J. P. CARRARA & SONS, INC | ERNEST J. BROD 2-13-15 | 1

CASTLETON BRIDGE
JPC # 23456-015

LIFT LOOP/LIFT INSERT DESIGN CALCULATIONS
(WORK W/ CARRARA SHOP DWNGS)

APPROACH SLABS $f'_c = 4000$ PSI $f'_{ci} = 3500$ PSI △ 3-26-15 EJB

WT = 13.48 T = 26.95 K

THERE ARE (4) LIFT POINTS

LOAD/LIFT POINT = 26.95/4 = 6.74 K

FROM ATTACHED PRODUCT LITERATURE, p 28

USE 4 T x 7 1/8" S.L. SWL (4:1 S.F.) = 8000 LB > 6.74 K, O.K.
(LOCATE APPROX. 2/5 POINTS EACH WAY, CENTER LIFTER PATTERN ON C.G. OF APPROACH SLABS.)

NEXT 28D BEAMS $f'_c = 10000$ PSI $f'_{ci} = 7500$ PSI

HEAVIEST BM WT = 56.90 T = 113.8 K (BEAMS W/ CURBS)

THERE ARE (4) LIFT POINTS

C.G. IS 5 11/16" FROM ℄ BM DUE TO CURB SEE SHT 3

MAX LOAD/LIFT POINT = $\frac{113.8}{2} \times \left(\frac{2'\text{-}6" + 5\frac{11}{16}"}{5'\text{-}0"}\right) = 33.8^K$

ASSUMING ~ 60" SLING ANGLE W/THE HORIZONTAL

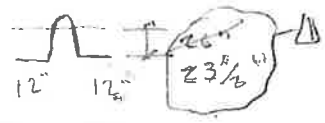
DESIGN LOAD/LIFT POINT

= 33.8/0.866 = 39.0 K

FROM ATTACHED PCI LITERATURE & ALLOWING FOR $f'_{ci} = 7500$ PSI

USE (4) 1/2" ϕ x 270 KSI STRAND LIFT LOOPS, MIN.

EMBEDMENT ~~26"~~ 23 1/2" △ WITH 1'-0" HOOKS



FROM ATTACHED PCI LITERATURE, TABLE 5.2.3

EMBEDMENT ≈ 26" + 12" = ~~38"~~ 35 1/2" △ > 39"

SWL (4:1 S.F.) ≈ $\left(\frac{29+41}{2}\right)\sqrt{\frac{7500}{3000}}$ = 55.3 K > 39.0 K, O.K., BY INTERPOLATION BETWEEN VERTICAL & 45° POL

(NOTE: IT IS REASONABLE TO INCREASE CAPACITY OF LIFT LOOP BY SQUARE ROOT RATIO OF THE CONCRETE STRENGTH)

STATE OF VERMONT ERNEST J. BROD No. 2814 REGISTERED PROFESSIONAL ENGINEER

TOPS

ALSO CHECK USING PCI FIGURE 6.15.7A, CASE 3 ATTACHED

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

$f'_c = 10000$ PSI

$f'_{ci} = 7500$ PSI

$$\phi P_c = 0.85 \times 2.67\sqrt{7500}\left(\frac{13.75+15}{2}\right)\left(0 + 2 \times \cancel{26}\ 23\tfrac{1}{2}\right) \quad (1)$$

$$= 146917 \text{ LB} = \cancel{146.9^k}\ 132.8^k \quad (1) \quad @ \text{ STRIPPING}$$

FOR S.F. = 4:1 SWL = $\cancel{146.9}\ 132.8 / 4 \cdot \cancel{36.7}\ 33.2^k \approx 39.0^k$ (1) (1) @ STRIPPING

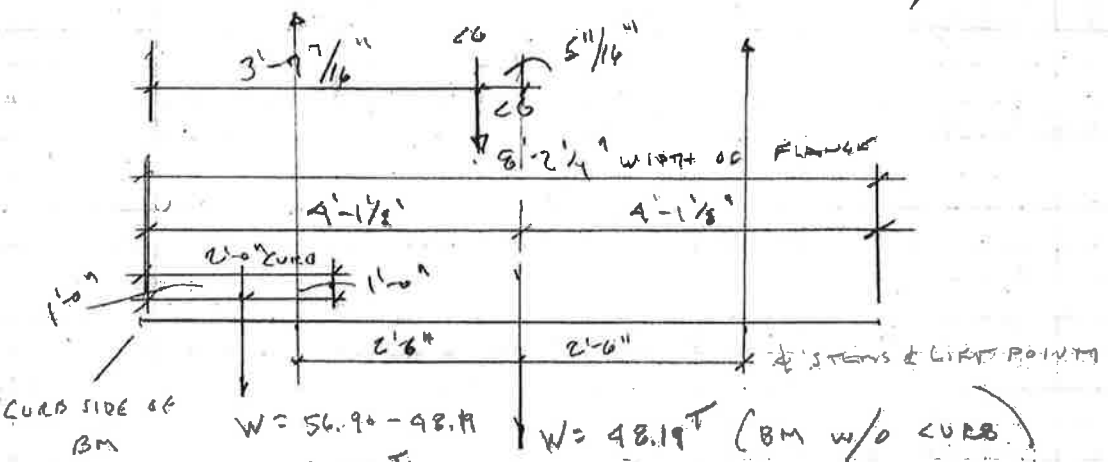
(WIN ACCEPT S.F. = $\frac{4 \times \cancel{36.7}\ 33.2}{39} = \cancel{3.76}\ 3.41$

ALSO FOR S.F. = 4: SWL = $\cancel{36.7}\ 33.2 \times \sqrt{\frac{10000}{7500}} = \cancel{42.4}\ 38.4^k$ @ ERECTION.

$> 36.7^k$, O.K. (1)

CASTLETON BRIDGE
JPC # 23456-01E

LOCATE C.G. OF BRIDGE 28D BEAM W/ CURBS



W = 56.90 − 48.19
= 8.71 T.
(CURB)

W = 48.19 T (BM w/o CURB)
FROM BEAM SHOP DWGS

$$\bar{x}_{\text{FROM LEFT}} = \frac{48.19 \times 4.09 + 8.71 \times 1.0}{48.19 + 8.71} = 3.67'$$

OR 3'-7 7/16" FROM LEFT SIDE OR

5 11/16" FROM ℄ BEAM

WING WALLS $f'_c = 5000$ PSI $f'_{ci} = 3500$ PSI

WT OF HEAVIEST WING WALL (WW 3) = 13.95^T

$f'_c = 5000$ PSI $f'_{ci} = 3500$ PSI THICKNESS = 18"

STRIPPING - PANEL IN FLAT

USE (4) LIFTERS IN FACE OF PANEL LOCATED SYMMETRICALLY ABOUT CG OF PANEL APPROX @ 1/5 POINTS E.W.

TENSION, LOAD/LIFT POINT = $\frac{13.95 \times 2}{4}$ = 6.98^K

FROM ATTACHED PRODUCT LITERATURE, SHT 28

USE 4^{T}x7 1/4" S.L. SWL(4:1 S.F.) = 8000 LB > 6.98^K, OK

ERECTION - ROLL PANEL ABOUT BOTTOM EDGE USING (2) LIFTERS @ T/EDGE OF PANEL

FIRST MAKE APPROXIMATE CHECK FOR CRACKING OF PANEL DURING PANEL ROTATION; USE TALLEST WING WALL (WW 2 @ 12'-0" HEIGHT, MAX)

w = 225 PLF; PANEL IS 18" THICK

$M = wl^2/8 = 1.3 \times 225 \times 12.0^2/8 = 5265$ LB-FT/FT; 1.3 IS ROTATION FACTOR

$S = bh^2/6 = 12 \times 18^2/6 = 648$ IN^3/FT

$f = 5265 \times 12/648 = 98$ PSI << $5\sqrt{3500} = 296$ PSI, OK,

PANEL CAN BE ROLLED UP W/O CRACKING

SHEAR LOAD, V. KITTOP INSERT

$$= \frac{13.95 \times 2}{2 \times 2} = 6.98^k$$

FROM ATTACHED PRODUCT LITERATURE, SHT 20

USE $8^T \times 13\frac{3}{4}''$ S.L., SWL(4:1 S.F.) = 6.7^k

IN 9" THICK PANEL & f'_c = 4500 PSI, ∴ BY

INSPECTION FOR 18" THICK PANEL & f'_{cu} = 3500 PSI

SWL(4:1 S.F.) > 6.98^k APPLIED SHEAR LOAD, O.K.

USE #5 TIGHT TO LIFTER ON TOP

TOPS

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.

THESE ARE GOVERNED BY SHEAR; FOR TENSION LOAD USE 45° SHEAR CONE I.E. EDGE DISTANCE IS ≈ LENGTH OF INSERT.

10"

10"

8"

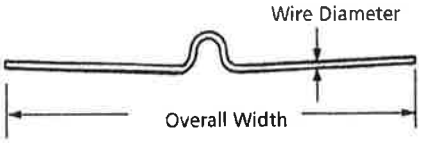
FOR TENSION LOAD

Swift Lift® System

DAYTON SUPERIOR

P-59 Swift Lift® Shear Bar
P-59 Swift Lift® Smooth Wire Shear Bar

P-59 Swift Lift Shear Bar
1-Ton and 8-Ton



P-59 Swift Lift Smooth Wire Shear Bar
2-Ton and 4-Ton

Dayton Superior Swift Lift Shear Bars (P-59) are utilized wh edge lifting precast elements. The shear bar is secured tightly the recess plug and at the time of lift helps to transfer the sh load deeper into the concrete. The P-59 Smooth Wire Shear B is designed to snap into the built-in clips on the P-54 recess plug. The standard shear bar is fabricated from rebar and mu be securely wired tightly to the Swift Lift recess plugs.

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

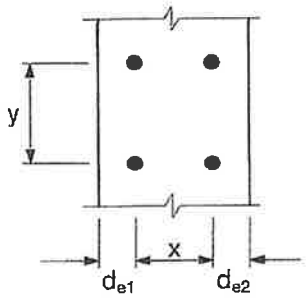
Example:
200, P-59 Swift Lift Shear Bars, 4-Ton.

Anchor Safe Working Load (tons)	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	Rebar	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.

17.

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

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

$$\phi = 0.85$$

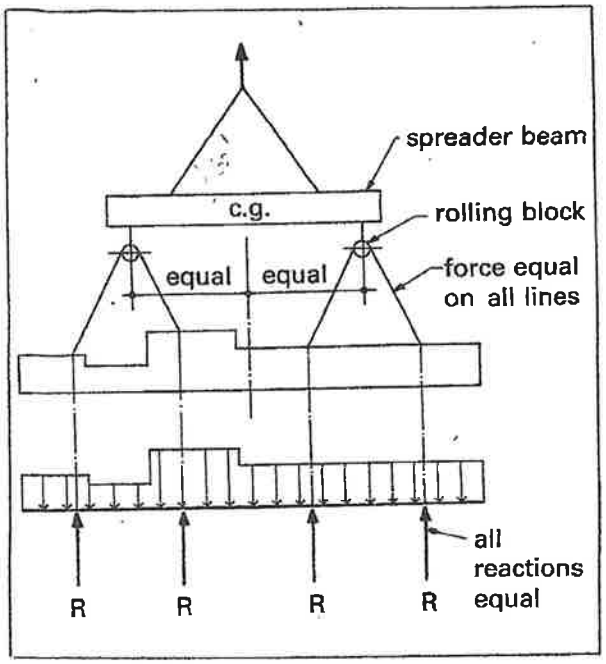
where: x_1 and y_1 are the dimensions of the flat bottom of the part of the truncated pyramid.

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 $\lambda\sqrt{f'_c/5000}$

h, 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
5	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
6	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

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.

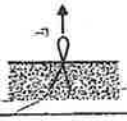
Reduced capacities for shorter embedment lengths may be suitable. In shallow products, providing a 90° bend can reduce the required embedment length significantly. Lightly rusted strand has better bond than bright strand.

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 %

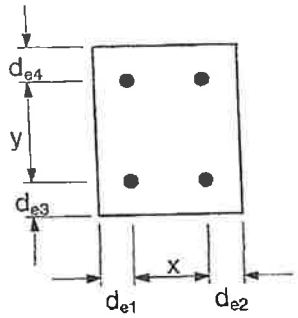
The angle of incline of lifting has little effect on the strand lifting loop capacity if the angle from the horizontal is more than about 20°. Typical handling methods are usually such that this angle is no less than 60°.

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.

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 [illegible] of the part of the truncated pyramid.

For Case 6: $x_1 = x + d_{e1} + d_{e2}$ $\quad y_1 = y - [illegible]$

Note: Table values are based on $\lambda = 1.0$ and $f'_c = 5000$ psi;

for different material properties, multiply [illegible] values by $\lambda\sqrt{f'_c/5000}$

ℓ_e in.	y_1, in. \ x_1, in.	2	4	6	8	10	12	14	16	18	20	22	24	26	[illegible]
		Design tensile strength, ϕP_{c1} (kips)													
3	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	[illegible]
	2	1	1	2	3	3	4	5	5	6	7	7	8	[illegible]	[illegible]
	4	1	3	4	5	7	8	9	10	11	13	14	15	[illegible]	[illegible]
	6	2	4	6	8	9	11	13	15	17	19	21	23	[illegible]	[illegible]
	8	3	5	8	10	13	15	18	21	23	25	29	31	[illegible]	[illegible]
	10	3	7	9	13	16	19	23	25	29	32	35	39	[illegible]	[illegible]
	12	4	8	11	15	19	23	27	31	35	39	42	46	[illegible]	[illegible]
	14	5	9	13	18	23	27	31	36	41	45	49	54	[illegible]	[illegible]
	16	5	10	15	21	25	31	36	41	46	51	35	61	[illegible]	[illegible]
4	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	[illegible]
	2	1	1	2	3	3	4	5	5	6	7	7	8	[illegible]	[illegible]
	4	1	3	4	5	7	8	9	10	11	13	14	15	[illegible]	[illegible]
	6	2	4	6	8	9	11	13	15	17	19	21	23	[illegible]	[illegible]
	8	3	5	8	10	13	15	18	21	23	25	28	31	[illegible]	[illegible]
	10	3	7	9	13	16	19	23	25	29	32	35	39	[illegible]	[illegible]
	12	4	8	11	15	19	23	27	31	35	39	42	46	[illegible]	[illegible]
	14	5	9	13	18	23	27	31	36	41	45	49	54	[illegible]	[illegible]
	16	5	10	15	21	25	31	36	41	46	51	57	61	[illegible]	[illegible]
6	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	[illegible]
	2	1	1	2	3	3	4	5	5	6	7	7	8	[illegible]	[illegible]
	4	1	3	4	5	7	8	9	10	11	13	14	15	[illegible]	[illegible]
	6	2	4	6	8	9	11	13	15	17	19	21	23	[illegible]	[illegible]
	8	3	5	8	10	13	15	18	21	23	25	28	31	[illegible]	[illegible]
	10	3	7	9	13	16	19	23	25	29	32	35	39	[illegible]	[illegible]
	12	4	8	11	15	19	23	27	31	35	39	42	46	[illegible]	[illegible]
	14	5	9	13	18	23	27	31	36	41	45	49	54	[illegible]	[illegible]
	16	5	10	15	21	25	31	36	41	46	51	57	61	[illegible]	[illegible]
8	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	[illegible]
	2	1	1	2	3	3	4	5	5	6	7	7	8	[illegible]	[illegible]
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	8	3	5	8	10	13	15	18	21	23	25	28	31	[illegible]	[illegible]
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