 1.767 in^2$

ALLOWABLE BEARING OF PC $= 0.9 \times 36 \times 1.767 = 57.3^k > 5.31^k$, O.K.

BEARING STRENGTH OF CONCRETE $= \pi \times \left(1\frac{1}{2} + \frac{2 \times \frac{1}{2}''}{2}\right)^2 \times 0.7 \times 0.85 \times 3.5 ksi$

$= 10.2^k > 7.44^k$, O.K.

BACKWALLS - ABUTMENTS #1 & #2 (MK. BW1 - BW7)

$f'_c = 5000$ psi $f'_{ci} = 3500$ psi

SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY & WEIGHT

$W_{TOTAL} = 12.5T = 25.K$ ("BW2")

• FOR STRIPPING:

THERE ARE (4) LIFT ANCHORS IN FACE OF BACKWALLS, FOR STRIPPING & HORIZONTAL HANDLING. ASSUME 60° SLING ANGLE WITH HORIZONTAL

DESIGN LOAD / LIFT LOCATION = $25^K / (4)(0.866) = 7.2K$

FROM ATTACHED PRODUCT LITERATURE FROM DAYTON, "12.24", USE 4 TON x 7 1/8" S.L. ANCHOR

SWL (4:1 S.F.) = 8K > 7.2K, O.K.

CENTER LIFTER PATTERN ON C.G. OF BACKWALLS

• FOR ROLLUP & ERECTION:

THERE ARE (2) LIFT ANCHORS IN TOP OF PANELS

SHEAR LOAD, V, AT TOP INSERT FOR ROLLUP:

$V = 25K / (2)(2) = 6.3K$

FROM ATTACHED PRODUCT LITERATURE, FROM DAYTON, "P.26", USE 8 TON x 13 3/8" SWIFT LIFT ANCHORS,

SWL (S.F. 4:1) = 6.7K IN 9" THICK PANEL & $f'_c = 4500$ psi

∴ BY INSPECTION FOR 15" THICK PANELS & $f'_c = 5000$ psi

SWL (S.F. 4:1) > 6.3K APPLIED SHEAR LOAD, O.K.

EVALUATE TOP INSERTS IN TENSION WHEN BACKWALL IS HANGING FROM (2) ANCHORS AT TOP EDGE:

TENSION LOAD, $T = 25^K/2 = 12.5^K$

FROM ATTACHED PRODUCT LITERATURE, "P.28".

USE 8 TON x 13 3/8 SWIFT LIFT ANCHORS

MECHANICAL SWL (4:1 S.F.) IN TENSION = 16.1K OK. ✓

ALSO EVALUATE TENSILE STRENGTH IN CONCRETE *

USING PCI FIG: 6.15.7A, CASE 5, ATTACHED.*

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

$$= (0.85)(2.67)(1.0)(5000)^{0.5}(15")(12" + 13\tfrac{3}{8}")/1000$$

$$= 61.1^K$$

FOR S.F. 4:1: $SWL = 61.1/4 = 15.3^K > 12.5^K$ OK. ✓

* EVALUATION IS CONSERVATIVE AS MOST CONSERVATIVE EDGE DISTANCES ARE USE TO EVALUATE CONCRETE AND APPLIED TENSION LOAD IS LOWER FOR THESE CASES.

Rt 100, Bridge 177
Waitsfield, VT

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4/7/16 Rev 0

Backwall CG Calculation

Waitsfield, VT Bridge **BW1** Abutment #1

Area	x	y	z		x-bar	y-bar	z-bar	Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3				
1	176.8125	67.375	21	rect	88.40625	33.6875	10.5	144.772909		12798.83	4877.037	1520.116
2	21	67.375	-21	tri	169.8125	33.6875	7	-8.5973307	tri	-1459.934	-289.6226	-60.18132
3	34.6875	67.375	-9		138.4688	33.6875	4.5	-12.172241		-1685.475	-410.0524	-54.77509
										9653.421	4177.362	1405.159
								124.003337	ft^3			
										77.84807 in.	33.6875 in.	11.33162 in.
								18,601	lbs.	x-bar	y-bar	z-bar
								9.30	T	6.487339 ft.	2.807292 ft.	0.944302 ft.
										6' 5-7/8"	2' 9-11/16"	0' 11-5/16"

Waitsfield, VT Bridge **BW2** Abutment #1

Area	x	y	z		x-bar	y-bar	z-bar	Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3				
1	202.5	65.875	21	rect	101.25	32.9375	10.5	162.114258		16414.07	5339.638	1702.2
2	101.75	2.5	21	tri	67.83333	66.70833	10.5	1.54568142	tri	104.8487	103.1098	16.22965
3	100.75	2	21	rect	152.125	66.875	10.5	2.44878472		372.5214	163.7625	25.71224
4	100.75	0.5	21	tri	135.3333	68.04167	10.5	0.30609809	tri	41.42527	20.82742	3.21403
										16932.86	5627.338	1747.356
								166.414822	ft^3			
										101.7509 in.	33.81513 in.	10.5 in.
								24,962	lbs.	x-bar	y-bar	z-bar
								12.48	T	8.479245 ft.	2.817927 ft.	0.875 ft.
										8' 5-3/4"	2' 9-13/16"	0' 10-1/2"

Waitsfield, VT Bridge **BW3** Abutment #1

Area	x	y	z		x-bar	y-bar	z-bar	Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3				
1	223.8125	60.375	21	rect	111.9063	30.1875	10.5	164.21659		18376.86	4957.2884	1724.274
2	223.8125	5.4375	21	tri	149.2083	62.1875	10.5	7.3948466	tri	1103.373	459.86702	77.64589
3	25.625	60.375	-9		12.8125	30.1875	4.5	-4.0289307	tri	-51.6207	-121.6233	-18.1302
4	12	60.375	-12	tri	4	30.1875	17	-2.515625	tri	-10.0625	-75.94043	-42.7656
5	9	60.375	-9	tri	28.625	30.1875	6	-1.4150391	tri	-40.5055	-42.71649	-8.49023
										19378.05	5176.8752	1732.534
								163.65185	ft^3			
										118.4102 in.	31.633467 in.	10.58671 in.
								24,548	lbs.	x-bar	y-bar	z-bar
								12.27	T	9.867516 ft.	2.6361222 ft.	0.882226 ft.
										9' 10-7/16"	2' 7-5/8"	0' 10-9/16"

Rt 100, Bridge 177
Waitsfield, VT

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Wingwall / Back Wall CG Calculation

Waitsfield, VT Bridge **BW4** Abutment #2

Area	x	y	z		x-bar	y-bar	z-bar		Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in		ft^3				
1	177.625	66.5	15	rect	88.8125	33.25	7.5	1	102.535265		9106.413	3409.298	769.0145
2	177.625	1	15	tri	59.20833	66.83333	7.5	1	0.77094184	tri	45.64618	51.52461	5.782064
3	153.625	19.625	12	rect	100.8125	50.8125	21	1	20.9367405		2110.685	1063.848	439.6715
4	153.625	0.875	12	tri	126.4167	60.91667	21	1	0.46674262	tri	59.00405	28.4324	9.801595
5	153.625	12	12	tri	100.8125	37	19	1	6.40104167	tri	645.305	236.8385	121.6198
6	15	66.5	-15	tri	5	33.25	10	1	-4.3294271	tri	-21.64714	-143.9535	-43.29427
											11945.41	4645.988	1302.595
									126.781304	ft^3			
											94.22057 in.	36.64569 in.	10.27435 in.
									19,017	lbs.	x-bar	y-bar	z-bar
									9.51	T	7.851714 ft.	3.053807 ft.	0.856196 ft.
											7' 10-1/4"	3' 5/8"	0' 10-1/4"

Waitsfield, VT Bridge **BW5** Abutment #2

Area	x	y	z		x-bar	y-bar	z-bar		Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in		ft^3				
1	151.5	65.1875	15	rect	75.75	32.59375	7.5	1	85.7283529		6493.923	2794.209	642.9626
2	60.75	1.8125	15	tri	111	65.79167	7.5	1	0.47790527	tri	53.04749	31.44218	3.58429
	90.75	1.3125	15	rect	45.375	65.84375	7.5	1	1.03393555		46.91483	68.07819	7.754517
	90.75	0.5	15	tri	60.5	66.66667	7.5	1	0.1969401	tri	11.91488	13.12934	1.477051
3	60.75	18.1875	12	rect	121.125	50.09375	21	1	7.67285156		929.3741	384.3619	161.1299
	60.75	1.8125	12	tri	111	59.79167	21	1	0.38232422	tri	42.43799	22.8598	8.028809
	90.75	19.5	12	rect	45.375	50.75	21	1	12.2890625		557.6162	623.6699	258.0703
	90.75	0.5	12	tri	60.5	60.66667	21	1	0.15755208	tri	9.531901	9.55816	3.308594
	151.5	12	12	tri	75.75	37	19	1	6.3125	tri	478.1719	233.5625	119.9375
											8622.932	4180.871	1206.254
									114.251424	ft^3			
											75.4733 in.	36.5936 in.	10.55789 in.
									17,138	lbs.	x-bar	y-bar	z-bar
									8.57	T	6.289442 ft.	3.049466 ft.	0.879824 ft.
											6' 3-1/2"	3' 9/16"	0' 10-9/16"

WINGWALLS - ABUTMENT #1 (MK. WW1 & WW2)

$f'_c = 5000$ psi $f'_{ci} = 3500$ psi

SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY & WEIGHT.

$W_T = 31.56\,T = 63.1\,K$

THERE ARE (2) LIFT LOCATIONS BALANCED ABOUT CENTER OF GRAVITY. ASSUME A 60° LIFT ANGLE WITH THE HORIZONTAL

DESIGN LOAD / LIFT LOCATION $= 63.1 / (2)(0.866) = 36.4\,K$

FROM ATTACHED PCI LITERATURE,

USE (3) 0.600"ϕ 270 KSI STRAND LIFT LOOPS WITH 8'-0" MINIMUM EMBEDMENT & 2" RECESS PLUG AT TOP & 8" PROTECTION

FROM TABLE 5.2.3, EXTRAPOLATE CAPACITY FOR 60° LIFT ANGLE BETWEEN 45° AND VERTICAL LIFT

$$\text{CAPACITY} = \left(\frac{29+41}{2}\right) \times 1.1 = 38.5\,K > 36.4\,K$$

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 6

$$\phi P_c = \phi\, 2.67 \lambda \sqrt{f'_{ci}}\,(x_1)(y_1)$$

$$= (0.85)(2.67)(1.0)(3500)^{0.5}(15''+15'')(21''+21'')/1000$$

$$= 169.2\,K$$

FOR S.F. 4:1 : SWL $= 169.2/4 = 42.3\,K > 36.4\,K$ O.K. ✓

W-2

Rt 100, Bridge 177
Waitsfield, VT

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Wingwall CG Calculation

Waitsfield, VT Bridge **WW-1** Abutment #1

Area	size x in	y in	z in		x-bar in	y-bar in	z-bar in	Volume ft^3		x-bar*V	y-bar*V	z-bar*V
1	247.6875	99	54.4375	rect	123.8438	49.5	27.21875	772.491516		95668.25	38238.33	21026.25
2	54.4375	99	-54.4375	tri	18.14583	49.5	18.14583	-84.890249	tri	-1540.404	-4202.067	-1540.404
3	83.9375	33	-18	tri	219.7083	11	9	-14.426758	tri	-3169.679	-158.6943	-129.8408
4	83.9375	99	-36.4375	rect	205.7188	49.5	36.21875	-175.225		-36047.07	-8673.637	-6346.43
5	36.4375	30	-36.4375	tri	151.6042	15	42.29167	-11.525099	tri	-1747.253	-172.8765	-487.4156
6	36.4375	69	-27.9375		145.5313	64.5	40.46875	-40.648214		-5915.585	-2621.81	-1644.982
7	13.5625	69	-8.5		156.9688	64.5	22.25	-4.6032444		-722.5655	-296.9093	-102.4222
8	20.4375	69	-8.5	tri	143.375	64.5	23.66667	-3.4683431	tri	-497.2737	-223.7081	-82.08412
9	27.9375	69	-27.9375	tri	120.4375	64.5	45.125	-15.582977	tri	-1876.775	-1005.102	-703.1819
10	127.25	18	-1		63.625	15	53.9375	-1.3255208		-84.33626	-19.88281	-71.49528
										44067.31	20863.64	9917.997
								420.796115	ft^3			
										104.7237 in.	49.58136 in.	23.5696 in.
								63,119	lbs.	x-bar	y-bar	z-bar
								31.56	T	8.726971 ft.	4.13178 ft.	1.964133 ft.
										8' 8-3/4"	4' 1-9/16"	1' 11-9/16"

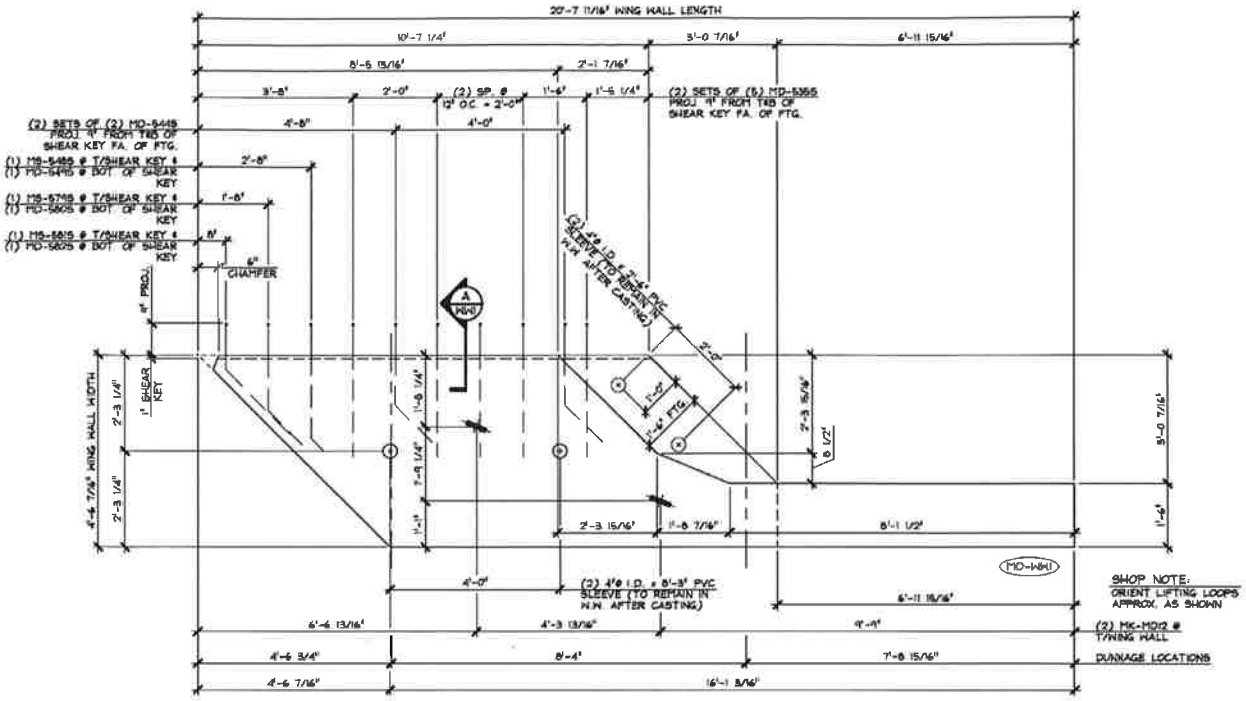
Waitsfield, VT Bridge **WW-2** Abutment #1

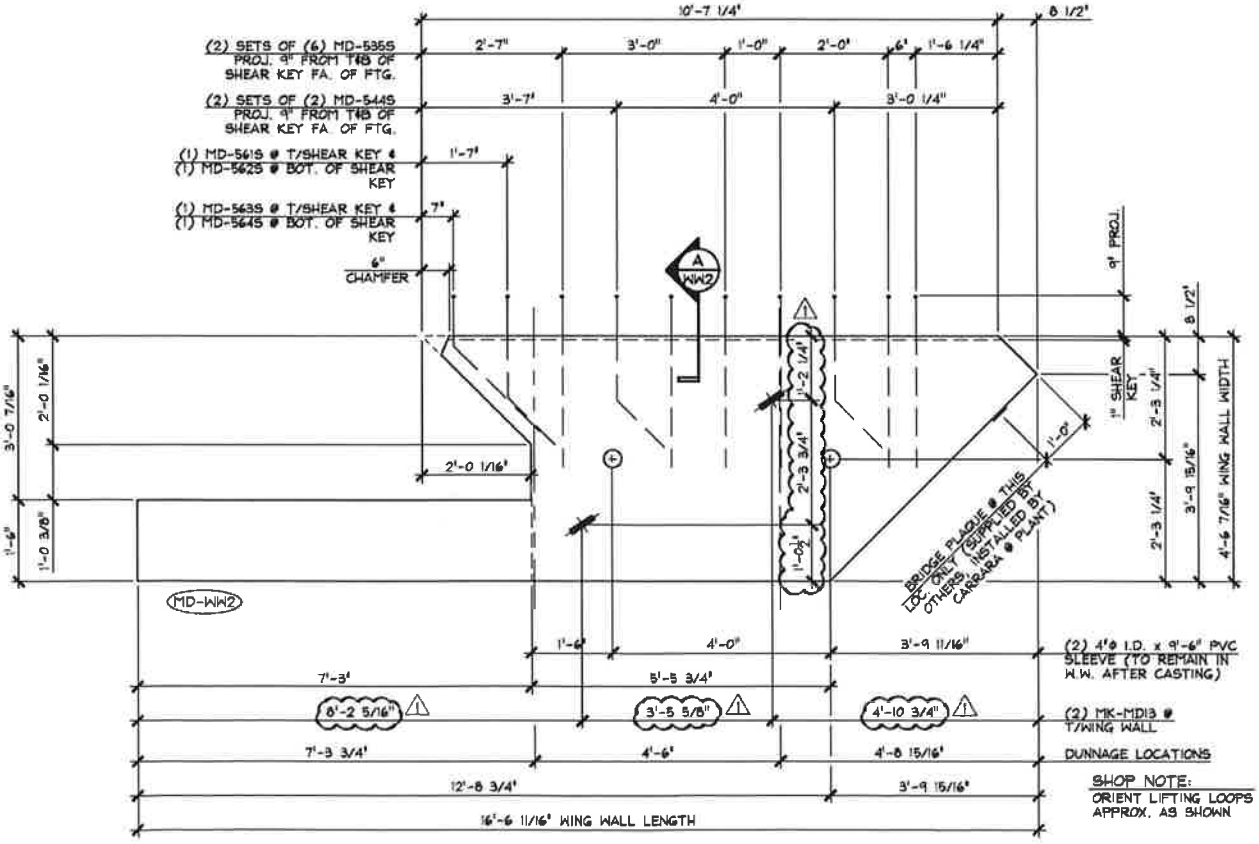
Area	size x in	y in	z in		x-bar in	y-bar in	z-bar in	Volume ft^3		x-bar*V	y-bar*V	z-bar*V
1	198.6875	114	54.4375	rect	99.34375	57	27.21875	713.558906		70887.62	40672.86	19422.18
2	45.9375	114	-45.9375	tri	183.375	57	15.3125	-69.60907	tri	-12764.56	-3967.717	-1065.889
3	8.5	114	-8.5	tri	195.8542	57	51.60417	-2.3832465	tri	-466.7688	-135.8451	-122.9855
4	87	33	-18		43.5	16.5	9	-29.90625		-1300.922	-493.4531	-269.1563
5	87	36	-18	tri	29	45	9	-16.3125	tri	-473.0625	-734.0625	-146.8125
6	62.9375	114	-36.4375		31.46875	57	36.21875	-151.29312		-4761.005	-8623.708	-5479.648
7	24.0625	114	-12.375		74.96875	57	24.1875	-19.644775		-1472.744	-1119.752	-475.158
8	24.0625	114	-24.0625	tri	70.95833	57	38.39583	-19.099087	tri	-1355.239	-1088.648	-733.3254
9	127.25	18	-1		126.5625	15	53.9375	-1.3255208		-167.7612	-19.88281	-71.49528
										48125.55	24489.79	11057.71
								403.985338	ft^3			
										119.127 in.	60.62049 in.	27.37157 in.
								60,598	lbs.	x-bar	y-bar	z-bar
								30.30	T	9.927248 ft.	5.051708 ft.	2.280964 ft.
										9' 11-1/8"	5' 5/8"	2' 3-3/8"

Waitsfield BRF 013 – 4(39)

J. P. Carrara & Sons, Inc.

Wingwall 1 & 2 Lift Loop Layout





WINGWALLS - ABUTMENT #2 (MK. WW3 & WW4)

$f'_c = 5000$ psi $f'_{ci} = 3500$ psi

SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY & WEIGHT

$W_T = 15.04\,T = 30.1\,K$ (300 PSF)

FOR STRIPPING:

THERE ARE (4) LIFT ANCHORS IN FACE OF WINGWALLS FOR STRIPPING & HORIZONTAL HANDLING. ASSUME 60° SLING ANGLE FROM HORIZONTAL

DESIGN LOAD / LIFT LOCATION $= 30.1\,K / (4)(0.866) = 8.7\,K$

FROM ATTACHED PRODUCT LITERATURE FROM DAYTON, "USE 8 TON x 10" SWIFT LIFT ANCHORS

SWL (4:1 S.F.) = 16K > 8.7K, O.K.

PLACEMENT OF LIFT ANCHORS ABOUT CENTER OF GRAVITY RESULTS IN 5'-11" +/- CANTILEVER AT 13/ PANEL WW3. CHECK STRESS IN PANEL:

$$M = \frac{300\,PLF \times 5.92^2}{2} \times 1.3 = 6{,}834\ LB\text{-}FT/FT$$

$$S = bh^2/6 = (12)(24)^2/6 = 1{,}152\,in^3$$

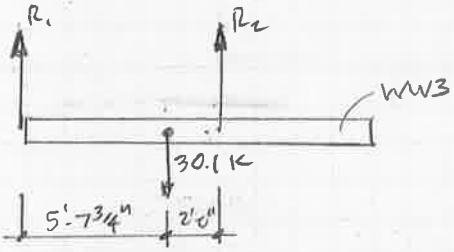
$$f = \frac{M}{S} = \frac{6{,}834 \times 12}{1{,}152} = 71\ psi < 6\sqrt{3500} = 355\ psi \quad \underline{O.K.}$$

(WINGWALLS - ABUTMENT #2)

FOR ROLLUP & ERECTION:

USE 2-LINE CRANE & PICK (2) LIFT ANCHORS IN TOPEDGE WITH OLD MAN AND (2) BOTTOM ANCHORS IN FACE FACE WITH WHIP LINE.

▷ CHECK BOTTOM ANCHORS FOR NEW TENSION LOAD IMPOSED:



$$R_2 = \left(\frac{5'\text{-}7\tfrac{3}{4}''}{7'\text{-}7\tfrac{3}{4}''}\right) \times 30.1^{K} = 22.2K$$

THERE ARE (2) LIFT ANCHORS AT THIS LOCATION, LOAD/LIFTER = $22.2/2 = 11.1K$

FROM PREVIOUS SHT., USE 8T x 10" S.L. ANCHOR

SWL (4:1 S.F.) = 16K > 11.1K, OK.

▷ CHECK TOP ANCHORS FOR SHEAR:

$$R_1 = \left(\frac{2'\text{-}0''}{7'\text{-}7\tfrac{3}{4}''}\right) 30.1^{K} = 7.9K$$

THERE ARE (2) LIFT ANCHORS IN THE TOP EDGE.

LOAD / LIFT ANCHOR = $7.9/2 = 3.9K$

FROM ATTACHED PRODUCT LITERATURE FOR DAYTON, "P.24"

USE (2) 20 TON X 19 3/4" SWIFT LIFT ANCHORS

BY INSPECTION, FROM "P26" IF 8TON x 13 3/8 S.L. SHEAR CAPACITY IS 6.7K, 20TON x 19 3/4" S.L. SHEAR CAPACITY IS EQUAL TO OR GREATER THAN THIS VALUE AND GREATER THAN 3.9K LOAD.

NTS.

(WINGWALLS - ABUTMENT #2)

CHECK TOP LIFT ANCHORS IN TENSION.

THERE ARE (2) LIFT ANCHORS IN TOP EDGE

LOAD / LIFT ANCHOR = $30.1^{K}/2 = 15.1K$

FROM ATTACHED PRODUCT LITERATURE FOR DAYTON, "P24"

USE 20TON x 19¾" LIFT ANCHORS

SWL (4:1 S.F.) = 40K > 15.1K , O.K. ✓

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 3

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

$$= (0.85)(2.67)(1.0)(5000)^{0.5}(24")(0" + 2(19\tfrac{3}{4}"))/1000$$

$$= 152.1K$$

FOR S.F. 4:1: SWL = $152.1/4 = 38.^{K} > 15.1K$ O.K. ✓

PLACE LIFT ANCHORS SYMMETRICALLY IN TOP EDGE OF PANEL ABOUT CENTER OF GRAVITY.

Rt 100, Bridge 177
Waitsfield, VT

J. P. Carrara & Sons, Inc.
Project 23480-015
4/11/16 Rev 0

Wingwall CG Calculation

Waitsfield, VT Bridge **WW3** Abutment #2

Area	size x in	y in	z in		x-bar in	y-bar in	z-bar in	Volume ft^3		x-bar*V	y-bar*V	z-bar*V
1	119.5	162	24	rect	59.75	81	12	268.875		16065.28	21778.88	3226.5
2	90	108	-24	rect	89.5	36	12	-67.5	tri	-6041.25	-2430	-810
3	119.5	0.75	-16.5	rect	59.75	161.625	12	-0.8557943		-51.13371	-138.3177	-10.26953
										9972.898	19210.56	2406.23
								200.519206	ft^3			
										49.73537 in.	95.80408 in.	12 in.
								30,078	lbs.	x-bar	y-bar	z-bar
								15.04	T	4.144614 ft.	7.983673 ft.	1 ft.
										4' 1-3/4"	7' 11-13/16"	1' 0"

Waitsfield, VT Bridge **WW4** Abutment #2

Area	size x in	y in	z in		x-bar in	y-bar in	z-bar in	Volume ft^3		x-bar*V	y-bar*V	z-bar*V
1	119.5	156	24	rect	59.75	78	12	258.916667		15470.27	20195.5	3107
2	90	96	-24	rect	30	32	12	-60	tri	-1800	-1920	-720
3	119.5	0.75	-16.5	rect	59.75	155.625	12	-0.8557943		-51.13371	-133.183	-10.26953
										13619.14	18142.32	2376.73
								198.060872	ft^3			
										68.76238 in.	91.5997 in.	12 in.
								29,709	lbs.	x-bar	y-bar	z-bar
								14.85	T	5.730198 ft.	7.633309 ft.	1 ft.
										5' 8-3/4"	7' 7-5/8"	1' 0"

PILE CAPS - ABUTMENT #2

LIFT DESIGN FOR "INSIDE PILE CAPS" ONLY, "MK. PC-2, PC-3, PC-4:

$f'_c = 5000$ psi $\quad f'_{ci} = 3500$ psi

SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY & WEIGHT:

MK. PC-2 19.32 T : USE (1) LIFT LOOP IN T/WALL

MK. PC-3 29.40 T : USE (2) LIFT LOOPS IN T/WALL

MK. PC-4 19.71 T : USE (1) LIFT LOOP IN T/WALL (CONTROLS)

THERE IS (1) LIFT LOOP PLACED AT CENTER OF GRAVITY IN PILE CAP 'MK. PC-4'

DESIGN LOAD / LIFT LOCATION = $19.71 \times 2 = 39.4$ KIPS

FROM ATTACHED PCI LICENSE LINE
USE (4) 0.600"ϕ 270 KSI STRAND LIFT LOOPS WITH 9'-9" EMBEDMENT.
& 2" RECESS & 8" PROJECTION.

FROM TABLE 5.2.3:
CAPACITY = $41 \times 1.1 = 45.1^K > 39.4^K$ O.K.

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 6

$$\phi P_c = \phi\, 2.67\, \lambda \sqrt{f'_{ci}}\,(x_1)(y_1)$$
$$= (0.85)(2.67)(1.0)(3500)^{0.5}(42")(28.5")/1000$$
$$= 160.7^K$$

FOR S.F. 4:1 : SWL = $160.7/4 = 40.2^K > 39.4^K$ O.K. ✓

NOTE:
INSTALL (2) LIFT LOOPS IN 'MK. PC3' AND (1) LIFT LOOP IN 'MK. PC2'

Rt 100, Bridge 177
Waitsfield, VT

J. P. Carrara & Sons, Inc.
Project 23480-015
4/7/16 Rev 0

Precast Pier Cap CG Calculation

Waitsfield, VT Bridge **PC-2** Abutment #2

Area	x	y	z		x-bar	y-bar	z-bar	Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3				
1	102.75	123	42	rect	51.375	61.5	21	307.179688		15781.36	18891.55	6450.773
2	0.75	123	14	rect	-0.375	61.5	21	0.74739583		-0.280273	45.96484	15.69531
3	0.75	123	-14	rect	103.125	61.5	21	-0.7473958		-77.0752	-45.96484	-15.69531
4	12	105	-1	cyl	24	52.5	21	-27.488936	cyl	-659.7345	-1443.169	-577.2677
5	4	18	-1	cyl	24	114	21	-0.5235988	cyl	-12.56637	-59.69026	-10.99557
6	12	105	-1	cyl	76.5	52.5	21	-27.488936	cyl	-2102.904	-1443.169	-577.2677
7	4	18	-1	cyl	76.5	114	21	-0.5235988	cyl	-40.05531	-59.69026	-10.99557
8	102.75	15.125	6	rect	51.375	115.4375	45	5.39615885		277.2277	622.9191	242.8271
9	102.75	6	6	tri	51.375	105.875	44	1.0703125	tri	54.9873	113.3193	47.09375
										13220.96	16622.07	5564.168
								257.62109	ft^3			
										51.31939 in.	64.52139 in.	21.59826 in.
								38,643	lbs.	x-bar	y-bar	z-bar
								19.32	T	4.276616 ft.	5.376782 ft.	1.799855 ft.
										4' 3-5/16"	5' 4-1/2"	1' 9-5/8"

Waitsfield, VT Bridge **PC-3** Abutment #2

Area	x	y	z		x-bar	y-bar	z-bar	Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3				
1	153	126	42	rect	76.5	63	21	468.5625		35845.03	29519.44	9839.813
2	0.75	126	-14	rect	-0.375	63	21	-0.765625		0.287109	-48.23438	-16.07813
3	0.75	126	-14	rect	153.375	63	21	-0.765625		-117.4277	-48.23438	-16.07813
4	12	108	-1	cyl	24	54	21	-28.274334	cyl	-678.584	-1526.814	-593.761
5	4	18	-1	cyl	24	117	21	-0.5235988	cyl	-12.56637	-61.26106	-10.99557
6	12	108	-1	cyl	76.5	54	21	-28.274334	cyl	-2162.987	-1526.814	-593.761
7	4	18	-1	cyl	76.5	117	21	-0.5235988	cyl	-40.05531	-61.26106	-10.99557
8	12	108	-1	cyl	129	54	21	-28.274334	cyl	-3647.389	-1526.814	-593.761
9	4	18	-1	cyl	129	117	21	-0.5235988	cyl	-67.54424	-61.26106	-10.99557
10	153	18.125	6	rect	76.5	116.9375	45	9.62890625		736.6113	1125.98	433.3008
11	153	6	6	tri	76.5	105.875	44	1.59375	tri	121.9219	168.7383	70.125
										29977.3	25953.46	8496.812
								391.860108	ft^3	76.5 in.	66.23145 in.	21.68328 in.
										x-bar	y-bar	z-bar
								58,779	lbs.	6.375 ft.	5.519287 ft.	1.80694 ft.
								29.39	T	6' 4-1/2"	5' 6-1/4"	1' 9-11/16"

Waitsfield, VT Bridge **PC-4** Abutment #2

Area	x	y	z		x-bar	y-bar	z-bar	Volume		x-bar*V	y-bar*V	z-bar*V
	in	in	in		in	in	in	ft^3				
1	102.75	125.0625	42	rect	51.375	62.53125	21	312.330566		16045.98	19530.42	6558.942
2	0.75	125.0625	-14	rect	-0.375	62.53125	21	-0.7599284		0.284973	-47.51927	-15.9585
3	0.75	125.0625	14	rect	103.125	62.53125	21	0.75992839		78.36761	47.51927	15.9585
4	12	105	-1	cyl	24	52.5	21	-27.488936	cyl	-659.7345	-1443.169	-577.2677
5	4	18	-1	cyl	24	116.0625	21	-0.5235988	cyl	-12.56637	-60.77018	-10.99557
6	12	105	-1	cyl	76.5	52.5	21	-27.488936	cyl	-2102.904	-1443.169	-577.2677
7	4	18	-1	cyl	76.5	116.0625	21	-0.5235988	cyl	-40.05531	-60.77018	-10.99557
8	102.75	15.125	6	rect	51.375	117.5	45	5.39615885		277.2277	634.0487	242.8271
9	102.75	6	6	tri	51.375	107.9375	44	1.0703125	tri	54.9873	115.5269	47.09375
										13641.59	17272.12	5672.336
								262.771969	ft^3	51.91418 in.	65.73044 in.	21.58654 in.
										x-bar	y-bar	z-bar
								39,416	lbs.	4.326181 ft.	5.477537 ft.	1.798878 ft.
								19.71	T	4' 3-15/16"	5' 5-3/4"	1' 9-9/16"

PILE CAPS - ABUTMENT #2

LIFT DESIGN FOR 'PC-1' PILE CAP

$f'_c = 5000$ psi $f'_{ci} = 3500$ psi

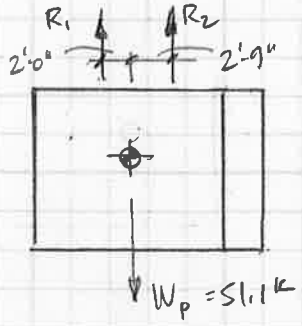
SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY & WEIGHT

MK. PC-1 25.56T = 51.1 KIPS (WEIGHT EXCLUDES PT SLEEVES/POCKETS)

THE CHEEKWALL PORTION OF THIS PANEL SHALL BE CAST-IN-PLACE IN THE FIELD, THEREFORE THE PILE CAP SHALL NOT BE 'OVERHEIGHT' AND MAY BE CAST, HANDLED, SHIPPED & INSTALLED WITH A VERTICAL ORIENTATION.

FOR VERTICAL LIFT:

THERE ARE (2) LIFT LOOPS PLACED IN TOP OF PILE CAP. DUE TO LOCATION OF CORRUGATED SLEEVES, LIFT LOOPS CANNOT BE PLACE SYMMETRICALLY ABOUT CG.



$R_1 = \left(\frac{2.75}{4.75}\right) 51.1 = 29.6^K$

$R_2 = \left(\frac{2}{4.75}\right) 51.1 = 21.5^K$

FROM ATTACHED PCI LITERATURE,
USE (4) 0.600" ϕ 270 KSI STRAND LIFT LOOPS WITH 9'-9" EMBEDMENT & 2" RECESS @ TOP & 8" PROJECTION

NTS.

TOPS

("PC-1" PILE CAP - ABUTMENT #2)

FROM TABLE 5.2.3

CAPACITY = 41 x 1.1 = 45.1K > 29.6K , OK ✓

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 6

$\phi P_c = \phi\, 2.67 \lambda \sqrt{f'_{ci}}\,(x_1)(y_1)$

$= (0.85)(2.67)(1.0)(3500)^{0.5}(42")(30")/1000$

$= 169.2K$

For S.F. 4:1: SWL = 169.2/4 = 42.3K > 29.6K , O.K. ✓

TOPS

P5

Rt 100, Bridge 177
Waitsfield, VT

J. P. Carrara & Sons, Inc.
Project 23480-015
4/22/2016 Rev 0

Precast Pile Cap CG Calculation

Waitsfield, VT Bridge **PC-1** Abutment #2

Area	size x in	y in	z in		x-bar in	y-bar in	z-bar in	Volume ft^3	x-bar*V	y-bar*V	z-bar*V
1	125.25	119.875	42	rect	62.625	59.9375	21	364.931966	22853.86	21873.11	7663.571
2	0.75	119.875	14	rect	-0.375	59.9375	21	0.72840712	-0.273153	43.658902	15.29655
3	12	101.875	-1	cyl	26.25	50.9375	21	-26.670813 cyl	-700.1088	-1358.545	-560.0871
4	4	18	-1	cyl	26.25	110.875	21	-0.5235988 cyl	-13.74447	-58.05401	-10.99557
5	12	101.875	-1	cyl	83.25	50.9375	21	-26.670813 cyl	-2220.345	-1358.545	-560.0871
6	4	18	-1	cyl	83.25	110.875	21	-0.5235988 cyl	-43.5896	-58.05401	-10.99557
7	23	119.875	23	tri	117.5833	59.9375	49.66667	18.3489222 tri	2157.527	1099.7885	911.3298
8	15	119.875	15	tri	130.25	59.9375	43	7.80436198 tri	1016.518	467.77395	335.5876
9	17	119.875	17	tri	128.9167	59.9375	53.66667	10.0242694 tri	1292.295	600.82965	537.9691
10	2	119.875	6	rect	124.25	59.9375	45	0.83246528	103.4338	49.895888	37.46094
11	100.3125	12	6	rect	50.15625	113.875	45	4.1796875	209.6375	475.96191	188.0859
12	100.3125	6	6	tri	50.15625	105.875	44	1.04492188 tri	52.40936	110.6311	45.97656
13	6	12	6	tri	102.3125	113.875	46	0.125 tri	12.78906	14.234375	5.75
14	1.5	125.25	-2	cyl	62.625	9	21	-1.0246992 cyl	-64.17179	-9.222292	-21.51868
15	1.5	125.25	-2	cyl	62.625	21	21	-1.0246992 cyl	-64.17179	-21.51868	-21.51868
16	1.5	125.25	-2	cyl	62.625	33	21	-1.0246992 cyl	-64.17179	-33.81507	-21.51868
17	1.5	125.25	-2	cyl	62.625	45	21	-1.0246992 cyl	-64.17179	-46.11146	-21.51868
18	1.5	125.25	-2	cyl	62.625	57	21	-1.0246992 cyl	-64.17179	-58.40785	-21.51868
19	1.5	125.25	-2	cyl	62.625	69	21	-1.0246992 cyl	-64.17179	-70.70424	-21.51868
20	1.5	125.25	-2	cyl	62.625	81	21	-1.0246992 cyl	-64.17179	-83.00063	-21.51868
21	1.5	125.25	-2	cyl	62.625	93	21	-1.0246992 cyl	-64.17179	-95.29702	-21.51868
22	1.5	125.25	-2	cyl	62.625	102	21	-1.0246992 cyl	-64.17179	-104.5193	-21.51868
23	1.5	125.25	-2	cyl	62.625	112	21	-1.0246992 cyl	-64.17179	-114.7663	-21.51868
24	5	6.75	-13.5	rect	122.75	9	21	-0.2636719	-32.36572	-2.373047	-5.537109
25	5	6.75	-13.5	rect	122.75	21	21	-0.2636719	-32.36572	-5.537109	-5.537109
26	5	6.75	-13.5	rect	122.75	33	21	-0.2636719	-32.36572	-8.701172	-5.537109
27	5	6.75	-13.5	rect	122.75	45	21	-0.2636719	-32.36572	-11.86523	-5.537109
28	5	6.75	-13.5	rect	122.75	57	21	-0.2636719	-32.36572	-15.0293	-5.537109
29	5	6.75	-13.5	rect	122.75	69	21	-0.2636719	-32.36572	-18.19336	-5.537109
30	5	6.75	-13.5	rect	122.75	81	21	-0.2636719	-32.36572	-21.35742	-5.537109
31	5	6.75	-13.5	rect	122.75	93	21	-0.2636719	-32.36572	-24.52148	-5.537109
32	5	6.75	-13.5	rect	122.75	102	21	-0.2636719	-32.36572	-26.89453	-5.537109
33	5	6.75	-13.5	rect	122.75	112	21	-0.2636719	-32.36572	-29.53125	-5.537109
									23755.04	21101.32	8328.305
								340.747468 ft^3			
									69.7145 in.	61.926565 in.	24.44128 in.
								51,112 lbs.	x-bar	y-bar	z-bar
								25.56 T	5.809542 ft.	5.1605471 ft.	2.036773 ft.
									5' 9-11/16"	5' 1-15/16"	2' 7/16"

PILE CAPS - ABUTMENT #2

LIFT DESIGN FOR 'PC-5' PILE CAP

$f'_c = 5000$ psi $f'_{ci} = 4000$

SEE ATTACHED CALCULATION FOR CENTER OF GRAVITY & WEIGHT

MK PC-5 22.65T = 45.3 KIPS (WEIGHT EXCLUDES PT SLEEVES/PONCHES)

AS FOR 'PC-1', THE CHEEKWALL PORTION OF THIS PANEL SHALL BE CAST-IN-PLACE IN THE FIELD, THEREFORE THE PILE CAP SHALL NOT BE 'OVERHEIGHT' AND MAY BE CAST, HANDLED, SHIPPED & INSTALLED WITH A VERTICAL ORIENTATION.

FOR VERTICAL LIFT:

THERE ARE (2) LIFT LOOPS PLACED ABOVE CENTER OF GRAVITY IN PILE CAP.

DESIGN LOAD/LIFT LOCATION = $45.3/2 = 22.7^K$

FROM ATTACHED PCI LITERATURE:

USE (3) 0.600" ϕ 270 KSI STRAND LIFT LOOPS WITH 9'-9" EMBEDMENT & 2" RECESS & 8" PROJECTION

FROM TABLE 5.2.3:

CAPACITY = $41 \times 1.1 = 45.1^K > 22.7^K$, OK.

TOPS

("PC-5" PILE CAP - ABUTMENT #2)

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 6

$$\phi P_c = \phi 2.67 \lambda \sqrt{f'_{c_i}} (x_i)(y_i)$$

$$= (0.85)(2.67)(1.0)(4000)^{0.5}(42")(31")/1000$$

$$= 186.9^k$$

FOR S.F. 4:1 : $SWL = 186.9/4 = 46.7^k > 2 \times 22.7 = 45.4^k$, OK

CAPACITY IS CONSERVATIVE AS EDGE DISTANCE IS BASED ON NARROWEST DIMENSION BETWEEN 2'-0" DIAMETER CORRUGATED SLEEVES.

Rt 100, Bridge 177
Waitsfield, VT

J. P. Carrara & Sons, Inc.
Project 23480-015
4/22/2016 Rev 0

Precast Pile Cap CG Calculation

Waitsfield, VT Bridge **PC-5** Abutment #2

Area	size x in	y in	z in		x-bar in	y-bar in	z-bar in	Volume ft^3	x-bar*V	y-bar*V	z-bar*V
1	110.25	125.0625	42	rect	55.125	62.53125	21	335.128418	18473.95	20955.999	7037.697
2	0.75	125.0625	14	rect	110.625	62.53125	21	0.75992839	84.06708	47.519272	15.9585
3	12	107.0625	-1	cyl	29	53.53125	21	-28.028897 cyl	-812.838	-1500.422	-588.6068
4	4	18	-1	cyl	29	116.0625	21	-0.5235988 cyl	-15.18436	-60.77018	-10.99557
5	12	107.0625	-1	cyl	84	53.53125	21	-28.028897 cyl	-2354.427	-1500.422	-588.6068
6	4	18	-1	cyl	84	116.0625	21	-0.5235988 cyl	-43.9823	-60.77018	-10.99557
7	23	125.0625	23	tri	24.33333	62.53125	49.66667	19.1429579 tri	465.812	1197.0331	950.7669
8	15	125.0625	15	tri	37	62.53125	53	8.14208984 tri	301.2573	509.13506	431.5308
9	9	125.0625	-9	tri	3	62.53125	39	-2.9311523 tri	-8.793457	-183.2886	-114.3149
10	17	125.0625	6	rect	40.5	62.53125	45	7.38216146	298.9775	461.61578	332.1973
11	6	125.0625	-6	tri	47	62.53125	44	-1.3027344 tri	-61.22852	-81.46161	-57.32031
12	61.3125	17.1875	6	rect	79.59375	116.4688	45	3.65905762	291.2381	426.16587	164.6576
13	61.3125	6	6	tri	79.59375	105.875	44	0.63867188 tri	50.83429	67.619385	28.10156
14	6	12	6	tri	47	116.4688	44	0.125 tri	5.875	14.558594	5.5
15	1.5	110.25	-2	cyl	55.125	9	21	-0.9019807 cyl	-49.72169	-8.117826	-18.94159
16	1.5	110.25	-2	cyl	55.125	21	21	-0.9019807 cyl	-49.72169	-18.94159	-18.94159
17	1.5	110.25	-2	cyl	55.125	33	21	-0.9019807 cyl	-49.72169	-29.76536	-18.94159
18	1.5	110.25	-2	cyl	55.125	45	21	-0.9019807 cyl	-49.72169	-40.58913	-18.94159
19	1.5	110.25	-2	cyl	55.125	57	21	-0.9019807 cyl	-49.72169	-51.4129	-18.94159
20	1.5	110.25	-2	cyl	55.125	69	21	-0.9019807 cyl	-49.72169	-62.23667	-18.94159
21	1.5	110.25	-2	cyl	55.125	81	21	-0.9019807 cyl	-49.72169	-73.06044	-18.94159
22	1.5	110.25	-2	cyl	55.125	93	21	-0.9019807 cyl	-49.72169	-83.88421	-18.94159
23	1.5	110.25	-2	cyl	55.125	102	21	-0.9019807 cyl	-49.72169	-92.00203	-18.94159
24	1.5	110.25	-2	cyl	55.125	112	21	-0.9019807 cyl	-49.72169	-101.0218	-18.94159
25	5	6.75	-13.5	rect	2.5	9	21	-0.2636719	-0.65918	-2.373047	-5.537109
26	5	6.75	-13.5	rect	2.5	21	21	-0.2636719	-0.65918	-5.537109	-5.537109
27	5	6.75	-13.5	rect	2.5	33	21	-0.2636719	-0.65918	-8.701172	-5.537109
28	5	6.75	-13.5	rect	2.5	45	21	-0.2636719	-0.65918	-11.86523	-5.537109
29	5	6.75	-13.5	rect	2.5	57	21	-0.2636719	-0.65918	-15.0293	-5.537109
30	5	6.75	-13.5	rect	2.5	69	21	-0.2636719	-0.65918	-18.19336	-5.537109
31	5	6.75	-13.5	rect	2.5	81	21	-0.2636719	-0.65918	-21.35742	-5.537109
32	5	6.75	-13.5	rect	2.5	93	21	-0.2636719	-0.65918	-24.52148	-5.537109
33	5	6.75	-13.5	rect	2.5	102	21	-0.2636719	-0.65918	-26.89453	-5.537109
34	5	6.75	-13.5	rect	2.5	112	21	-0.2636719	-0.65918	-29.53125	-5.537109
									16171.75	19567.476	7350.782
								301.982881 ft^3			
									53.55189 in.	64.796639 in.	24.34172 in.
								45,297 lbs.	x-bar	y-bar	z-bar
								22.65 T	4.462657 ft.	5.3997199 ft.	2.028477 ft.
									4' 5-9/16"	5' 4-13/16"	2' 5/16"

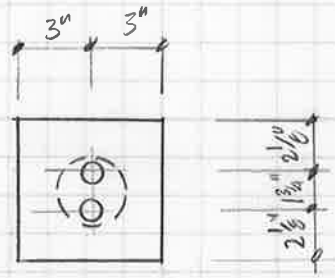
▸ PILE CAP - ABUTMENT #2

POST-TENSIONING

- CHECK BEARING STRENGTH OF CONCRETE AT LOCATION OF POST TENSIONING RECESS POCKETS

POST TENSIONING PLATE

PL 1"x6"x6" w/(2) 3/4"ϕ HOLES



BEARING AREA A_1 (PLATE MINUS 3" SLEEVE)

$A_1 = 6" \times 6" - \pi(1.5)^2$

$A_1 = 28.93 in^2$

PT FORCE

$P = 32^k \times 2 \times 1000 = 64{,}000^{lb}$

BEARING STRESS

$f_{brg} = \frac{64{,}000}{28.93} = 2{,}212$ psi

DETERMINE DESIGN BEARING STRENGTH

FROM PCI MNL 120, 7TH ED. SECTION 5.6.1, BEARING ON PLAIN CONCRETE

$\phi V_n = \phi C_r (0.85) f'_c A_1$

$= (0.65)(1.0)(0.85)(5000)(28.93)$

$= 79{,}900$ LB.

$\phi = 0.65$
$C_r = 1.0$
$f'_c = 5000$ psi
$A_1 = 28.93 in^2$

$\phi V_n = 79{,}900^{lb} > 64{,}000^{lb}$. OK ✓

ADDITIONAL REINFORCEMENT AT RECESS POCKETS FOR PT PLATE BEARING IS NOT REQUIRED.

NTS.

TOPS

P-10

5

5.6 Bearing

5.6.1 Bearing on Plain Concrete

Plain concrete bearing may be used in situations where the bearing is uniform and the bearing stresses are low, for example, as is typical in hollow-core and solid slabs. In other situations, and in thin-stemmed components where the bearing area is smaller than 20 in.^{2}, a minimum reinforcement equal to $N_u/\phi f_y$ (but not less than (1) #3) is recommended. It is recommended that, when properly designed bearing pads are used, N_u be taken as 0.2 times the sustained load portion of V_u unless otherwise calculated.

The design bearing strength of plain concrete may be calculated as:

$$\phi V_n = \phi C_r \left(0.85 f_c' A_1\right)\sqrt{\frac{A_2}{A_1}} \le 1.1 f_c' A_1 \qquad \text{(Eq. 5-54)}$$

where:

ϕV_n = design bearing strength, kip

ϕ = 0.65

$$C_r = \left(\frac{sw}{200}\right)^{\frac{N_u}{V_u}}$$

= 1.0 when reinforcement is provided in direction of N_u, in accordance with Section 5.6.2 or when N_u is zero. The product sw should not be taken greater than 9.0 in.^{2}

See Fig. 5.6.1 for additional definitions.

5.6.2 Reinforced Concrete Bearing

If the applied load V_u exceeds the design bearing strength ϕV_n as calculated by Eq. 5-54, reinforcement is required in the bearing area. This reinforcement can be designed by shear-friction as discussed in Section 5.3.6. Referring to Fig. 5.6.2, the reinforcement $A_{vf} + A_n$ nominally parallel to the direction of the axial load N_u is determined by Eq. 5-32 through 5-34 with $A_{cr} = bh$ (that is, $\theta \approx 0$).[11] This reinforcement must be appropriately developed by hooks or by welding to an anchor bar, bearing plate, or angle.

Vertical reinforcement across potential horizontal cracks can be calculated by:

$$A_{sh} = \frac{\left(A_{vf} + A_n\right) f_y}{\mu_e f_{ys}} \qquad \text{(Eq. 5-55)}$$

Stirrups or welded-wire reinforcement used for diagonal tension reinforcement can be considered to act as A_{sh} reinforcement.

When components are subjected to bearing force in excess of $1.1 f_c' A_1$, confinement reinforcement in all directions may be necessary.

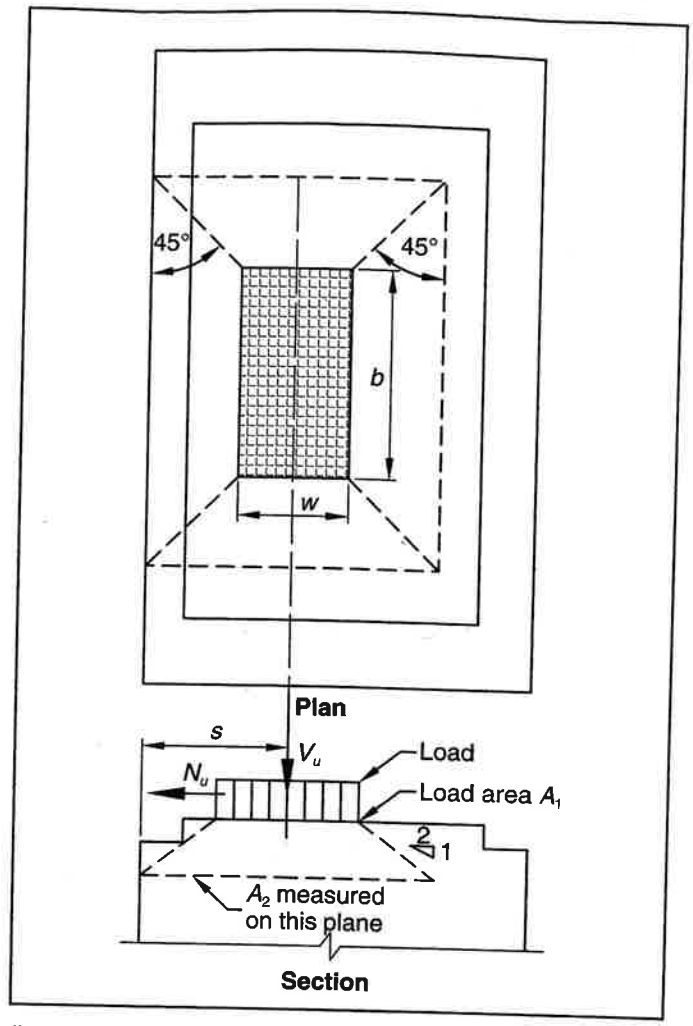
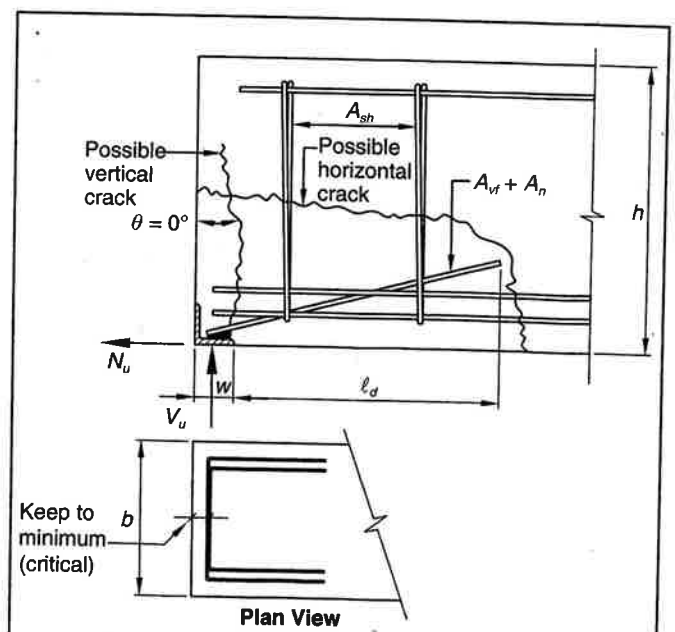


Fig. 5.6.1 Bearing on plain concrete.



Alternative scheme for $A_{vf} + A_n$ anchorage at bearing not recommended for thin-stemmed members such as double-tees.

Fig. 5.6.2 Reinforced concrete bearing.

P52 Swift Lift® Anchor

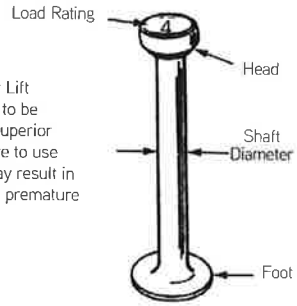
The P52 Swift Lift Anchor is hot forged from carbon steel. The formed head provides spherical seating that the Lifting Eye engages, while a disc-shaped foot is embedded in the concrete.

Due to its being a forged part, the Swift Lift Anchor does not depend on welds or thread engagement to develop its safe working load. Forging provides maximum safety with its advantageous material structure. This allows the anchor to easily meet the OSHA requirement of a 4 to 1 factor of safety.

In addition to the carbon steel anchors, Type 304 or 316 Stainless Steel Swift Lift Anchors are available on special order. Use stainless steel anchors when maximum protection against corrosion is required.

For safety, refer to the P52 Swift Lift Anchor Selection Table to determine the actual safe working load of an individual anchor. The MAXIMUM safe working load is clearly visible on the head of the anchor for easy recognition of the appropriate hardware and accessories for-use with each Swift Lift Anchor.

Caution: The Swift Lift Anchor is designed to be used with Dayton Superior components. Failure to use this combination may result in concrete spalling or premature failure.



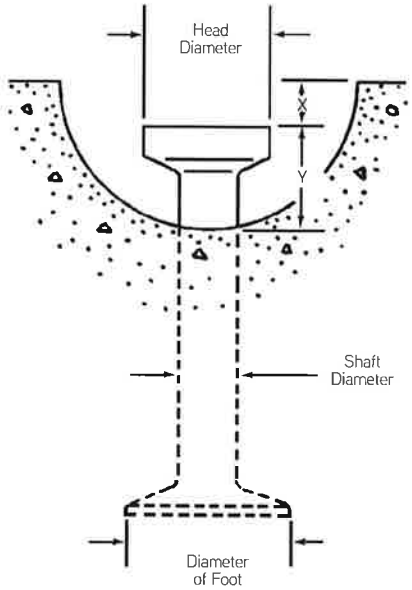
To Order:

Specify: (1) quantity, (2) name, (3) system size, (4) length

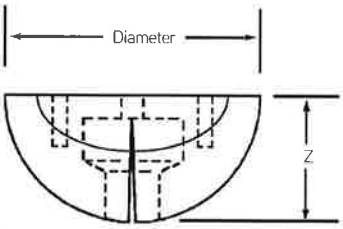
Example:

200, P52 Swift Lift Anchors, 4 ton, 9-1/2" long

P52 Swift Lift Anchor and Recess Plug Dimensions



P52 Swift Lift Anchor



Swift Lift Round Recess Plug

Swift Lift Round Recess Plug Dimensions

Swift Lift Anchor	Diameter of Recess Plug	Dimension Z
1 Ton	2-7/16"	1-3/16"
2 Tons	3-5/16"	1-7/16"
4 Tons	4"	1-13/16"
8 Tons	5"	2-5/16"
20 Tons	6-3/8"	3-1/8"

Note: The diameter of the narrow recess plug is the same as the diameter of the round recess plug.

P52 Swift Lift Anchor Dimensions

Swift Lift Anchor	Dimension X	Dimension Y	Shaft Diameter	Foot Diameter	Head Diameter
1 Ton	5/16"	7/8"	3/8"	1"	11/16"
2 Tons	7/16"	1-1/16"	9/16"	1-3/8"	1-1/32"
4 Tons	9/16"	1-5/16"	3/4"	1-7/8"	1-11/32"
8 Tons	9/16"	1-5/8"	1-3/32"	2-5/8"	1-7/8"
20 Tons	9/16"	2-5/8"	1-1/2"	3-3/4"	2-3/4"

P52 Swift Lift® Anchor Tensile and Shear Capacity

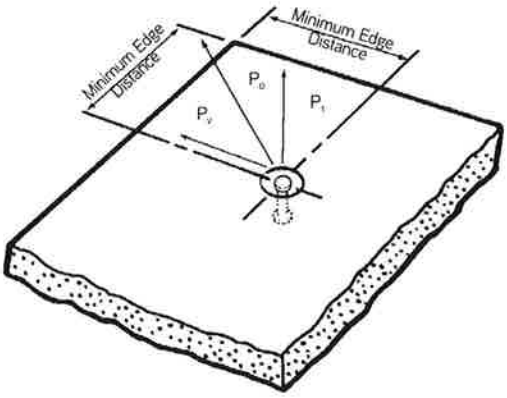
When anchors are used in the face of thin concrete elements

The following table lists the P52 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 P52 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.



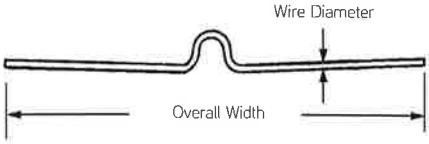
Swift Lift Anchor Ton x Length	Safe Working Load	Minimum Concrete Strength	Minimum Edge Distance
1 ton x 2-5/8"	1,700 lbs.	3,500 psi	8"
1 ton x 3-3/8"	2,000 lbs.	2,200 psi	10"
1 ton x 4-3/4"	2,000 lbs.	1,600 psi	10"
1 ton x 8"	2,000 lbs.	1,600 psi	10"
1 ton x 9-1/2"	2,000 lbs.	1,600 psi	10"
2 ton x 2-3/4"	2,100 lbs.	3,500 psi	8"
2 ton x 3-3/8"	2,900 lbs.	3,500 psi	10"
2 ton x 4-3/4"	4,000 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 5-1/4"	6,850 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.

P59 Swift Lift® Shear Bar
P59 Swift Lift® Smooth Wire Shear Bar

Dayton Superior Swift Lift Shear Bars (P59) are utilized when edge lifting precast elements. The shear bar is secured tightly to the recess plug and at the time of lift helps to transfer the shear load deeper into the concrete. The P59 Smooth Wire Shear Bar is designed to snap into the built-in clips on the P54 recess plug. The standard shear bar is fabricated from rebar and must be securely wired tightly to the Swift Lift recess plugs.

P59 Swift Lift Shear Bar
1-Ton and 8-Ton



P59 Swift Lift Smooth Wire Shear Bar
2-Ton and 4-Ton

To Order:
Specify: (1) quantity, (2) name, (3) system size.

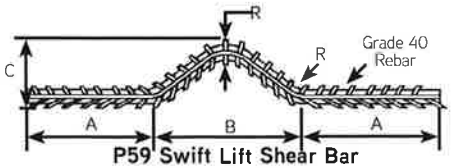
Example:
200, P59 Swift Lift Shear Bars, 4-Ton.

Anchor Safe Working Load	Shear Bar Type	Overall Width	Minimum Anchor Length	Wall Thickness	Minimum Corner Distance	Safe Working Load
1	Rebar	19"	4-3/4"	5"	15"	1,750 lbs.
1	Rebar	19"	4-3/4"	5-1/2"	16"	2,000 lbs.
1	Wire	19"	4-3/4"	6"	17"	2,000 lbs.
2	Wire	24"	6-3/4"	5"	15"	1,750 lbs.
2	Wire	24"	6-3/4"	5-1/2"	16"	2,200 lbs.
2	Wire	24"	6-3/4"	6"	17"	2,600 lbs.
2	Wire	24"	6-3/4"	7"	19"	2,700 lbs.
2	Wire	24"	6-3/4"	8"	21"	2,800 lbs.
4	Wire	30"	9-1/2"	5-1/2"	16"	2,100 lbs.
4	Wire	30"	9-1/2"	6"	17"	2,200 lbs.
4	Wire	30"	9-1/2"	7"	19"	2,600 lbs.
4	Wire	30"	9-1/2"	8"	21"	2,800 lbs.
8	Rebar	34"	13-3/8"	8"	19"	5,600 lbs.
8	Rebar	34"	13-3/8"	9"	21"	6,700 lbs.

Safe Working Load provides a factor of safety of approximately 4 to 1 in 4,500 psi normal weight concrete.

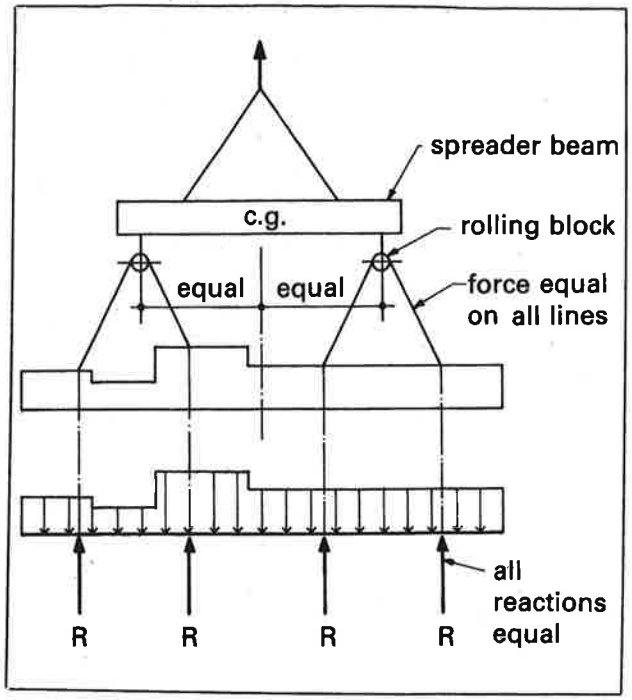
P59 Shear Bar Dimensions

Anchor Rated Load (Tons)	A	B	C	R	Rebar Size
1	7" 178 mm	5" 127 mm	2½" 64 mm	1" 25 mm	#4 #10M
2	9" 229 mm	7" 178 mm	3" 76 mm	1 7/16" 37 mm	#4 #10M
4	10" 254 mm	9" 229 mm	3¾" 95 mm	1¾" 44 mm	#4 #10M
8	12" 305 mm	10" 254 mm	3¾" 95 mm	2¼" 57 mm	#6 #20M



P59 Swift Lift Shear Bar
1-Ton and 8-Ton

Fig. 5.2.10 Arrangement for equalizing lifting loads



lines equal. The member can then be analyzed as a beam with varying load supported by equal reactions.

The force in inclined lift lines can be determined from Fig. 5.2.7.

5.2.8 Handling devices

The most common lifting devices are prestressing strand or cable loops projecting from the concrete, threaded inserts, or special proprietary devices.

Since lifting devices are subject to dynamic loads, ductility of the material is part of the design requirement. Deformed reinforcing bars should not be used since the deformations result in stress concentrations from the shackle pin. Also, reinforcing bars are often hard-grade or re-rolled rail steel with little ductility and low impact strength at cold temperatures. Smooth bars of a known steel grade may be used if adequate embedment or mechanical anchorage is provided. The diameter must be such that localized failure will not occur by bearing on the shackle pin.

Prestressing strand is often used for lifting loops. The variables involved make it almost impossible to calculate a capacity which can be used for all situations. Generally, producers will establish standard criteria for use in handling the standard products manufactured by that plant. Table 5.2.3 is an example which has been used successfully.

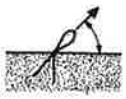
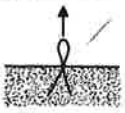
Reduced capacities for shorter embedment lengths may be suitable. In shallow products, providing a 90° bend can reduce the required embedment length significantly. Lightly rusted strand has better bond than bright strand.

The diameter of the bend of the loop should be at least 4 in. For smaller diameters, the loop capacities in Table 5.2.3 should be reduced to:

1 in. dia. — 70 %
2 in. dia. — 85 %
3 in. dia. — 90 %

The angle of incline of lifting has little effect on the strand lifting loop capacity if the angle from the horizontal is more than about 20°. Typical handling methods are usually such that this angle is no less than 60°.

Table 5.2.3 Capacity of ½ in. diameter, 270 ksi strands used as lifting loops

Lifting angle	Embedment length (in.)	Single loop (kips)	Double loop (kips)	Triple loop (kips)
45 degrees 	16	5	8.5	11.5
	22	8	13	17.5
	28	10	18	23
	34	11	23	29
Vertical 	16	7.5	12.5	16.5
	22	11.5	19	24.5
	28	15.5	25.5	33
	34	16	32.5	41

1. These values are limited by slippage rather than strand strength, with a factor of safety of 4. For other strand diameters, multiply table values by 0.75 for ⅜ in. diameter, 0.85 for 7/16 in. diameter, and 1.1 for 0.6 in. diameter.
2. Minimum f'_c = 3000 psi.
3. Multiple strand loops must be fabricated to ensure equal force on each strand.

ability of headed stud design. The design methods used here should be considered an interim step toward a final headed stud design procedure. It is recommended that this procedure be limited to headed studs with an embedment not greater than 8 in.

An important factor in the performance of headed studs when controlled by concrete capacity is the confinement of the failure area with reinforcement. In shear, design capacity is increased with such reinforcement. In tension, ductility can be provided. It is recommended that reinforcement be placed to cross failure planes around headed stud anchorages.

Welded headed studs are designed to resist direct tension, shear or a combination of the two. The design equations given below are applicable to studs which are welded to steel plates or other structural members, and embedded in unconfined concrete.

Where feasible, headed stud connections should be designed and detailed such that the connection failure is precipitated by failure (typically defined as yielding) of the stud material rather than failure of the surrounding concrete. The in-place strength should be taken as the smaller of the values based on concrete and steel.

6.5.2.1 Tension

The design tensile strength governed by concrete failure is [9]:

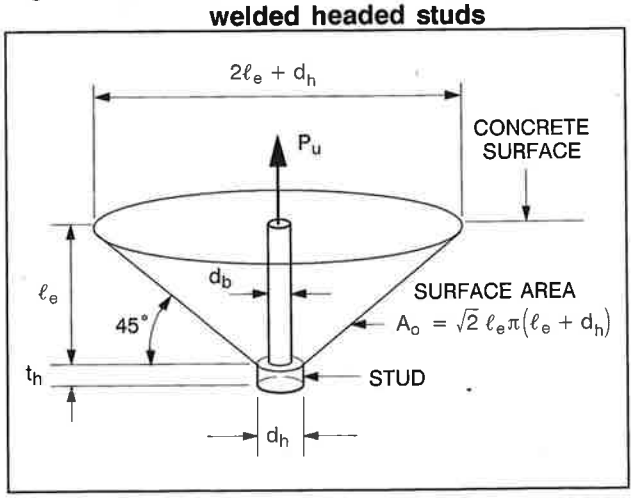
$$\phi P_c = \phi A_o(2.8\lambda\sqrt{f'_c}) \qquad \text{(Eq. 6.5.2)}$$

where:

ϕ = 0.85

A_o = area of the assumed failure surface which, for a single stud not located near a free edge, is taken to be that of a 45° truncated cone as shown in Figure 6.5.3.

Figure 6.5.3 Shear cone development for welded headed studs



Using the 45° cone area and ϕ = 0.85, Eq. 6.5.2 may be written as:

$$\phi P_c = 10.7\ell_e(\ell_e + d_h)\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.3)}$$

Note: The stud length is often used in place of the actual embedment length, ℓ_e, which is equal to the stud length minus the thickness of the head. This simplification is generally acceptable except in short studs. In short studs (length ≤ 4 in.), the use of actual embedment length is recommended. It should also be noted that short stud capacities are also sensitive to fabrication tolerances. Thus, use of a larger overall factor of safety may be appropriate for short studs. See Sect. 6.3.

For a stud located closer to a free edge than the embedment length, ℓ_e, the design tensile strength given by Eq. 6.5.3, should be reduced by multiplying it by C_{es}:

$$C_{es} = \frac{d_e}{\ell_e} \le 1.0 \qquad \text{(Eq. 6.5.4)}$$

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. 6.5.4 should be applied twice, once for each edge distance. Figure 6.15.6 lists values based on Eqs. 6.5.3 and 6.5.4.

For a group of studs, the concrete failure surface may be along a truncated pyramid rather than separate shear cones, as shown in Figure 6.5.4.

For this case, the design tensile strength is:

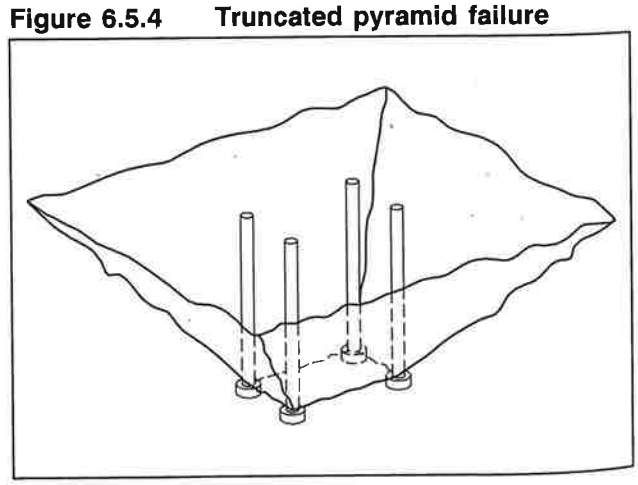
$$\phi P_c = \phi\lambda\left(\frac{2}{3}\right)\sqrt{f'_c}\,(2.8A_{slope} + 4A_{flat}) \qquad \text{(Eq. 6.5.5)}$$

where:

A_{slope} = sum of the areas of the sloping sides

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

Figure 6.5.4 Truncated pyramid failure



where z is lesser of x and y

$h_{min} = [8 + 2(8)]/2 = 12$ in. (Note: Same result can be read from Figure 6.15.7B)

Since h (= 10 in.) < h_{min}, failure surface is likely to penetrate through the slab.

3. Calculate tension strength based on concrete for the studs as a group:

The applicable equation is given in Figure 6.5.6 (Case 4, $h < h_{min}$):

$$\phi P_c = \phi 2.67\lambda\sqrt{f'_c}\,[(x + \ell_e + d_{e1})(y + \ell_e + d_{e3}) - A_R]$$

where $A_R = (x + 2\ell_e - 2h)(y + 2\ell_e - 2h)$

substituting given values:

$x + \ell_e + d_{e1} = 16 + 8 + 4 = 28$ in.

$y + \ell_e + d_{e3} = 8 + 8 + 6 = 22$ in.

$A_R = [16 + 2(8) - 2(10)][8 + 2(8) - 2(10)]$
$= 12(4) = 48$ in^2

$$\phi P_c = 0.85(2.67)1.0\sqrt{4000}\frac{28(22) - 48}{1000}$$
$$= 88.4 - 6.9 = 81.5 \text{ kips}$$

Alternatively, from Figure 6.15.7A (Case 4) and 6.15.7C:

For $\ell_e = 8$ in., $x = 16$ in., $x_1 = 20$ in.,
$y = 8$ in.
$y_1 = 14$ in., $h - \ell_e = 2$ in.

$$\phi P_{c1} = 99\sqrt{\frac{4000}{5000}} = 88.5 \text{ kips}$$

$$\phi P_{c2} = 7.7\sqrt{\frac{4000}{5000}} = 6.9 \text{ kips}$$

Therefore, $\phi P_c = \phi P_{c1} - \phi P_{c2} = 88.5 - 6.9$
$= 81.6$ kips

Note: Pull-out strength based on individual studs should also be checked.

4. Check capacity based on steel failure:

From Figure 6.15.6, for ¾ in. diam. stud:

$$\phi P_y = 19.9 \text{ kips/stud}$$

Total $\phi P_y = 4(19.9) = 79.6$ kips < 81.5

Tension strength of the group is 79.6 kips

6.5.2.2 Shear

The design shear strength governed by concrete failure is based on the concepts and results given in Refs. 9 and 10. Some modifications were made to improve correlation with test data obtained under a PCI Research Fellowship, the results of which are reported in Ref. 23.

The design shear strength limited by concrete, ϕV_c, is calculated as follows:

$$\phi V_c = \phi V'_c C_w C_t C_c \qquad \text{(Eq. 6.5.7)}$$

where:

$\phi V'_c$ = design shear strength of a single stud in the back row (Eq. 6.5.8).

C_w, C_t, C_c = adjustment factors for group width, member thickness and vicinity to member corner effects, respectively (see Figure 6.5.7). These quantities are given by Eqs. 6.5.9 through 6.5.11.

$$\phi V'_c = \phi 12.5 d_e^{1.5}\lambda\sqrt{f'_c} \qquad \text{(Eq. 6.5.8)}$$

where:

ϕ = 0.85

d_e = distance from free edge of concrete to back row of studs in direction of load

$$C_w = 1 + \frac{b}{3.5d_e} \le n_s \qquad \text{(Eq. 6.5.9)}$$

where:

b = center-to-center distance between the outer-most studs in the back row of the group

n_s = number of studs in the back row

$$C_t = \frac{h}{1.3d_e} \le 1.0 \qquad \text{(Eq. 6.5.10)}$$

where:

h = thickness of the concrete member

$$C_c = 0.4 + 0.7\frac{d_c}{d_e} \le 1.0 \qquad \text{(Eq. 6.5.11)}$$

where:

d_c = the distance, measured perpendicular to the direction of the load, from free edge of concrete to the centerline of the nearest stud.

The design shear strength as governed by steel is:

$$\phi V_y = \phi 0.9 f_y A_b n = 40{,}500 A_b n \qquad \text{(Eq. 6.5.12)}$$

where $\phi = 0.9$, or, in terms of stud diameter, d_b:

$$\phi V_y = (31{,}800\, d_b^2)n \qquad \text{(Eq. 6.5.12a)}$$

Figure 6.15.8 gives values for $\phi V'_c$ (Eq. 6.5.8) and ϕV_y (Eq. 6.5.12a).

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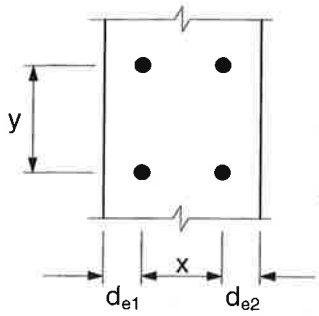
Figure 6.15.7A (continued) Design tensile strength for $h \geq h_{min}$, ϕP_{c1}—Case 3



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

Case 3: Free edges on two opposite sides

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

$$\phi = 0.85$$

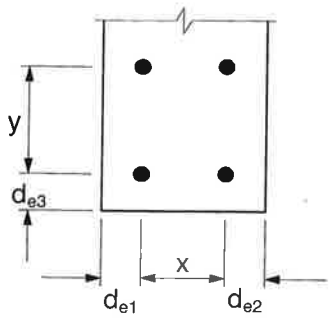
where: x_1 and y_1 are the dimensions of the flat bottom of the part of the truncated pyramid.

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

Figure 6.15.7A (continued) Design tensile strength for $h \geq h_{min}$, ϕP_{c1}—Case 5



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

Case 5: Free edges on three sides

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

$\phi = 0.85$

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

For Case 5: $x_1 = x + d_{e1} + d_{e2}$ $\quad y_1 = y + d_{e3}$

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	1	2	3	4	5	6	7	8	9	9	11	11	13	13	15
	2	1	3	5	7	8	9	11	13	15	16	17	19	21	23	24
	4	2	5	7	9	11	13	16	18	20	23	25	27	29	31	33
	6	3	6	9	11	15	17	20	23	26	29	32	35	37	41	43
	8	3	7	11	14	17	21	25	28	32	35	39	42	46	49	53
	10	4	9	13	17	21	25	29	33	37	42	46	50	54	59	63
	12	5	9	15	19	24	29	33	39	43	48	53	58	63	67	72
	14	5	11	17	22	27	33	38	43	49	55	60	65	71	76	82
	16	6	12	18	25	31	37	43	49	55	61	67	73	79	85	91
4	0	1	3	4	5	7	8	9	10	11	13	14	15	17	18	19
	2	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	4	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	6	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	8	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	10	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	12	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
	14	6	11	17	23	29	35	41	46	52	58	63	69	75	81	87
	16	7	13	19	25	32	39	45	51	58	64	71	77	83	90	96
6	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
8	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	30	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

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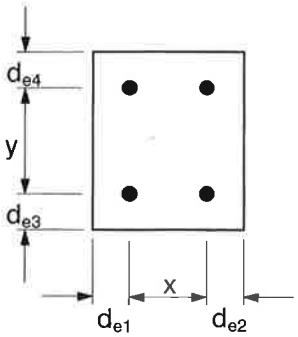
Figure 6.15.7A (continued) Design tensile strength for $h \geq h_{min}$, ϕP_{c1}—Case 6



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

Case 6: Free edges on four adjacent sides

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

$\phi = 0.85$

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

For Case 6: $x_1 = x + d_{e1} + d_{e2}$ $\quad y_1 = y + d_{e3} + d_{e4}$

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	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18	19
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	8	3	5	8	10	13	15	18	21	23	25	29	31	33	36	39
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	16	5	10	15	21	25	31	36	41	46	51	35	61	67	72	77
4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18	19
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	8	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	16	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18	19
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	8	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	16	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	1	1	2	3	3	4	5	5	6	7	7	8	9	9	9
	4	1	3	4	5	7	8	9	10	11	13	14	15	17	18	19
	6	2	4	6	8	9	11	13	15	17	19	21	23	25	27	29
	8	3	5	8	10	13	15	18	21	23	25	28	31	33	36	39
	10	3	7	9	13	16	19	23	25	29	32	35	39	42	45	48
	12	4	8	11	15	19	23	27	31	35	39	42	46	50	54	58
	14	5	9	13	18	23	27	31	36	41	45	49	54	59	63	67
	16	5	10	15	21	25	31	36	41	46	51	57	61	67	72	77