Appendix A

Test Set Up Design Calculations

A.1 General Assumption

Assume the target (maximum) story shear distribution is 600 kips at roof level and 300 kips at the top of the first story. Thus, the total shear is 900 kips at base. The friction coefficient between reconfigurable reaction blocks is assumed equal to 0.5 under the condition that the blocks were properly grouted (Mosalam and Elkhoraibi, 2004). While the friction coefficient between steel plate and concrete surface is assumed equal to 0.33 without grouting and equal to 0.5 with properly grouting between them. Assume the eccentricity of the actuator measured from the concrete block surface to the center line of the actuator is 36 inch in this case. The total eccentricity from the center of gravity of concrete block sections to the center line of actuator is 36 + (10 ft $\times$ 12 in/ft / 2) = 96 inch.

A.2 Materials:

<u>Concrete:</u>

Strong floor: $f_c^{'} = 4500\,psi$ (Skidmore, Owings & Merrill 1963 and 1964)

Reconfigurable reaction wall (RRW): $f_c^{'} = 8000\,psi$ (Arici and Mosalam, 2001; Mosalam and Elkhoraibi, 2004; Clyde, 2001)

<u>Steel plate and rebar:</u>

ASTM A36 steel plate: $f_y = 36\,ksi$, $f_u = 58\,ksi$ (ASTM, 2004)

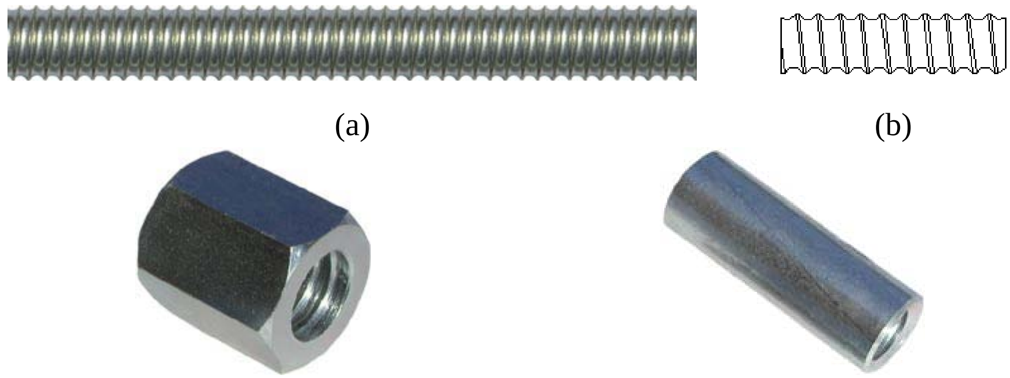
ASTM A572 Grade 50 steel plate: $f_y = 50\,ksi$, $f_u = 65\,ksi$ (ASTM, 2003)

ASTM A706 rebar: $f_y = 60\,ksi$ (ASTM, 2004)

<u>Pre-stress rod (all-thread-bar):</u>

Williams Form 150 ksi all-thread-bar: $f_y = 127.7\,ksi$, $f_u = 150\,ksi$ (Williams Form Eng. Corp., 2008].

The appearance of a typical all-thread-bar is shown in Fig. A.1 and Table A.1 shows the typical dimension and material properties of 150 ksi all-thread-bar used in the NEES Berkeley lab.



(a) (b) (c) (d)

Figure A.1 Appearance of a typical all-thread-bar, nut and coupler
(adopted from Williams Form Eng. Corp. website)

Table A.1 Typical dimension and material properties of 150 ksi all-thread-bar

Outside Diameter (in)	Nominal Diameter (in)	Net Area (in^2)	P_y (kips)	P_u (kips)	70% P_u (kips)
1-9/16	1-3/8	1.48	189	222	155

(Note: the strong floor has 2.5 in inside diameter holes 3 ft on center each direction)

Consider the outside diameter of coupling is normally 7/8 inch larger than the nominal diameter of the all-thread-bar. For a 2.5 inch hole and the all-thread-bar with 1-3/8 inch nominal diameter, $1.375 + 7/8 = 2.25 < 2.5$ inch, the holes in the strong floor are large enough for using the all-thread-bar with couplers. Table A.2 and A.3 show the available all-thread-bars in the lab and the required quantities for the test setup.

Table A.2 Number of all-thread-bars available in RFS lab

Outside Diameter (in)	Nominal Diameter (in)	Length (ft)	Quantity	Note
1-9/16	1-3/8	13	> 90	8 ft length also available

(Need to check the quantities we have in RFS)

Table A.3 Number of required all-thread-bars and accessories for three stacks of RRW

Post-tension Direction	Grip Length (ft)	All-thread-bars Length		Couplers, Nuts and End Plates
		13 ft	8 ft	
Vertical	27	30	60	60 each
Horizontal	27	20	40	40 each
Horizontal (inside RRW)	4	0	60	120 (nuts and plates)
Total		50	160	220 (nuts and plates)

(Note: Assume at least 2 ft working length for pre-stressing)

A.3 Design Checks and Calculations

A.3.1. Check the possible failure modes of test setup

The calculation checks basically follow the load path from the upper level of the RRW (Reconfigurable Reaction Wall) to the bottom level of the RRW and then to the lab strong floor. The following paragraphs describe the design check sequences.

(1) Check the actuator bracket capacity:

The two existing brackets in the lab were originally designed for the Caltrans research project (Astaneh-Asl and Ravat, 1998). The capacity of combined brackets (back-to-back) was 1500 kips to fit in with the 1.5 million pound actuators available in the lab. Therefore, the capacity of each bracket is initially estimated to be at least 750 kips. Here we conservatively check the critical stress in the back plate of a bracket under 600 kips loading and assume the steel brackets were made by ASTM A572 Grade 50 steel. Fig. A.2 shows the simple beam model to calculate the stress distribution in the back plate. From the formula listed in the structural design manual (Kiyota and Tamagawa, 2004), we can calculate the stresses at three critical points (middle and both ends of the beam) in the beam model:

$$a = 13''; \; b = 34''; \; l = 2a + b = 60''; \; t = 4''$$

$$I = \frac{1}{12} \cdot 44'' \cdot (4'')^3 = 235\, in^4$$

$$w = \frac{600\, kips}{34''} = 17.6\, kip/in$$

$$M_A = M_B = \frac{wb}{24l}(3l^2 - \frac{b^2}{2}) = 4248\, kip-in$$

$$\sigma_A = \sigma_B = \frac{M_A \cdot t}{2 \cdot I} = 36.2\,ksi < 50\,ksi$$

$$M_C = \frac{wb}{24l}(3l^2 + \frac{b^2}{2} - 3bl) = 2185\,kip - in$$

$$\sigma_C = \frac{M_C \cdot t}{2 \cdot I} = 18.6\,ksi < 50\,ksi$$

$$\delta_C = \frac{wb}{216EI}(l^3 - \frac{b^2 l}{2} + \frac{b^3}{2}) = 0.082''$$

The stresses in the beam model are less than the minimum yield strength of the back plates assumed in the calculation.

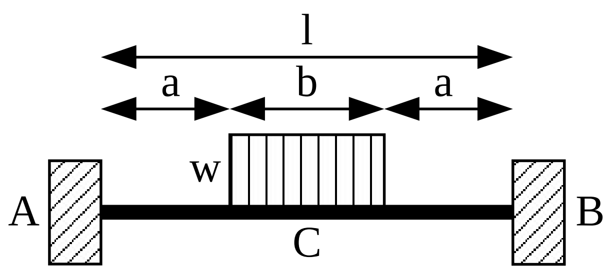


Figure A.2 The simple beam model for stress check

(2) Check the upper level bracket base plate to concrete block surface friction: (assume 100 kips and 50 kips pretension forces in the all-thread-bars)

$$\mu = 0.33$$

$$R = (100\,kips \cdot 21 + 50\,kips \cdot 18) \cdot 0.33 = 990\,kips > 600\,kips \ \ (OK)$$

(3) Check the strain in the base plate of upper level bracket to prevent cracks in the RRW blocks:

$$f_t = 0.1 \cdot f_c^{'} = 0.1 \cdot 8000\ psi = 800\ psi$$

$$E_c = w_c^{1.5} \cdot 33\sqrt{f_c^{'}} = 145^{1.5} \cdot 33 \cdot \sqrt{8000} = 5153.6\,ksi$$

$$\varepsilon_{cr} = \frac{f_t}{E_c} = 1.55 \times 10^{-4}$$

$$check\ \ \varepsilon_{\text{plate}} < \varepsilon_{cr}\ \ (under\ \ 600\,kips)$$

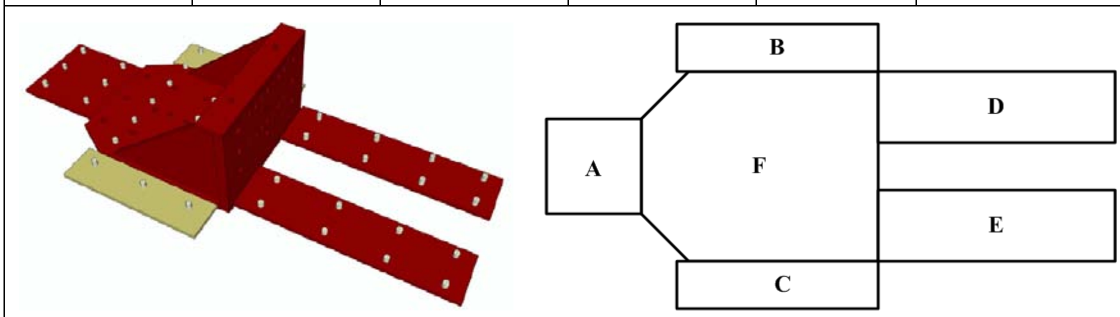
$$\sigma_{plate} = E_S \cdot \varepsilon_{plate} < E_S \cdot \varepsilon_{cr} = 4.5\,ksi$$

$$check\ \frac{P}{A_S} < 4.5\,ksi$$

From Table A.4, we can clearly see that stresses in all plate extensions are less than the estimated allowable stress.

Table A.4 Stress distribution in each base plate extensions

Zone	Force Ratio	Force P (kips)	A_s (in^2)	Stress (ksi)	Safety Factor (S.F.)
A	0.15	90	84	1.07	4.2
B	0.083	49.8	36	1.38	3.3
C	0.083	49.8	36	1.38	3.3
D	0.2	120	48	2.5	1.8
E	0.2	120	48	2.5	1.8
F	0.284	170.4	210	0.81	5.6



(4) Check local shear in RRW block:

Refer to Fig. A.3 below. One can check the shear force transferred in the concrete blocks. Shear force in the typical profile in each concrete block:

$$\frac{600\,kips}{6\cdot 2} = 50\,kips$$

$$V_n = V_c + V_s$$

$$V_c = 2\cdot\sqrt{8000}\cdot 30''\cdot(23.5 - 1.5\times 2 - 0.5\times 2) = 104.6\,kips$$

$$V_s = \frac{A_v f_y d}{s} = \frac{2\cdot 0.20\cdot 60ksi\cdot 19.5}{4} = 117\,kips$$

$$\phi V_n = 0.85\cdot 221.6 = 188.4 > 50\,kips \;\; (OK)$$

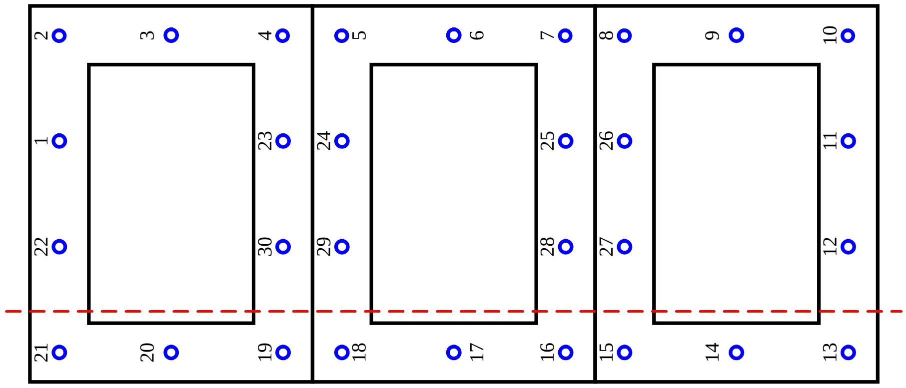


Figure A.3 Local shear force profile for the concrete blocks

(5) Check the torsional resistance between the concrete blocks: (assume 100 kips pretension for all-thread-bars)

Using the traditional elastic (vector) analysis method (Salmon and Johnson, 1996) to calculate the torsional resistance of the concrete blocks. Fig. A.4 demonstrates the concept of this method.

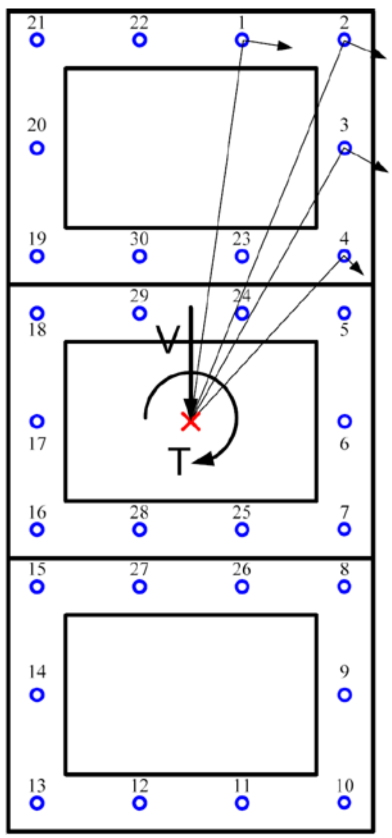


Figure A.4 Torsional force profile for the RRW concrete blocks

In Fig. A.4,

$$T = 600\,kips \cdot 96\,in = 57600\ \ kip - in$$

$$V = 600\,kips$$

$$F = \mu N = 0.5 \cdot 100 = 50\,kips$$

The screenshot from the excel calculation sheet is shown in Fig. A.5.

PT#	xi	yi	ri	ri^2	ri / ri^2	Fi	Fix	Fiy	Fv	Vector Sum	Check
Clockwise	in	in	in	in^2	1 / in	kips	kips	kips	kips	kips	50
1	18	144	145.12	21060.00	0.000452	26.06	25.86	3.23	20	34.76	Okay
2	54	144	153.79	23652.00	0.000479	27.62	25.86	9.70	20	39.38	Okay
3	54	108	120.75	14580.00	0.000376	21.68	19.39	9.70	20	35.47	Okay
4	54	72	90.00	8100.00	0.000281	16.16	12.93	9.70	20	32.39	Okay
5	54	36	64.90	4212.00	0.000202	11.65	6.46	9.70	20	30.39	Okay
6	54	0	54.00	2916.00	0.000168	9.70	0.00	9.70	20	29.70	Okay
7	54	-36	64.90	4212.00	0.000202	11.65	-6.46	9.70	20	30.39	Okay
8	54	-72	90.00	8100.00	0.000281	16.16	-12.93	9.70	20	32.39	Okay
9	54	-108	120.75	14580.00	0.000376	21.68	-19.39	9.70	20	35.47	Okay
10	54	-144	153.79	23652.00	0.000479	27.62	-25.86	9.70	20	39.38	Okay
11	18	-144	145.12	21060.00	0.000452	26.06	-25.86	3.23	20	34.76	Okay
12	-18	-144	145.12	21060.00	0.000452	26.06	-25.86	-3.23	20	30.82	Okay
13	-54	-144	153.79	23652.00	0.000479	27.62	-25.86	-9.70	20	27.84	Okay
14	-54	-108	120.75	14580.00	0.000376	21.68	-19.39	-9.70	20	21.96	Okay
15	-54	-72	90.00	8100.00	0.000281	16.16	-12.93	-9.70	20	16.53	Okay
16	-54	-36	64.90	4212.00	0.000202	11.65	-6.46	-9.70	20	12.16	Okay
17	-54	0	54.00	2916.00	0.000168	9.70	0.00	-9.70	20	10.30	Okay
18	-54	36	64.90	4212.00	0.000202	11.65	6.46	-9.70	20	12.16	Okay
19	-54	72	90.00	8100.00	0.000281	16.16	12.93	-9.70	20	16.53	Okay
20	-54	108	120.75	14580.00	0.000376	21.68	19.39	-9.70	20	21.96	Okay
21	-54	144	153.79	23652.00	0.000479	27.62	25.86	-9.70	20	27.84	Okay
22	-18	144	145.12	21060.00	0.000452	26.06	25.86	-3.23	20	30.82	Okay
23	18	72	74.22	5508.00	0.000231	13.33	12.93	3.23	20	26.59	Okay
24	18	36	40.25	1620.00	0.000125	7.23	6.46	3.23	20	24.11	Okay
25	18	-36	40.25	1620.00	0.000125	7.23	-6.46	3.23	20	24.11	Okay
26	18	-72	74.22	5508.00	0.000231	13.33	-12.93	3.23	20	26.59	Okay
27	-18	-72	74.22	5508.00	0.000231	13.33	-12.93	-3.23	20	21.17	Okay
28	-18	-36	40.25	1620.00	0.000125	7.23	-6.46	-3.23	20	17.97	Okay
29	-18	36	40.25	1620.00	0.000125	7.23	6.46	-3.23	20	17.97	Okay
30	-18	72	74.22	5508.00	0.000231	13.33	12.93	-3.23	20	21.17	Okay
				320760.00							

T	57600	kip-in	(clockwise)
V	600	kips	(downward)
ecc	96	in	

Pretension	100	kips
μ	0.5	

Figure A.5 A screenshot from the excel calculation sheet

(6) Check the torsion in the concrete blocks:

Assume three RRW blocks behave together as a cell at upper level:

$$A_{cp} = (10\times12)(27\times12) = 38880\ \ in^2$$
$$p_{cp} = 2\cdot[(10\times12)+(27\times12)] = 888\ \ in$$
$$t = \frac{A_{cp}}{p_{cp}} = 43.8\ \ in$$

$$T_{\min} = \phi\sqrt{f_c^{'}}\frac{A_{cp}^2}{p_{cp}} = 1.3\times10^8\ \ \mathrm{lb}\text{-}in = 129421\ \ kip\text{-}in > 57600\ \ kip\text{-}in \qquad \text{(Meyer, 1996)}$$

Thus, we can neglect the torsional effect in the RRW block, this failure mode will not govern.

(7) Check shear force in the entire concrete blocks:

Assume three RRW blocks behave together at upper level: (Fig. A.6)

$$V_u = 600\ \ kips$$
$$V_n = V_c + V_s$$
$$V_c = 2\cdot\sqrt{8000}\cdot(18.75\times2)\cdot(27\times12-1.5\times2-0.5\times2) = 2146.6\,kips$$
$$\phi V_c = 0.85\cdot2146.6 = 1824.6 > 600\,kips\ \ (OK)$$

From the calculation shown above (neglect contribution of shear reinforcements), this failure mode will not control.

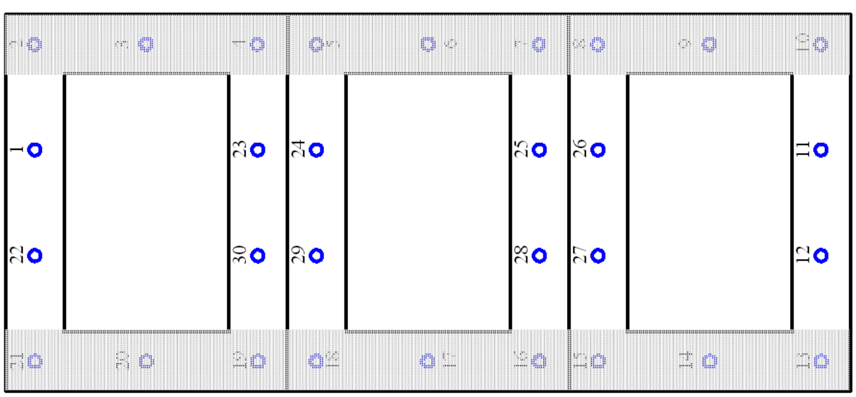


Figure A.6 Assumed shear force profile for the concrete blocks

(8) Check the (shear) friction between concrete blocks:

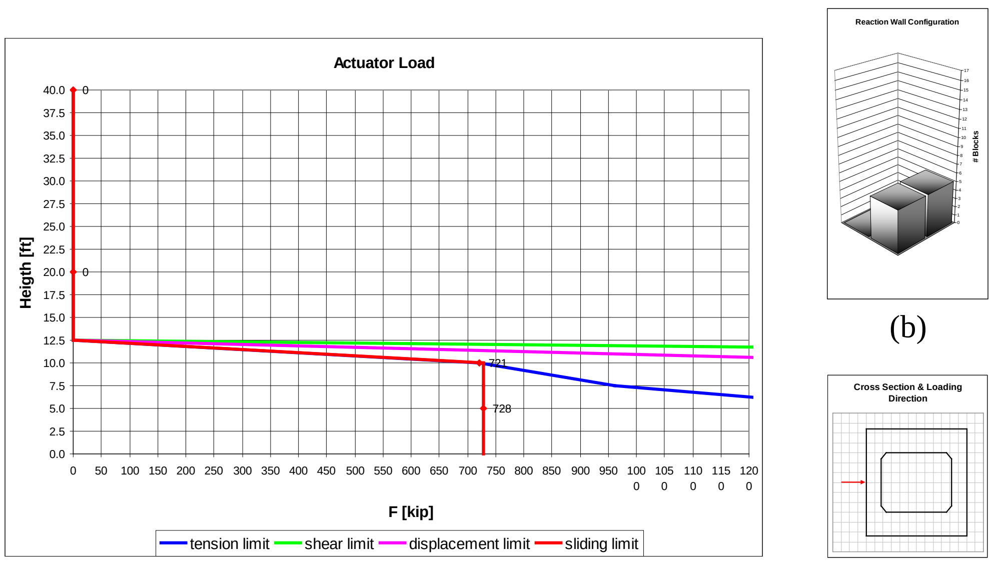
$$\mu = 0.5$$
$$(10\times3)\times100\ \ kips\times\mu = 1500\ \ kips > 600\ \ kips\ \ (OK)$$

(9) Check the overturning moment for upper level (10 ft height):

From the excel calculation file (Schellenberg, 2004) downloaded from NEES@berkeley website. One can calculate the moment capacity for the two-stack RRW:

$$600 kips \times 10\,ft = 6000\,k - ft < 7210\,k - ft\;(OK)$$

Note that this is the two-stack RRW result. It is reasonable to assume that the three-stack RRW will have larger moment capacity. Fig. A.7 shows screen shots of the excel calculation results.



(a) (b) (c)

Figure A.7 The screenshots of the excel calculation results

(10) Check the lower level bracket base plate to concrete block surface friction: (assume 100 kips and 50 kips pretension forces in the all-thread-bars)

$$\mu = 0.33$$

$$R = (100\,kips \cdot 23 + 50\,kips \cdot 17) \cdot 0.33 = 1040\,kips > 300\,kips\;\;(OK)$$

(11) Check the strain in the base plate of lower level bracket to prevent cracks in the RRW concrete blocks:

$$f_t = 0.1 \cdot f_c^{'} = 0.1 \cdot 8000\ psi = 800\ psi$$

$$E_c = w_c^{1.5} \cdot 33\sqrt{f_c^{'}} = 145^{1.5} \cdot 33 \cdot \sqrt{8000} = 5153.6\,ksi$$

$$\varepsilon_{cr} = \frac{f_t}{E_c} = 1.55 \times 10^{-4}$$

$$check\ \ \varepsilon_{\text{plate}} < \varepsilon_{cr}\ \ (under\ \ 300\,kips)$$

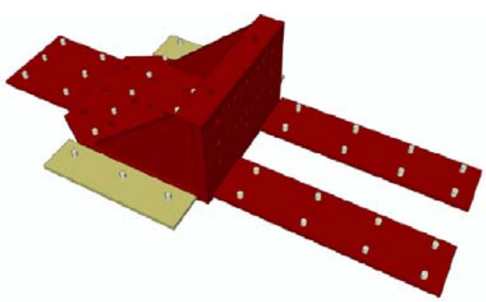
$$\sigma_{plate} = E_S \cdot \varepsilon_{plate} < E_S \cdot \varepsilon_{cr} = 4.5\,ksi$$

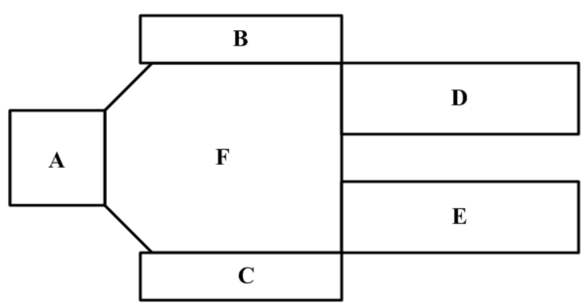
$$check\ \frac{P}{A_S} < 4.5\,ksi$$

From Table A.5, we can clearly see that stresses in all plate extensions are less than the estimated allowable stress.

Table A.5 Stress distribution in each base plate extensions

Zone	Force Ratio	Force P (kips)	A_S (in^2)	Stress (ksi)	Safety Factor (S.F.)
A	0.143	42.9	84	0.51	8.8
B	0.08	24	36	0.67	6.7
C	0.08	24	36	0.67	6.7
D	0.19	57	48	1.19	3.8
E	0.19	57	48	1.19	3.8
F	0.317	95.1	210	0.45	10.0





(12) Check local shear in RRW concrete block:

Refer to Fig. A.8 below. One can check the shear force transferred in the concrete blocks. Shear force in the typical profile in each concrete block:

$$\frac{600\,kips + 300\,kips}{6 \cdot 2} = 75\,kips$$

$$V_n = V_c + V_s$$
$$V_c = 2 \cdot \sqrt{8000} \cdot 30'' \cdot (23.5 - 1.5 \times 2 - 0.5 \times 2) = 104.6\,kips$$
$$V_s = \frac{A_v f_y d}{s} = \frac{2 \cdot 0.20 \cdot 60ksi \cdot 19.5}{4} = 117\,kips$$
$$\phi V_n = 0.85 \cdot 221.6 = 188.4 > 75\,kips \;\; (OK)$$

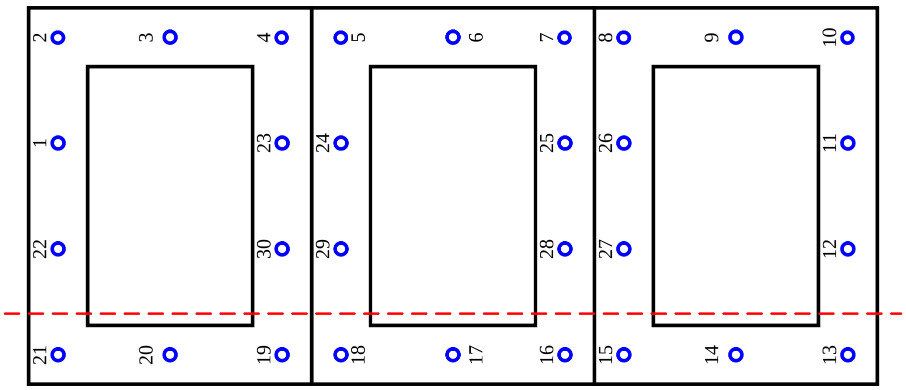


Figure A.8 Local shear force profile in the RRW concrete blocks

(13) Check the torsional resistance between the concrete blocks: (assume 100 kips pretension for all-thread-bars)

Using the traditional elastic (vector) analysis method (Salmon and Johnson, 1996) to calculate the torsional resistance of the concrete blocks. Fig. A.9 demonstrates the concept of this method.

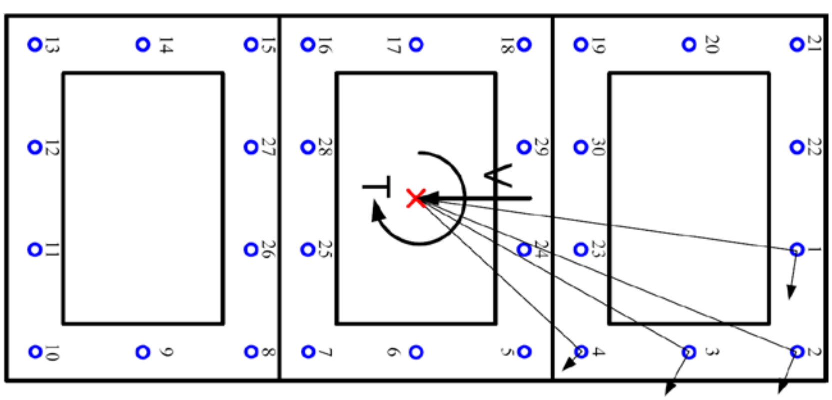


Figure A.9 Torsional force profile for the concrete blocks

In Fig. A.9,

$$T = 900\, kips \cdot 96\, in = 86400 \;\; kip - in$$

$$V = 900\, kips$$

$$F = \mu N = 0.5 \cdot 100 = 50\, kips$$

The screenshot of the detail calculation sheet is shown in Fig. A.10 below.

PT#	xi	yi	ri	ri^2	ri / ri^2	Fi	Fix	Fiy	Fv	Vector Sum	Check
Clockwise	in	in	in	in^2	1 / in	kips	kips	kips	kips	kips	50
1	18	144	145.12	21060.00	0.000452	39.09	38.79	4.85	30	52.14	NG
2	54	144	153.79	23652.00	0.000479	41.43	38.79	14.55	30	59.07	NG
3	54	108	120.75	14580.00	0.000376	32.52	29.09	14.55	30	53.20	NG
4	54	72	90.00	8100.00	0.000281	24.24	19.39	14.55	30	48.58	Okay
5	54	36	64.90	4212.00	0.000202	17.48	9.70	14.55	30	45.59	Okay
6	54	0	54.00	2916.00	0.000168	14.55	0.00	14.55	30	44.55	Okay
7	54	-36	64.90	4212.00	0.000202	17.48	-9.70	14.55	30	45.59	Okay
8	54	-72	90.00	8100.00	0.000281	24.24	-19.39	14.55	30	48.58	Okay
9	54	-108	120.75	14580.00	0.000376	32.52	-29.09	14.55	30	53.20	NG
10	54	-144	153.79	23652.00	0.000479	41.43	-38.79	14.55	30	59.07	NG
11	18	-144	145.12	21060.00	0.000452	39.09	-38.79	4.85	30	52.14	NG
12	-18	-144	145.12	21060.00	0.000452	39.09	-38.79	-4.85	30	46.23	Okay
13	-54	-144	153.79	23652.00	0.000479	41.43	-38.79	-14.55	30	41.75	Okay
14	-54	-108	120.75	14580.00	0.000376	32.52	-29.09	-14.55	30	32.94	Okay
15	-54	-72	90.00	8100.00	0.000281	24.24	-19.39	-14.55	30	24.80	Okay
16	-54	-36	64.90	4212.00	0.000202	17.48	-9.70	-14.55	30	18.24	Okay
17	-54	0	54.00	2916.00	0.000168	14.55	0.00	-14.55	30	15.45	Okay
18	-54	36	64.90	4212.00	0.000202	17.48	9.70	-14.55	30	18.24	Okay
19	-54	72	90.00	8100.00	0.000281	24.24	19.39	-14.55	30	24.80	Okay
20	-54	108	120.75	14580.00	0.000376	32.52	29.09	-14.55	30	32.94	Okay
21	-54	144	153.79	23652.00	0.000479	41.43	38.79	-14.55	30	41.75	Okay
22	-18	144	145.12	21060.00	0.000452	39.09	38.79	-4.85	30	46.23	Okay
23	18	72	74.22	5508.00	0.000231	19.99	19.39	4.85	30	39.88	Okay
24	18	36	40.25	1620.00	0.000125	10.84	9.70	4.85	30	36.17	Okay
25	18	-36	40.25	1620.00	0.000125	10.84	-9.70	4.85	30	36.17	Okay
26	18	-72	74.22	5508.00	0.000231	19.99	-19.39	4.85	30	39.88	Okay
27	-18	-72	74.22	5508.00	0.000231	19.99	-19.39	-4.85	30	31.76	Okay
28	-18	-36	40.25	1620.00	0.000125	10.84	-9.70	-4.85	30	26.96	Okay
29	-18	36	40.25	1620.00	0.000125	10.84	9.70	-4.85	30	26.96	Okay
30	-18	72	74.22	5508.00	0.000231	19.99	19.39	-4.85	30	31.76	Okay
				320760.00							

T	86400	kip-in	(clockwise)
V	900	kips	(downward)
ecc	96	in	

Pretension	100	kips
μ	0.5	

Figure A.10 The screenshot of the excel calculation sheet

The required pretension forces for each post-tension rod to resist the torsion force are shown in Fig. A.11 below.

PT#	xi	yi	ri	ri^2	ri / ri^2	Fi	Fix	Fiy	Fv	Vector Sum	Check
Clockwise	in	in	in	in^2	1 / in	kips	kips	kips	kips	kips	59.5
1	18	144	145.12	21060.00	0.000452	39.09	38.79	4.85	30	52.14	Okay
2	54	144	153.79	23652.00	0.000479	41.43	38.79	14.55	30	59.07	Okay
3	54	108	120.75	14580.00	0.000376	32.52	29.09	14.55	30	53.20	Okay
4	54	72	90.00	8100.00	0.000281	24.24	19.39	14.55	30	48.58	Okay
5	54	36	64.90	4212.00	0.000202	17.48	9.70	14.55	30	45.59	Okay
6	54	0	54.00	2916.00	0.000168	14.55	0.00	14.55	30	44.55	Okay
7	54	-36	64.90	4212.00	0.000202	17.48	-9.70	14.55	30	45.59	Okay
8	54	-72	90.00	8100.00	0.000281	24.24	-19.39	14.55	30	48.58	Okay
9	54	-108	120.75	14580.00	0.000376	32.52	-29.09	14.55	30	53.20	Okay
10	54	-144	153.79	23652.00	0.000479	41.43	-38.79	14.55	30	59.07	Okay
11	18	-144	145.12	21060.00	0.000452	39.09	-38.79	4.85	30	52.14	Okay
12	-18	-144	145.12	21060.00	0.000452	39.09	-38.79	-4.85	30	46.23	Okay
13	-54	-144	153.79	23652.00	0.000479	41.43	-38.79	-14.55	30	41.75	Okay
14	-54	-108	120.75	14580.00	0.000376	32.52	-29.09	-14.55	30	32.94	Okay
15	-54	-72	90.00	8100.00	0.000281	24.24	-19.39	-14.55	30	24.80	Okay
16	-54	-36	64.90	4212.00	0.000202	17.48	-9.70	-14.55	30	18.24	Okay
17	-54	0	54.00	2916.00	0.000168	14.55	0.00	-14.55	30	15.45	Okay
18	-54	36	64.90	4212.00	0.000202	17.48	9.70	-14.55	30	18.24	Okay
19	-54	72	90.00	8100.00	0.000281	24.24	19.39	-14.55	30	24.80	Okay
20	-54	108	120.75	14580.00	0.000376	32.52	29.09	-14.55	30	32.94	Okay
21	-54	144	153.79	23652.00	0.000479	41.43	38.79	-14.55	30	41.75	Okay
22	-18	144	145.12	21060.00	0.000452	39.09	38.79	-4.85	30	46.23	Okay
23	18	72	74.22	5508.00	0.000231	19.99	19.39	4.85	30	39.88	Okay
24	18	36	40.25	1620.00	0.000125	10.84	9.70	4.85	30	36.17	Okay
25	18	-36	40.25	1620.00	0.000125	10.84	-9.70	4.85	30	36.17	Okay
26	18	-72	74.22	5508.00	0.000231	19.99	-19.39	4.85	30	39.88	Okay
27	-18	-72	74.22	5508.00	0.000231	19.99	-19.39	-4.85	30	31.76	Okay
28	-18	-36	40.25	1620.00	0.000125	10.84	-9.70	-4.85	30	26.96	Okay
29	-18	36	40.25	1620.00	0.000125	10.84	9.70	-4.85	30	26.96	Okay
30	-18	72	74.22	5508.00	0.000231	19.99	19.39	-4.85	30	31.76	Okay
				320760.00							

T	86400	kip-in	(clockwise)
V	900	kips	(downward)
ecc	96	in	

Pretension	119	kips
μ	0.5	

Figure A.11 The screenshot of the required post-tension forces from the excel calculation sheet

Thus, is it suggested to increase the post-tension force from 100 kips to 120 kips per all-thread-bar.

(14) Check the torsion in the concrete blocks:

Assume three RRW blocks behave together as a cell at lower level:

$$A_{cp} = (10\times 12)(27\times 12) = 38880\ \ in^2$$
$$p_{cp} = 2\cdot[(10\times 12)+(27\times 12)] = 888\ \ in$$
$$t = \frac{A_{cp}}{p_{cp}} = 43.8\ \ in$$

$$T_{\min} = \phi\sqrt{f_c^{'}}\frac{A_{cp}^2}{p_{cp}} = 1.3\times10^8 \text{ lb-}in = 129421 \ kip\text{-}in > 86400 \ kip\text{-}in \quad \text{(Meyer, 1996)}$$

Thus, we can neglect the torsional effect in the RRW concrete blocks, this failure mode will not govern.

(15) Check shear force in the entire concrete blocks:

Assume three RRW blocks behave together at lower level: (Fig. A.12)

$$V_u = 900 \ kips$$
$$V_n = V_c + V_s$$
$$V_c = 2\cdot\sqrt{8000}\cdot(18.75\times2)\cdot(27\times12-1.5\times2-0.5\times2) = 2146.6\,kips$$
$$\phi V_c = 0.85\cdot2146.6 = 1824.6 > 900\,kips \ (OK)$$

From the calculation shown above (neglect contribution of shear reinforcements), this mode will not control.

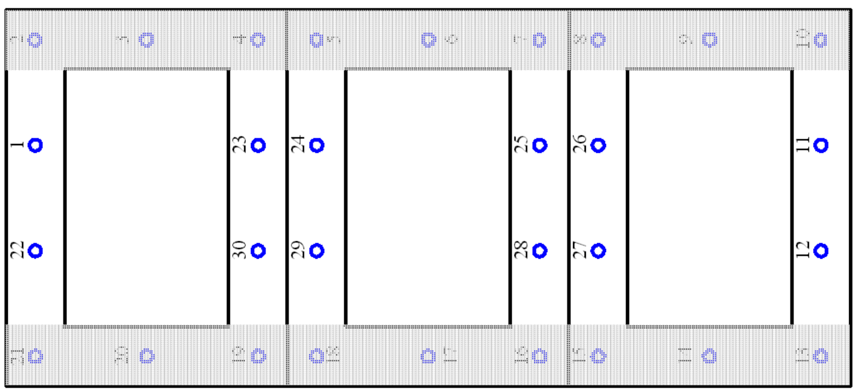


Figure A.12 Shear force profile for the concrete blocks

(16) Check the friction between concrete blocks: (Fig. A.13)

$$\mu = 0.5$$
$$(10\times3)\times100 \ kips\times\mu = 1500 \ kips > 900 \ kips \ (OK)$$

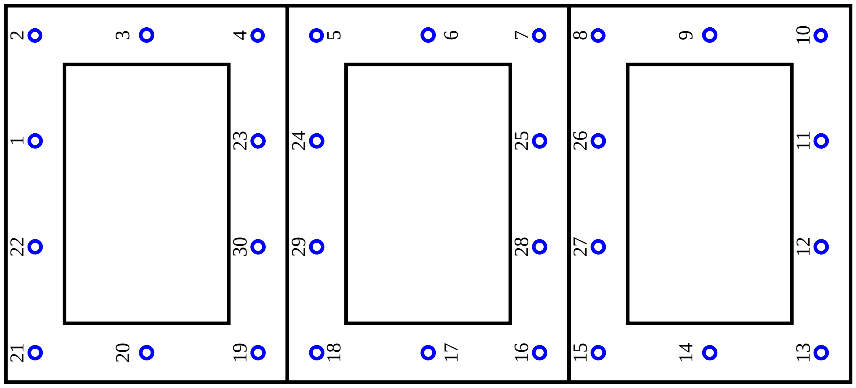


Figure A.13 Friction profile for the concrete blocks

(17) Check the overturning moment at RRW base for overall RRW (20 ft height):

Using the similar concept (Fig. A.14) in the original excel file (Schellenberg, 2004) on NEES@berkeley website, from the modified excel file we found (Fig. A.15):

$$16992\,k-ft > (600\cdot 20+300\cdot 10)=15000\,k-ft$$
$$15000\,k-ft < 20000\,k-ft \ \ (OK)$$

Also note that the design flexural capacity of strong floor, two-cell box girder, is 20000 kip-ft and the shear capacity is 1500 kips, distributed equally to three webs, which are adopted from the UCB/EERC-81/07 report (Aktan and Bertero, 1981, page 130 and 131). From the calculation sheet shows in Fig. A.16, the maximum all-thread-bar tension force after applying moment is 142.3 kip, less than 70% P_u (155 kips) of all-thread-bar (Shigley, 1972).

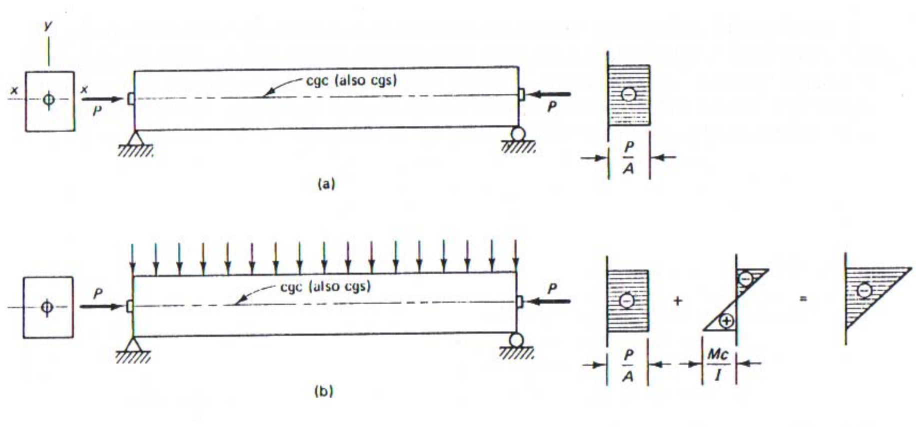


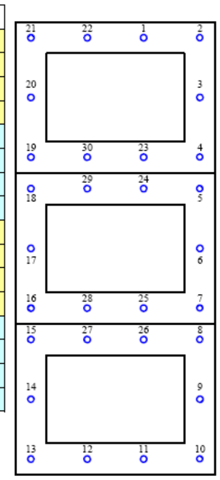
Figure A.14 Concepts of prestressing (Nawy, 1996)

Load Direction	weak	strong	axis
b	10.0	9.0	ft
d	9.0	10.0	ft
bt	2.0	1.5	ft
dt	1.5	2.0	ft
I	534.58	607.08	ft^4
(per block)	1.11E+07	1.26E+07	in^4
A	55.00	55.00	ft^2
(per block)	7920.00	7920.00	in^2
RRW weight / block	20.0	20.0	kip
PT per rod	100	100	kip
# of PT rods per RRW	10	10	rods
# of RRW per stack	10	10	blocks
# of stacks	3	3	stacks
Normal Stress	151.5	151.5	psi
y	162	180	in
$I_{overall}$	2.18E+08	2.66E+08	in^4
M_{allow}	16992	18649	kip-ft

Figure A.15 Detail calculation of the allowable overturning moment

dhole	2.5	in
dslab	24	in
drrw	300	in
dplate	5.25	in
Ab	1.48	in2
fc'_RRW	8000	psi
fc'_slab	4500	psi
Es	29000	ksi
Ec_RRW	5098	ksi
Ec_slab	3824	ksi
kb	130	kip/in
kms	216913	kip/in
kmc1	6256	kip/in
kmc2	667	kip/in
km	601	kip/in
Rb	0.1782	-
Rm	0.8218	-

Fi	P	Pb	Fb	Pm	Fm
120	83.3	14.84	134.84	68.46	-51.54
120	125.01	22.27	142.27	102.74	-17.26
120	41.67	7.42	127.42	34.25	-85.75
120	20.84	3.71	123.71	17.13	-102.87
120	-20.84	-3.71	116.29	-17.13	-137.13
120	-41.67	-7.42	112.58	-34.25	-154.25
120	-125.01	-22.27	97.73	-102.74	-222.74
120	-83.3	-14.84	105.16	-68.46	-188.46
100	83.3	14.84	114.84	68.46	-31.54
100	125.01	22.27	122.27	102.74	2.74
100	41.67	7.42	107.42	34.25	-65.75
100	20.84	3.71	103.71	17.13	-82.87
100	-20.84	-3.71	96.29	-17.13	-117.13
100	-41.67	-7.42	92.58	-34.25	-134.25
100	-125.01	-22.27	77.73	-102.74	-202.74
100	-83.3	-14.84	85.16	-68.46	-168.46



location	force	arm	bars	force/bar
P1	333.34	12	4	83.34
P2	250.01	9	2	125.00
P3	166.67	6	4	41.67
P4	83.34	3	4	20.83

M	15000	k-ft

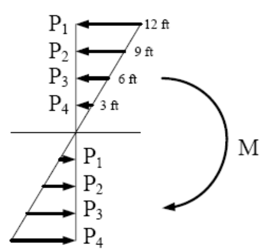


Figure A.16 Detail calculations of the pre-stressing forces in the post-tension rods

(18) Check the overturning moment that the strong floor can take (check the moment transfer through shear, see Fig. A.17):

From the formula adopted from Meyer's book (Meyer, 1996):

$$h = 24\,in,\ \ d = h - 1.5 = 22.5\,in$$
$$b_1 = a + d = 9 \cdot 3 \cdot 12 + 22.5 = 346.5\,in,\ \ b_2 = b + d = 10 \cdot 12 + 22.5 = 142.5\,in$$
$$c = \frac{b_1}{2} = 173.3\,in$$
$$J = 2\left(\frac{b_1^3 d}{12} + \frac{b_1 d^3}{12}\right) + 2b_2 d \cdot \left(\frac{b_1}{2}\right)^2 = 3.5 \times 10^8\ in^4,\ \ \gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} = 0.51$$
$$M_v = \gamma_v \cdot M_u = 0.51 \cdot (600 \cdot 20 + 300 \cdot 10) = 7650\,k - ft$$
$$v_u = \frac{V_u}{b_o d} + \frac{\gamma_v \cdot M_u \cdot c}{J} = \left(\frac{20\,kips \cdot 30\,blocks}{978 \cdot 22.5} + \frac{(7650\,k - ft) \cdot 12 \cdot 173.3}{3.5 \times 10^8}\right) \cdot 1000$$
$$v_u = 27.3 + 45.5 = 72.8\ psi$$
$$v_c = \frac{V_c}{b_o d} = \min\left[\ (2 + \frac{4}{\beta_c})\sqrt{f_c^{'}},\ (2 + \frac{\alpha_s d}{b_o})\sqrt{f_c^{'}},\ 4\sqrt{f_c^{'}}\ \right]$$
$$where\ \ \beta_c = \frac{346.5}{142.5} = 2.4,\ \ \alpha_s = 40,\ \ b_o = (b_1 + b_2) \times 2 = 978\,in$$
$$v_c = \min\left[\ 3.7\sqrt{f_c^{'}},\ 2.9\sqrt{f_c^{'}},\ 4\sqrt{f_c^{'}}\ \ \right] = 2.9\sqrt{f_c^{'}} = 2.9\sqrt{4500} = 194.5\ \ psi$$
$$\phi = 0.85 \Rightarrow v_u = 0.85 \times 194.5 = 165.4\ \ \text{psi} > 72.8\,\text{psi (OK)}$$

This failure mode will not control.

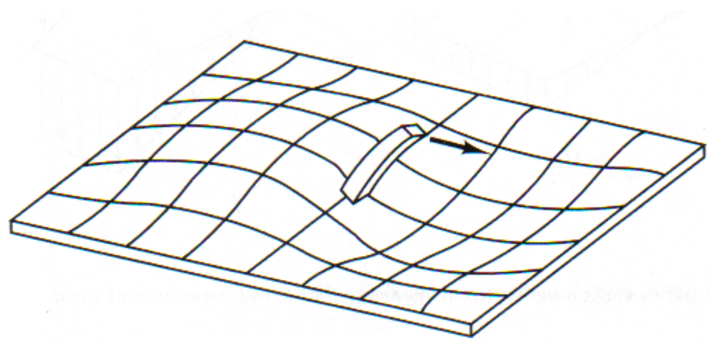


Figure A.17 Demonstration of the moment transfer through shear to the slab (Meyer, 1996)

(19) Check the required horizontal post-tension forces:

The horizontal shear force between RRW blocks is about 900 kip at lower level. Using the formula in mechanics of material textbook (Gere and Timoshenko, 1990) to check the vertical shear flow between RRW blocks:

$$f = \frac{VQ}{I} = \frac{900\,kip \cdot (10 \times 9 - 7 \times 5) \times 144\,\text{in}^2 \cdot 108\,in}{220195713\,in^4} = 3.5\,kip/in$$
$$F = f \cdot l = 3.5\,kip/in \cdot 120\,in = 420\,kips\ (vertical)$$

Assume $\mu = 0.5$, then we need at least 840 kips pre-stress force for lower four blocks. Provide two horizontal pre-stress rods per block and 120 kips pre-stress load for each rod:

$$R = 0.5 \times 4 \times 2 \times 120\, kips = 480\, kips > 420\, kips \ \ (OK)$$

(20) Check the vertical shear in the RRW concrete block assembly:

$$f = \frac{VQ}{I} = \frac{900\, kip \cdot (10 \times 9 - 7 \times 5) \times 144\, \mathrm{in}^2 (108\, in + 0.5 \cdot 34.6\, in)}{220195713\, in^4} = 4.1\, kip / in$$

$$F = f \cdot l = 4.1\, kip / in \cdot 120\, in = 492\, kips\, (vertical)$$

$$V_c = (2 \cdot \sqrt{8000}) \cdot [4\, blocks \cdot (2.5 \cdot 12) \cdot (17.5 - 1.5 \cdot 2 - 0.5 \cdot 2)] \times 2\, sides = 579.6\, kips$$

$$\phi V_c = 0.85 \cdot 579.6 = 492.7 > 492\, kips \ \ (OK)$$

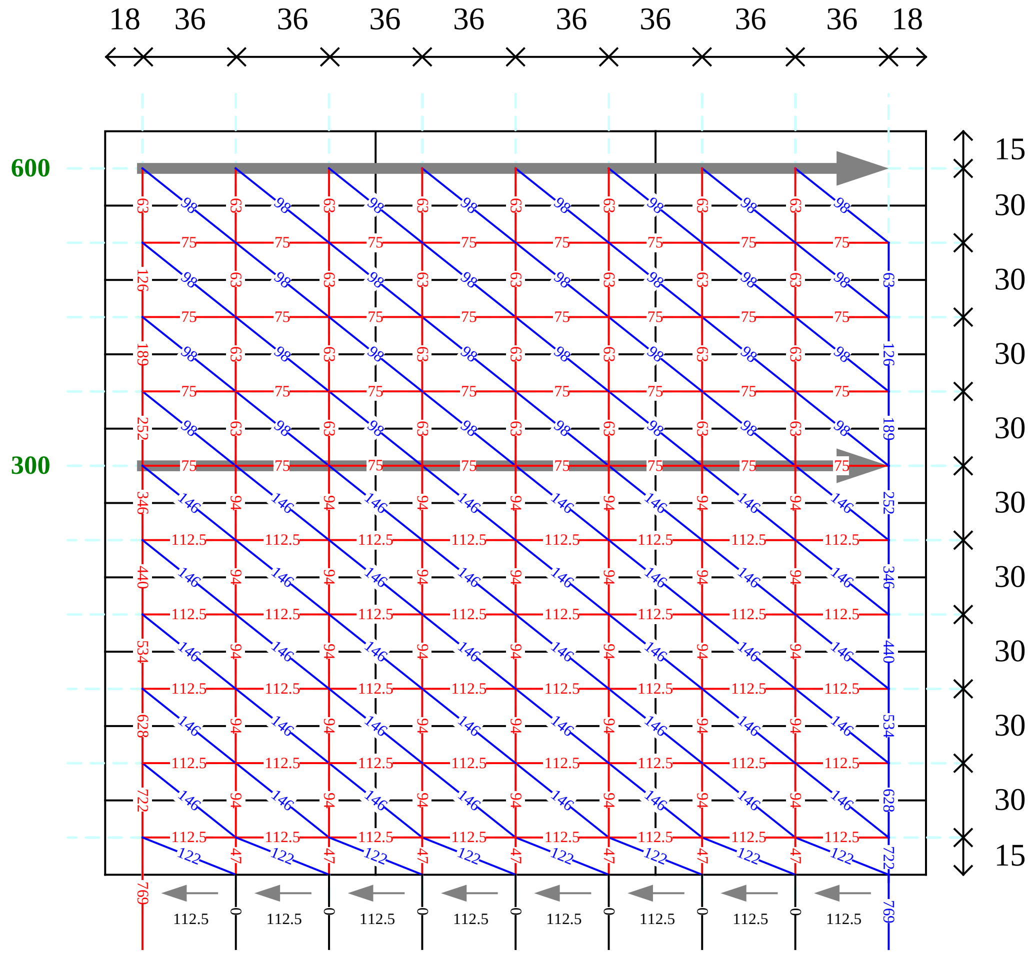
(21) Check the RRW using strut and tie models:

Assume the strut angle $25^\circ < \theta < 65^\circ$ (ACI 318-05), and three stacks of blocks work as a single block together. Then we can calculate $\theta = \tan^{-1}(30/36) = 40^\circ$. The width of compression strut is: $30 \cdot \cos(\tan^{-1}(30/36)) = 23.1''$ and the thickness of the RRW is about 18 inch. Then:

$$23.1'' \cdot 18'' \cdot 0.85 f_c' = 2827\, kip\, (CCC\ node)$$

$$23.1'' \cdot 18'' \cdot 0.65 f_c' = 2162\, kip\, (CTT\ node)$$

From Fig. A.18, all the node forces are less than 800 kips. This RRW wall can sustain the applied forces.



(Check equilibrium, OK!)

Figure A.18 The RRW strut and tie model

(22) Check the strong floor punching shear per hole: (Meyer, 1996)

Refer to Fig. A.19, without considering the contribution of shear reinforcement in the floor and assume three different possible cases.

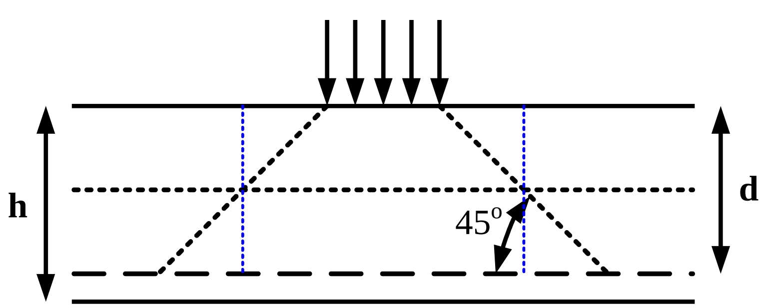


Figure A.19 The sketch of punching shear mechanism

(a) Assume 9 in by 9 in square plate as a washer:

$$h = 24\,in;\, d = h - 1.5 = 22.5\,in$$

$$V_c = \min[\ (2 + \frac{4}{\beta_c})\sqrt{f_c^{'}}b_o d,\ (2 + \frac{\alpha_s d}{b_o})\sqrt{f_c^{'}}b_o d,\ 4\sqrt{f_c^{'}}b_o d\ \ ]$$

$$where\ \ \beta_c = \frac{9}{9} = 1,\ \alpha_s = 40,\ b_o = (9 + 22.5) \times 4 = 126\,in$$

$$f_c^{'} = 4500\ psi$$

$$V_c = \min[\ 6\sqrt{f_c^{'}}b_o d,\ 9.14\sqrt{f_c^{'}}b_o d,\ 4\sqrt{f_c^{'}}b_o d\ \ ]$$

$$= 4\sqrt{f_c^{'}}b_o d = 4\sqrt{4500} \times 126 \times 22.5 = 760.7\ \ kips$$

$$\phi = 0.85 \Rightarrow V_u = 0.85 \times 760.7 = 646.6\ \ kips$$

(b) Assume 7 in by 5 in square plate as a washer:

$$h = 24\,in;\, d = h - 1.5 = 22.5\,in$$

$$V_c = \min[\ (2 + \frac{4}{\beta_c})\sqrt{f_c^{'}}b_o d,\ (2 + \frac{\alpha_s d}{b_o})\sqrt{f_c^{'}}b_o d,\ 4\sqrt{f_c^{'}}b_o d\ \ ]$$

$$where\ \ \beta_c = \frac{7}{5} = 1.4,\ \alpha_s = 40,\ b_o = (7 + 5 + 22.5 \times 2) \times 2 = 114\,in$$

$$f_c^{'} = 4500\ psi$$

$$V_c = \min[\ 4.8\sqrt{f_c^{'}}b_o d,\ 9.9\sqrt{f_c^{'}}b_o d,\ 4\sqrt{f_c^{'}}b_o d\ \ ]$$

$$= 4\sqrt{f_c^{'}}b_o d = 4\sqrt{4500} \times 114 \times 22.5 = 688.3\ \ kips$$

$$\phi = 0.85 \Rightarrow V_u = 0.85 \times 688.3 = 585.1\ \ kips$$

(c) Assume no washer: (Note: the strong floor has 2.5 in inside diameter holes 3 ft on center each direction)

$$h = 24\,in;\, d = h - 1.5 = 22.5\,in$$

$$r = \frac{2.5}{2} = 1.2\,in$$

$$V_c = \min[\ (2 + \frac{4}{\beta_c})\sqrt{f_c^{'}}b_o d,\ (2 + \frac{\alpha_s d}{b_o})\sqrt{f_c^{'}}b_o d,\ 4\sqrt{f_c^{'}}b_o d\ \ ]$$

$$where\ \ \beta_c = 1,\ \alpha_s = 40,\ b_o = 2 \cdot \pi \cdot (r + \frac{d}{2}) = 78.2\,in$$

$$f_c^{'} = 4500\ psi$$

$$V_c = \min[\ 6\sqrt{f_c^{'}}b_o d,\ 13.5\sqrt{f_c^{'}}b_o d,\ 4\sqrt{f_c^{'}}b_o d\ \ ]$$

$$= 4\sqrt{f_c^{'}}b_o d = 4\sqrt{4500} \times 78.2 \times 22.5 = 472.1\ \ kips$$

$$\phi = 0.85 \Rightarrow V_u = 0.85 \times 472.1 = 401.3\ \ kips$$

From the calculations shown above, the punching shear will not govern in this case.

(23) Check the local moment and shear in the two-cell box girder:

Using SAP2000 (Computers and Structures, Inc., 2005) to analysis a 2-D frame cutting transversely from the two-cell box girder. Fig. A.20 shows the 2-D model, bending moment diagram and element forces for loading condition seven in SAP2000. For checking purpose, total ten loading conditions are selected and demonstrated in Fig. A.22. Note that each loading point has 100 kips concentrate force. Notations for Member and Joint End Forces are shown in Fig. A.21. For a 3-ft-wide strip (using concept of tributary area), under ten different loading conditions on the slab, the maximum member forces and joint end forces are summarized in Table A.6.

The bending moment and shear capacity at several locations in the frame model are calculated and briefly described below:

For tension capacity:

$$T_n = (0.1 \cdot 4500) \cdot (3' \cdot 12) \cdot 16'' = 259.2\ kips$$

(did not consider the contribution of reinforcements in the rib)

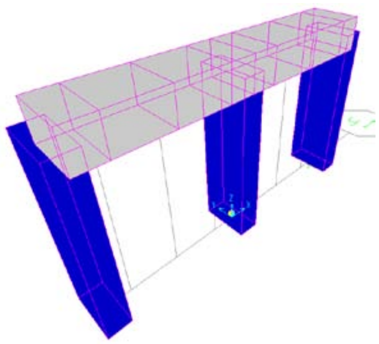
For shear capacity:

$$V_n = (3 \cdot \sqrt{4500}) \cdot (3' \cdot 12) \cdot 24'' = 173.9\ kips$$

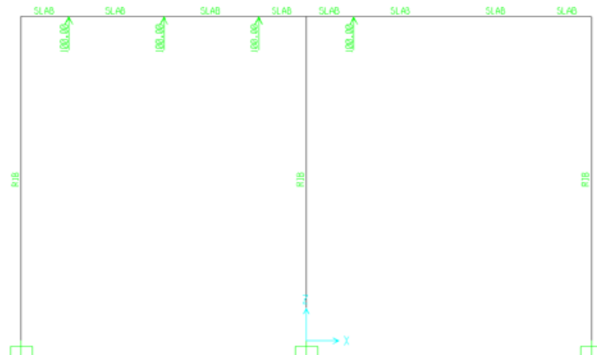
(the shear strength of concrete is somewhere between $1.9\sqrt{f_c^{'}}$ and $3.5\sqrt{f_c^{'}}$, did not consider the contribution of shear reinforcements in the slab)

For joint moment capacity (derived from joint shear capacity):

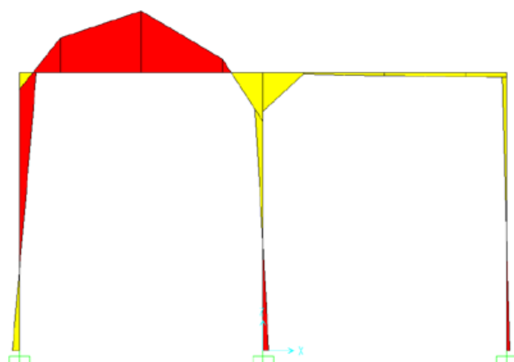
$$M_n = [(3 \cdot \sqrt{4500}) \cdot (3' \cdot 12) \cdot 16''] \times (24'' - 3'' - \frac{24''/3}{3}) = 177.1\ kip - ft$$



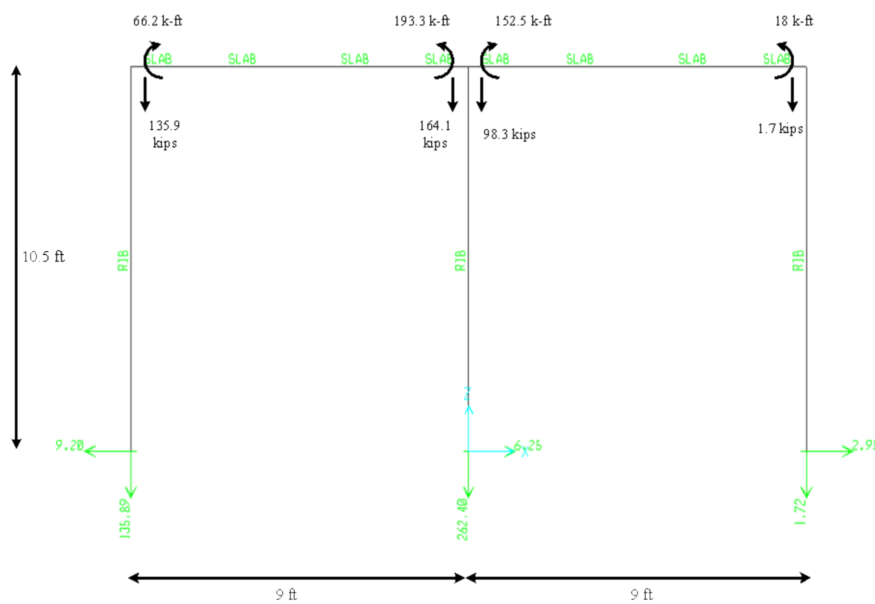
(a) 2-D model



(b) Joint external loads



(c) Bending moment diagram on tension side



(d) Element forces

Figure A.20 Analytical model in SAP2000

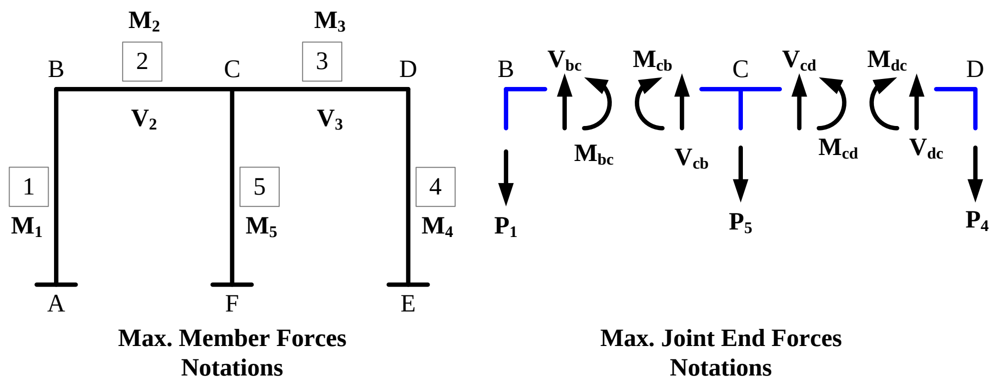


Figure A.21 Notations for member and joint end forces

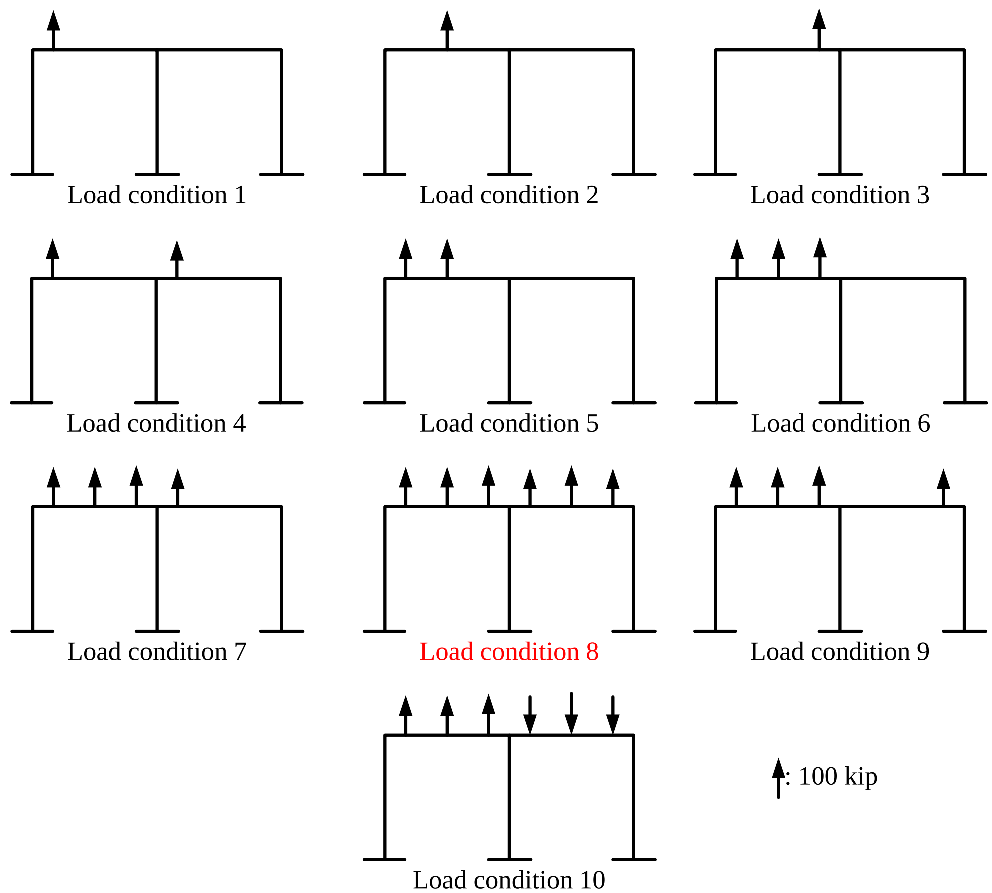


Figure A.22 Load conditions and load patterns
(note that each loading point has 100 kips concentrate force)

Table A.6 Maximum member forces and joint end forces

Load Conditions	1	2	3	4	5	6	7	8	9	10
M_1	18.3	30.3	16.6	19.3	48.7	65.2	66.2	66.6	65.7	63.8
M_2	102.9	169.7	85.8	97.3	215	259.8	245.3	273.9	246.0	314.9
V_2	80.8	55.5	86.4	77.8	125.3	161.1	164.1	173.0	164.0	150.8
M_3	26.9	53.7	28.1	70.7	80.6	108.8	152.2	273.9	150.0	314.9
V_3	2.9	6.0	3.0	89.3	8.9	11.9	98.3	173.0	68.9	150.8
M_4	2.0	1.9	1.0	17.1	3.8	4.3	18.0	66.6	19.7	63.8
M_5	14.2	26.5	15.6	1.3	40.8	56.3	40.8	0	42.1	112.7
M_{bc}	18.3	30.3	16.6	19.3	48.7	65.2	66.2	66.6	65.7	63.8
M_{cb}	41.2	80.2	43.7	69.3	121.4	165.1	193.3	273.9	192.1	56.3
M_{cd}	26.9	53.7	28.1	70.7	80.6	108.8	152.5	273.9	150.0	-56.3
M_{dc}	0.5	-0.2	1.0	17.1	0.4	1.4	18.0	66.6	19.7	-63.8
V_{bc}	80.8	44.5	13.7	77.8	125.3	138.9	135.9	127.0	136.0	150.8
V_{cb}	19.2	55.5	86.4	22.2	74.8	161.1	164.1	173.0	164.0	149.2
V_{cd}	2.9	6.0	3.0	89.3	80.6	11.9	98.3	173.0	31.1	-149.2
V_{dc}	-2.9	-6.0	-3.0	10.7	-8.9	-11.9	1.7	127.0	68.9	-150.8
P_1	80.8	44.5	13.7	77.8	125.3	138.9	135.9	127.0	136.0	150.8
P_5	22.2	61.5	89.4	111.5	83.7	173.3	262.4	346.0	195.2	0
P_4	-2.9	-6.0	-3.0	10.7	-8.9	-11.9	1.7	127.0	68.9	-150.8

(unit: kip-ft and kip)

For moment capacity:

A moment-curvature relationship for the section is developed and the moment corresponds to the extreme concrete compression strain equal to 0.05% is selected as the elastic limit. From Mander's concrete model (Mander et. al, 1988), the 0.05% strain in concrete corresponds to 1892 psi, which is $0.42 f_c^{'}$ in this case. This means the section behaves essentially elastic (see Figs. A.23 and A.24) although the bending moment exceed cracking moment. Figs. A.25 and A.26 show the moment curvature relationships for slab strip section and rib strip section, respectively. Table A.7 summaries the safety factors of the sections under different loading conditions in different stage of behaviors. Table A.8 shows the safety factors of the sections correspond to sectional elastic limit under different loading conditions.

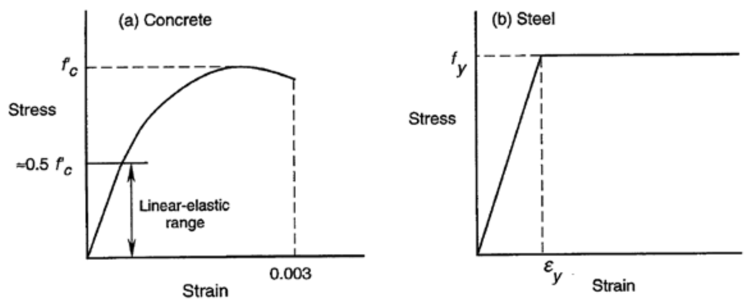


Figure A.23 Stress-strain relations for concrete and steel rebar (Meyer, 1996)

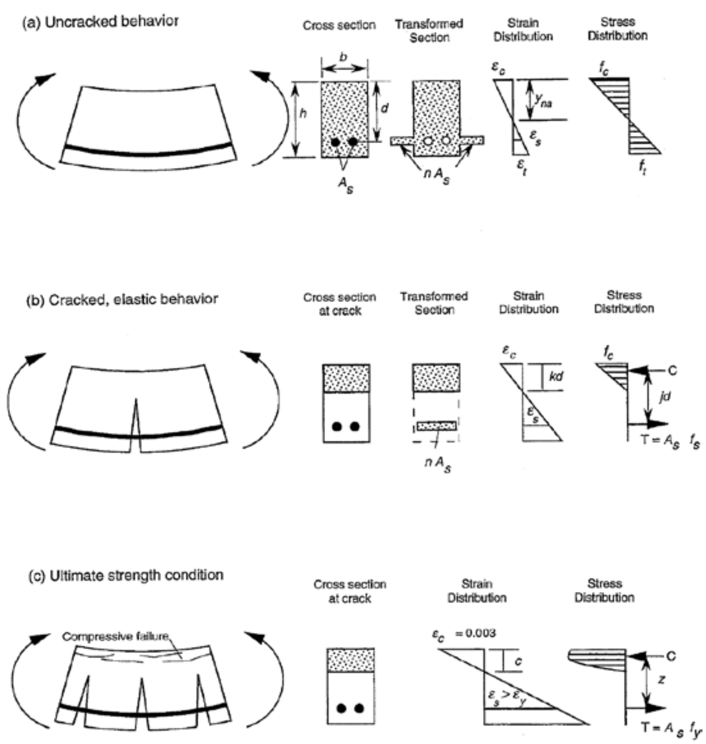


Figure A.24 Reinforced concrete beam behavior in different stages (Meyer, 1996)

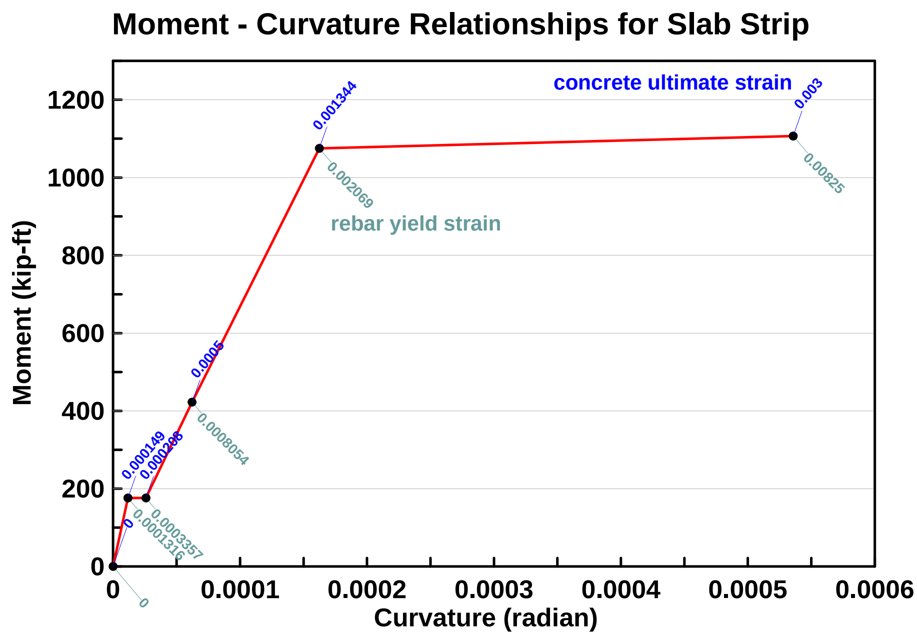


Figure A.25 Moment-curvature relationships for slab strip

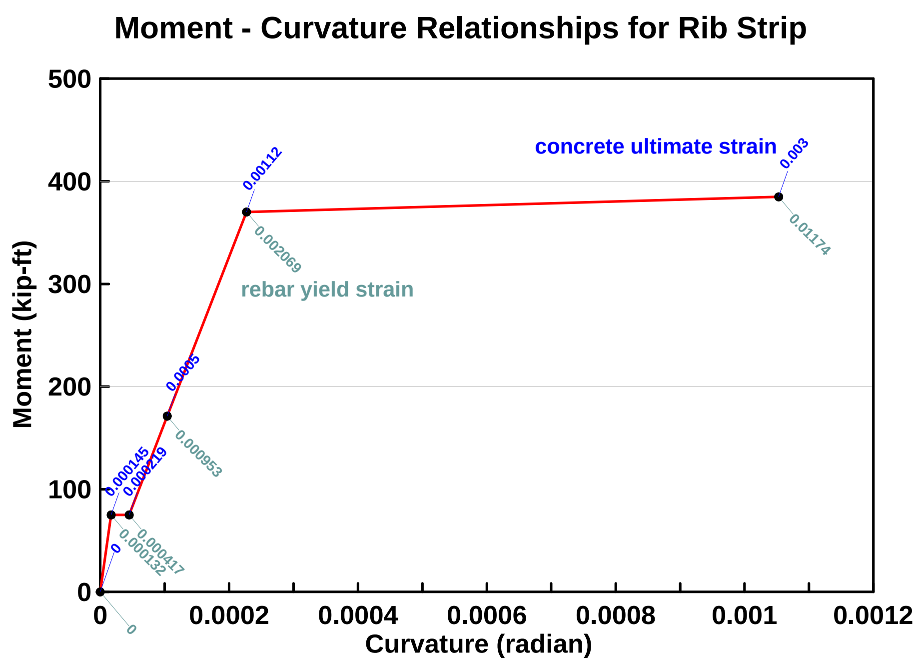


Figure A.26 Moment-curvature relationships for rib strip

Table A.7 Element and joint capacity vs. safety factors under different load conditions

Load Conditions		1	2	3	4	5	6	7	8	9	10
Capacity		Safety Factors (SF)									
M_1	74.9 (cracking)	4.09	2.47	4.51	3.88	1.54	1.15	1.13	1.12	1.14	1.17
	171.2 (elastic limit)	9.36	5.65	10.3	8.87	3.52	2.63	2.59	2.57	2.61	2.68
	384.8 (ultimate)	21.0	12.7	23.2	19.9	7.90	5.90	5.81	5.78	5.86	6.03
M_2	176.1 (cracking)	1.71	1.04	2.05	1.81	0.82	0.68	0.72	0.64	0.72	0.56
	422.5 (elastic limit)	4.11	2.49	4.92	4.34	1.97	1.63	1.72	1.54	1.72	1.34
	1106.6 (ultimate)	10.8	6.52	12.9	11.4	5.15	4.26	4.51	4.04	4.50	3.51
V_2	173.9	2.15	3.13	2.01	2.24	1.39	1.08	1.06	1.01	1.06	1.15
M_3	176.1 (cracking)	6.55	3.28	6.27	2.49	2.18	1.62	1.16	0.64	1.17	0.56
	422.5 (elastic limit)	15.7	7.87	15.0	5.98	5.24	3.88	2.78	1.54	2.82	1.34
	1106.6 (ultimate)	41.1	20.6	39.4	15.7	13.7	10.2	7.27	4.04	7.38	3.51
V_3	173.9	60.0	29.0	58.0	1.95	19.5	14.6	1.77	1.01	2.52	1.15
M_4	74.9 (cracking)	37.5	39.4	74.9	4.38	19.7	17.4	4.16	1.12	3.80	1.17
	171.2 (elastic limit)	85.6	90.1	171	10.0	45.1	39.8	9.51	2.57	8.69	2.68
	384.8 (ultimate)	192	203	385	22.5	101	89.5	21.4	5.78	19.5	6.03
M_5	74.9 (cracking)	5.27	2.83	4.80	57.6	1.84	1.33	1.84	-	1.78	0.66
	171.2 (elastic limit)	12.1	6.46	11.0	132	4.20	3.04	4.20	-	4.07	1.52
	384.8 (ultimate)	27.1	14.5	24.7	296	9.43	6.83	9.43	-	9.14	3.41
M_{bc}	177.1	9.68	5.84	10.7	9.18	3.64	2.72	2.68	2.66	2.70	2.78
M_{cb} (unbalanced)	177.1	12.4	6.70	11.4	127	4.30	3.10	4.30	-	4.20	1.57
M_{cd} (unbalanced)	177.1	12.4	6.70	11.4	126.5	4.30	3.10	4.30	-	4.20	1.57
M_{dc}	177.1	354	886	177	10.4	443	126	9.84	2.66	8.99	2.78
V_{bc}	173.9	2.15	3.91	12.7	2.24	1.39	1.25	1.28	1.37	1.28	1.15
V_{cb}	173.9	9.06	3.13	2.01	7.83	2.32	1.08	1.06	1.01	1.06	1.17
V_{cd}	173.9	59.9	28.9	57.9	1.95	2.16	14.6	1.77	1.01	5.59	1.17
V_{dc}	173.9	59.9	28.9	57.9	16.3	19.5	14.6	102	1.37	2.52	1.15
P_1	259.2	3.21	5.82	18.9	3.33	2.07	1.87	1.91	2.04	1.91	1.72

P_5	259.2	11.7	4.21	2.90	2.32	3.10	1.50	1.0	0.75	1.33	-
P_4	259.2	89.4	43.2	86.4	24.2	29.1	21.8	153	2.04	3.76	1.72
Min. (SF)	**-**	**1.71**	**1.04**	**2.01**	**1.81**	**0.82**	**0.68**	**0.72**	**0.64**	**0.72**	**0.56**

(unit: kip-ft and kip)

Table A.8 Minimum Safety Factors for each Load Conditions

Load Conditions	1	2	3	4	5	6	7	8	9	10
	Safety Factors (SF)									
Min. (SF)*	2.15	2.49	2.01	1.95	1.39	1.08	1.06	**0.75**	1.06	1.15
Critical Location	V_2	M_2	V_2	V_3	V_2	V_2	V_2	P_5	V_2	V_2

*Note: the safety factors correspond to the cross section elastic limit.

From the tables show above, loading condition 8 will govern under these ten loading conditions.

(24) Check the load conditions for different RRW configurations under applied overturning moment on the slab:

Assume the applied overturning moment is 600 kip x 20 ft + 300 kip x 10 ft = 15000 k-ft. From previous results we can derive the load combinations acting on the frame model as shown in Fig. A.27. Note that some reaction forces are extracted from SAP2000 analysis results under 800 kips uplifting force at one end of floor beam and made an assumption of providing four stiff load transfer beam below the floor slab longitudinally at both ends of floor beam. Thus:

$$\frac{1}{3}(118.3 + 266.1 + 80.2) = 155\,kips$$

$$\frac{1}{3}(73.5 + 178.5 + 39.6) = 97.2\,kips$$

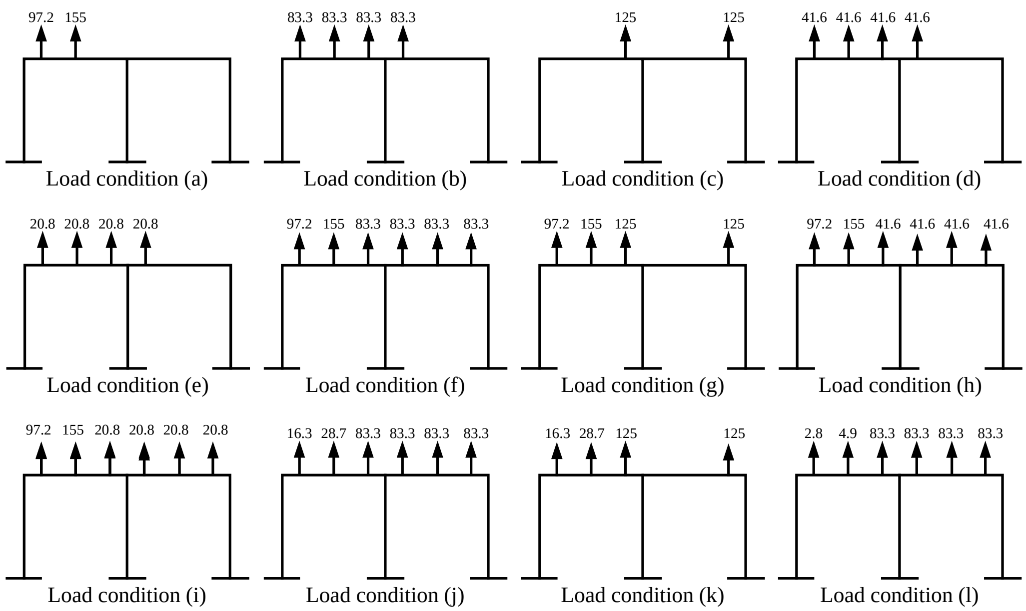


Figure A.27 Possible loading combinations (unit: kip)

From the results shown in Table A.9 and Table A.10, the option (i) is selected as the final configuration. In actual case, the concentrated uplifting force acting on the floor slab will lower than 800 kips and more close to 675 kips:

$$P_{uplift} = (600\,kips \cdot 18\,ft + 300\,kips \cdot 9\,ft)/20\,ft = 675\,kips$$

$$S.F. = 1.06 \times \frac{800}{675} = 1.26$$

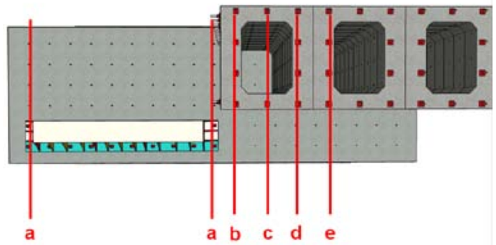
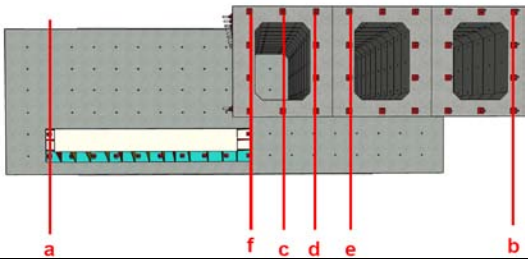
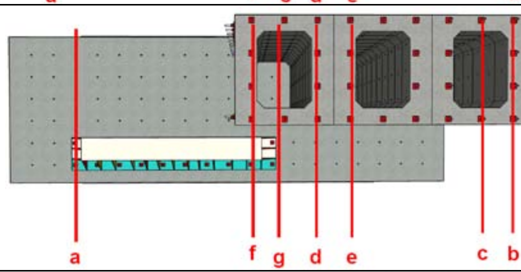
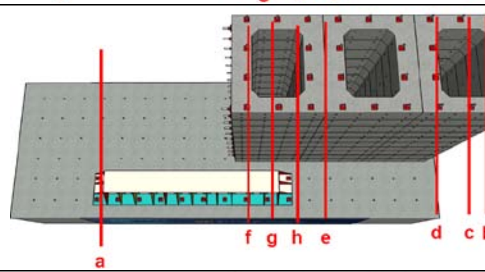
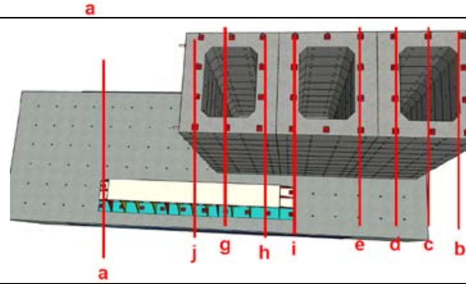
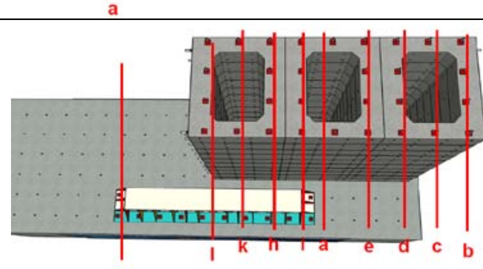
The actual safety factor for option (i) will be at least 1.26, conservatively.

Table A.9 Safety factors for possible loading combinations

Load Conditions	a	b	c	d	e	f	g	h	i	j	k	l
	Safety Factors (SF)											
Min. (SF)*	1.06	1.19	1.56	2.34	4.74	0.77	0.8	0.97	1.01	1.08	1.33	1.17
Critical Location	V_2	P_5	V_3	P_5	P_5	P_5	V_{cb}	V_2	V_2	P_5	V_{cb}	P_5

*Note: the safety factors correspond to the cross section elastic limit.

Table A.10 Critical safety factors for different configurations

Configuration	Plan view & Load Conditions	Min. SF & Critical Loaction		
(i) Shift 6 ft outward		**a**	**1.06**	**V_2**
(ii) Shift 3 ft outward		**f**	**0.77**	**P_5**
(iii) Current position		**f**	**0.77**	**P_5**
(iv) Shift 3 ft toward		**f**	**0.77**	**P_5**
(v) Shift 6 ft toward		**g**	**0.80**	**V_{cb}**
(vi) Shift 9 ft toward		**h**	**0.97**	**V_2**

A.3.2. Check the floor beam above the test slab

The floor beam is selected to be modified from the existing floor beam in Davis Hall laboratory. Total twenty anchor holes are available to mount to the strong floor in the lab.

$$R = 0.5 \times (20 \times 100\, kips) = 1000\, kips > 900\, kips$$

Thus, it is suggested to prestress the all-thread-bars to at least 100 kips. Simple calculation of the uplifting force per side:

$$P_{uplift} = (18\, ft \times 600\, kips + 9\, ft \times 300\, kips) / 20\, ft = 675\, kips$$

The entire floor beam is modeled using shell elements in SAP2000, Figs. A.28 and A.29 illustrate the mesh distribution and the boundary condition settings in the model. Conservatively using 800 kips as the uplift force transfers to the floor beam in SAP model. The von Mises stress distribution in the floor beam under 800 kips uplift force is shown in Figs. A.30 and A.31. The maximum von Mises stress is about 27 ksi. The distribution of reaction forces under 800 kips uplift force is superimposed on the stress distribution as shown in Fig. A.32. Maximum reaction force is 135 kips. If using hinge supports in the SAP model, which represents the extreme case, the maximum reaction force is 266 kips as shown in Fig. A.33.

Note that the existing floor beam in Davis Hall is weight about 22.5 kip. After welding the stiffeners and plates aside the flange, the estimated weight is about 30.3 kip. The overhead traveling crane in the Richmond Field Station structural lab (i.e. NEES Berkeley lab) has about 26270 lb (117 kN, 12 US-ton) capacity.

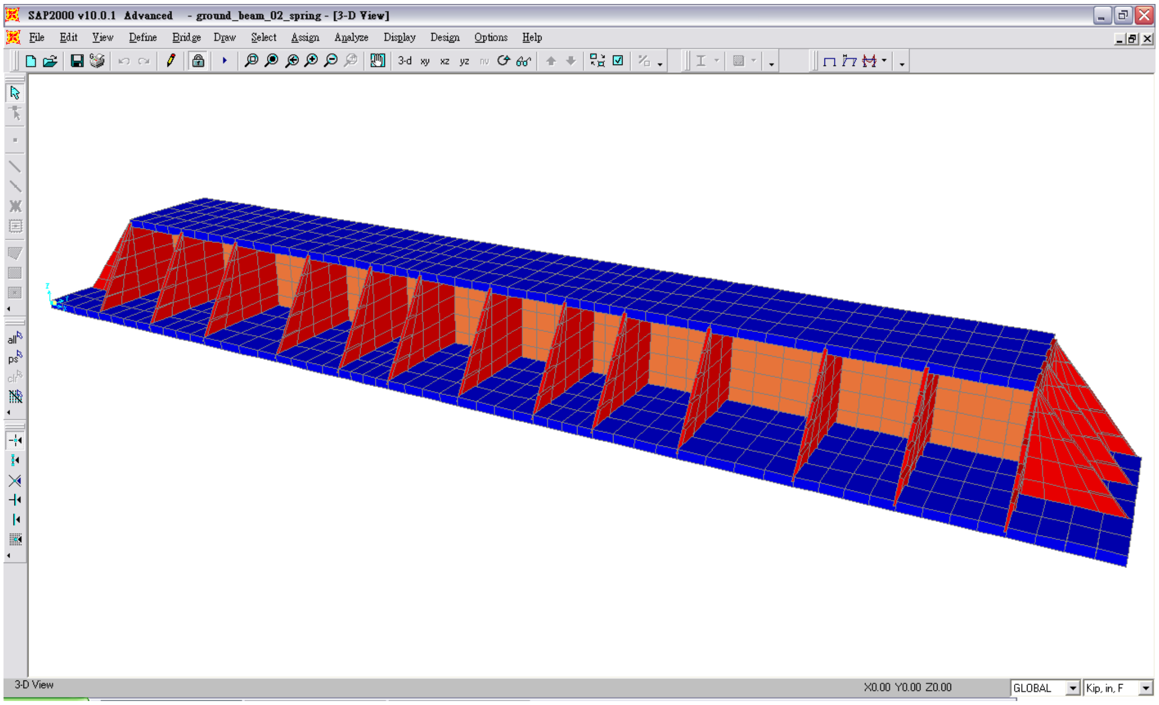


Figure A.28 The SAP2000 shell elements model for floor beam

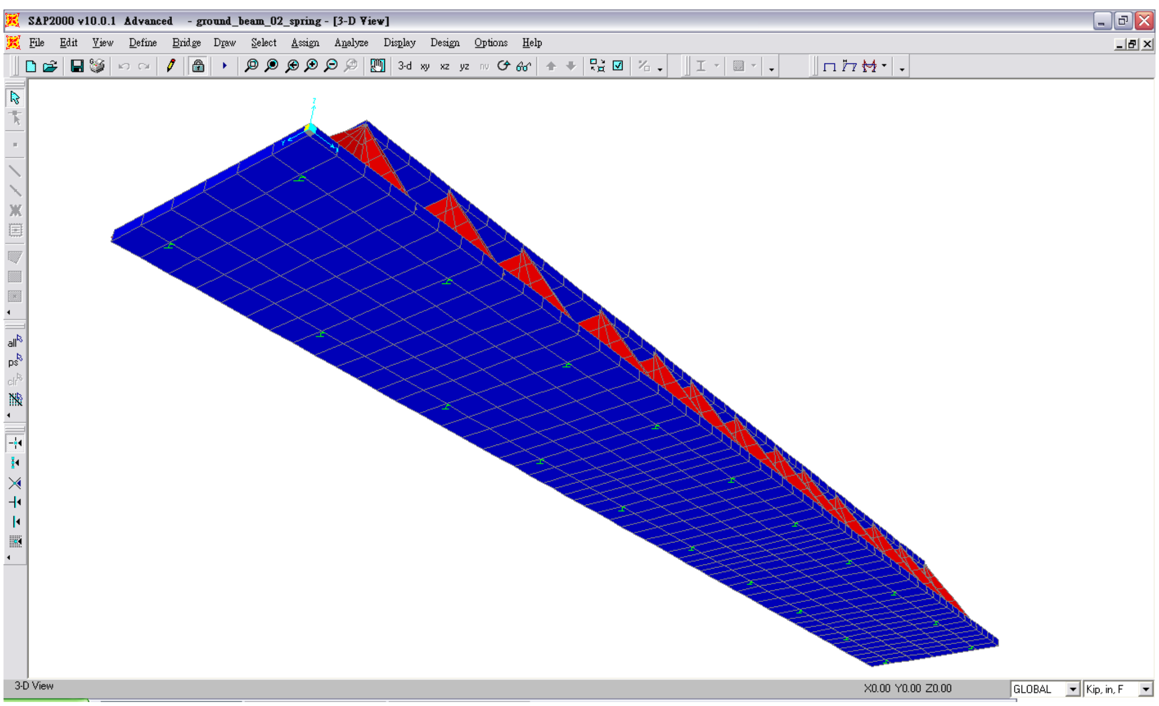


Figure A.29 Spring boundary condition used in the SAP2000 model

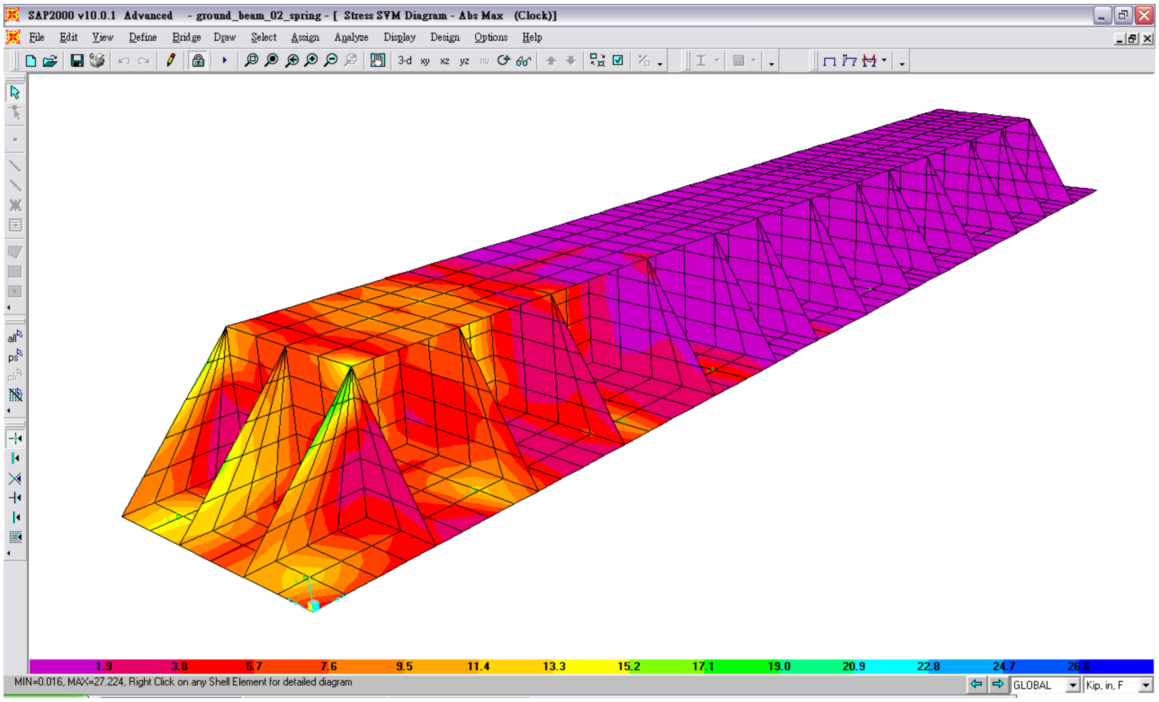


Figure A.30 The von Mises stress distribution in the floor beam under 800 kips uplift force (averaged stress, view from top)

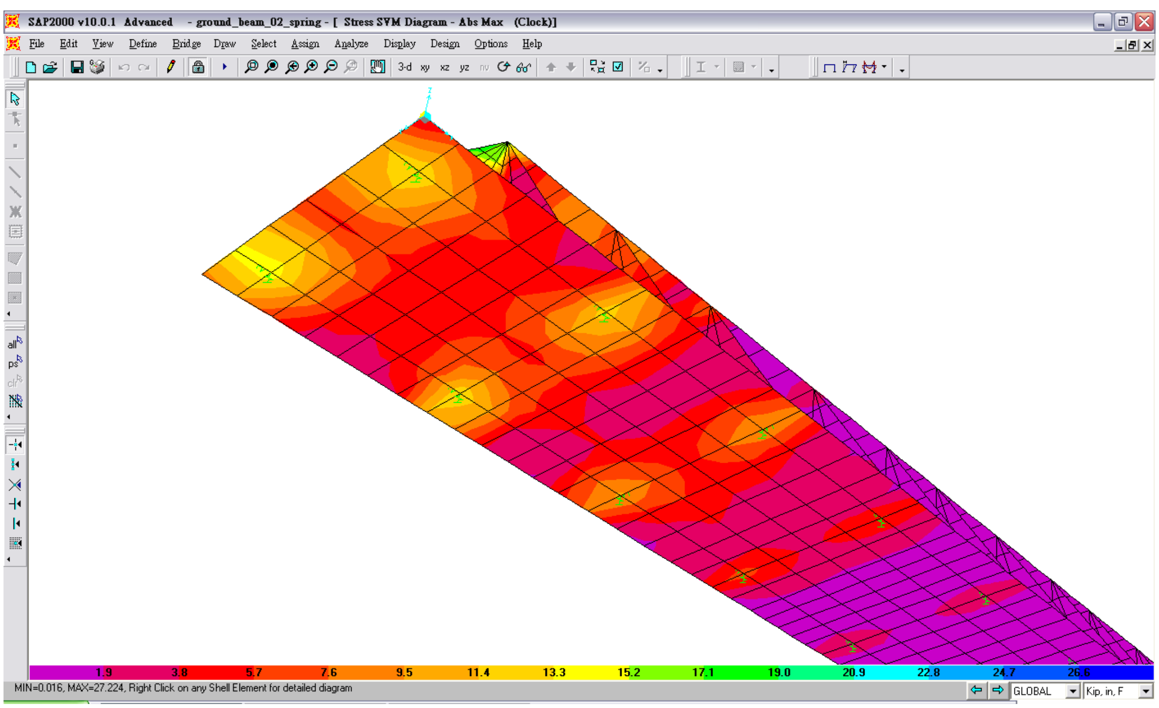


Figure A.31 The von Mises stress distribution in the floor beam under 800 kips uplift force (averaged stress, view from bottom)

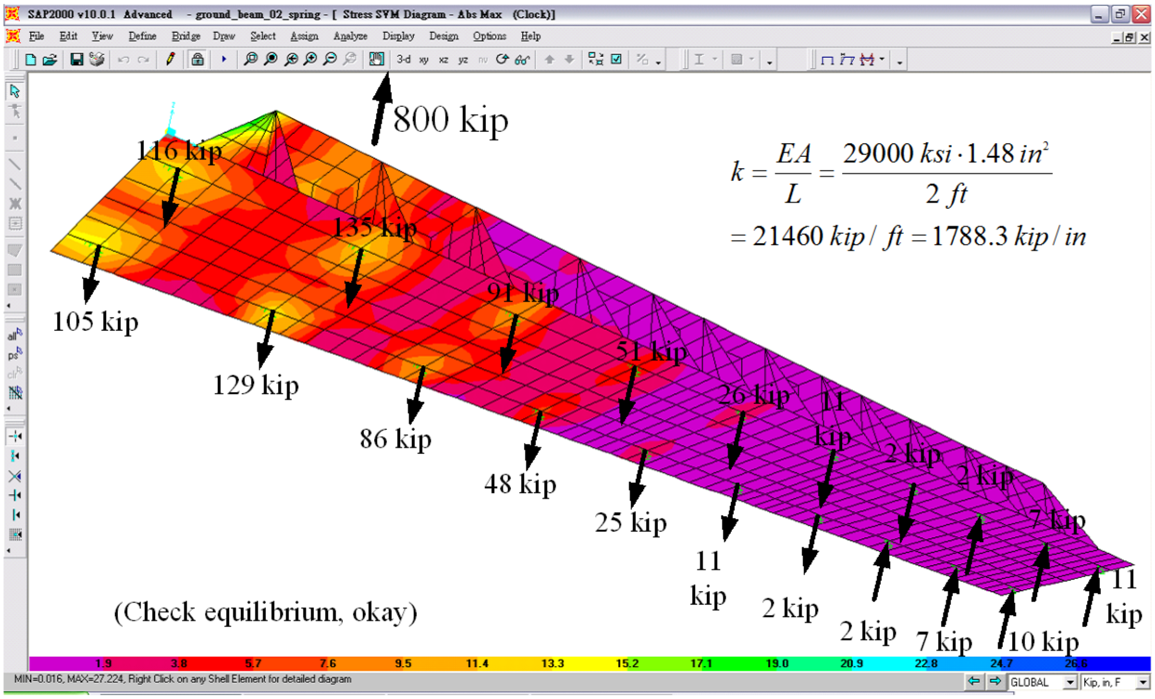


Figure A.32 The distribution of reaction forces under 800 kips uplift force (spring supports)

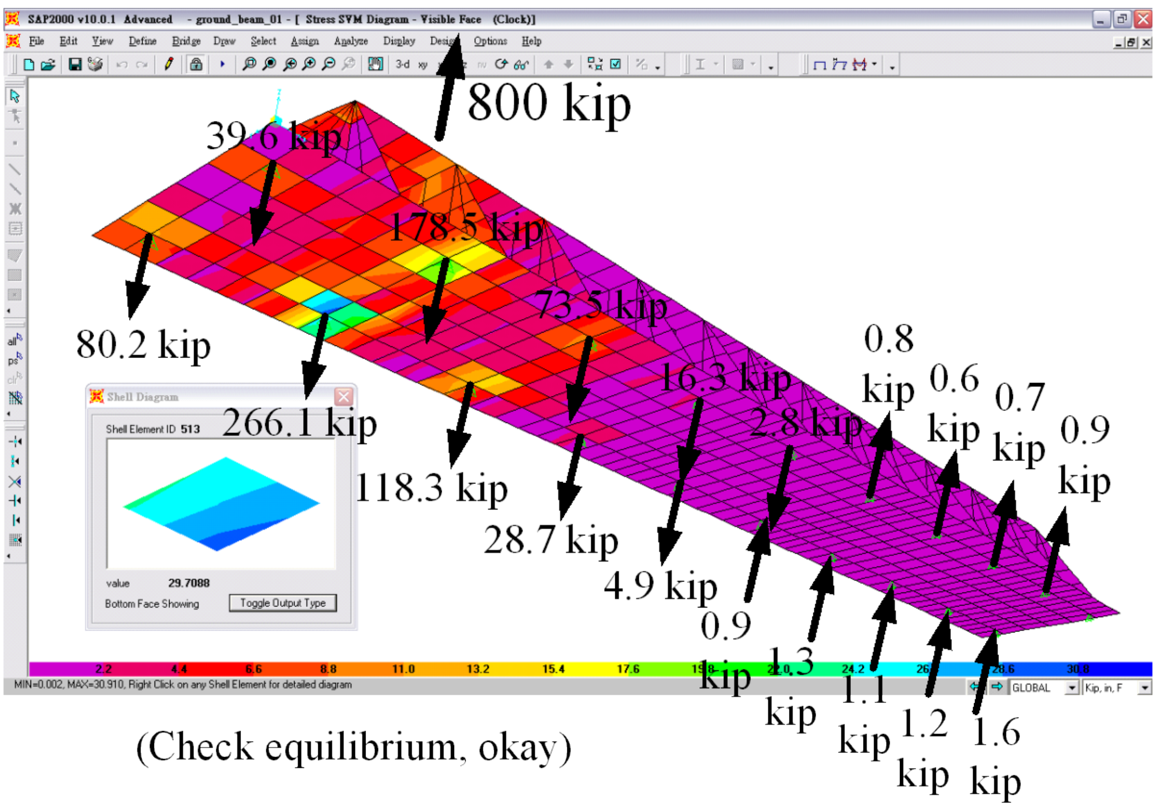


Figure A.33 The distribution of reaction forces under 800 kips uplift force (hinge supports)

Now we consider all-thread-bar, floor beam base plate, under-floor beam and concrete slab in series since they are gripped together. Based on the assumption in mechanical design handbook (Shigley, 1972; Norton 2006), the portion of total external load on post tensioned assembly taken by all-thread-bar and members (i.e. ground beam base plate, under-floor beam and concrete slab) as well as the resultant loads in all-thread-bar and the resultant loads on members can be calculated. Table A.11 shows the safety factors for yielding and separation in the assembly under different external loads and different all-thread-bar diameters.

Table A.11 Safety factors for yielding and separation in the assembly

Nominal Diameter (in)	1-3/8	1-3/8	1-3/4	1-3/4
Minimum Yield Strength (kips)	190	190	320	320
Pretention Load (kips)	140	140	200	200
External Load (kips)	131	224	131	224
P_b (kips)	29	50	44	75
P_m (kips)	102	174	87	149
Safety Factor for Yielding	1.12	0.99	1.31	1.16
Safety Factor for Separation	1.38	0.80	2.31	1.34

Note: (from Fig. A.33)

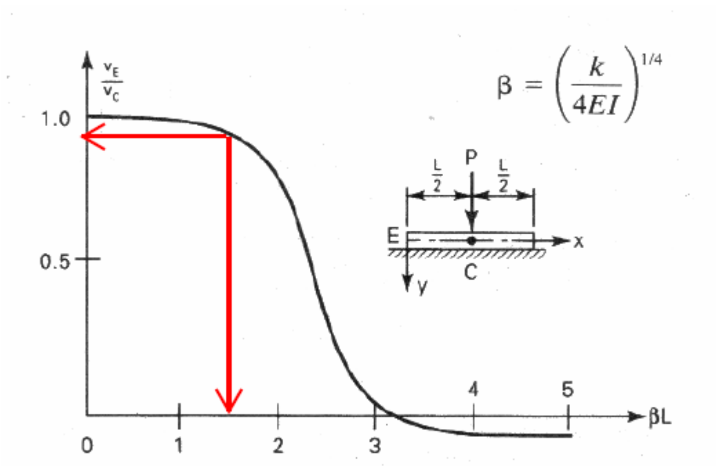
$$\frac{1}{3}(118.3+266.1+80.2)=155\,kips \quad \Rightarrow 155\cdot\frac{675}{800}=131\,kips$$

$$266.1\cdot\frac{675}{800}=224\,kips$$

From the calculation results shown above, using larger rod diameter with higher pretension load (i.e. 1-3/4” rod with 200 kips pretension load) can prevent separation of the assembly and reduce the probability to yield the post tension rods. This also increase the shear resistant on the floor beam to at least 1320 kips if the friction coefficient is taken as 0.33 for all 20 rods. Again, this is under a very conservative loading and analysis condition.

A.3.3. Design the floor beam below the test slab (the under-floor beam)

Now we need to find the required flexural stiffness of the beam under the floor slab to spread out the uplift force. From Ugural and Fenster “Advanced Strength and Applied Elasticity” Chapter 9 (Ugural and Fenster, 2004), we can find theoretical solutions for a finite beam sitting on an elastic foundation. In order to spread out uplift force to adjacent anchor points, we need a relatively rigid beam to achieve this goal. By comparing the center and end deflection (theoretically) of a finite beam on an elastic foundation subjected to a centrally concentrated load, the required flexural stiffness of the beam under the floor slab can be determined using the formula and figure provided in the book. Choose βL = 1.5 (see the left of Fig. A.34) and the beam length is equal to 72 inch (6 ft), and then we can determine the required moment of inertia of the beam which is 7564.5 in^4. This is closed to the moment of inertia of the AISC W24 x 229 section (I = 7650 in^4). The required moment of inertia for different beam length is listed on the right in Fig. A.34.



(a) Comparison of the center and end deformations of a finite length beam on an elastic support under a concentrated load at center (Ugural and Fenster, 2004)

βL	1.5	in^2
E	29000	ksi
k	165.3	ksi
L	72	in
I	7564.5	in^4

L	I
in	in^4
36	472.8
48	1494.2
60	3648.0
72	7564.5
84	14014.2
96	23907.5
108	38295.2
120	58368.0

(b) Calculation results

Figure A.34 Required moment of inertia for the under-floor beam

The selected W24 x 229 beam, both web and flange are compact and the material type is ASTM A572 Grade 50 steel. The following failure modes are checked:

(1) Web shear yielding: (no tension field action)

$$V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v = 0.6 \cdot 50 \cdot (26'' \cdot 0.96'') \cdot 1.0 = 749\, kips$$

$$\phi V_n = 0.9 \cdot 749 = 674\, kips > 320\, kips$$

(2) Web local yielding:

$$\text{Assume } N = 6"; k = 2.23" \Rightarrow N + 5k = 17.2" \Rightarrow \frac{N+5k}{2} = 8.6" > 12"$$

$$d = 26" > 12"$$

$$R_n = (2.5k + N) \cdot F_{yw} \cdot t_w = 556\,kips \Rightarrow \phi R_n = 1.0 \cdot 556 = 556\,kips > 320\,kips$$

(3) Web crippling:

$$\frac{d}{2} = 13" > 12"; \frac{N}{d} = \frac{6}{26} = 0.23 > 0.2$$

$$\Rightarrow R_n = 0.4 \cdot t_w^2 [1 + (\frac{4N}{d} - 0.2)(\frac{t_w}{t_f})^{1.5}] \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}} = 774\,kips$$

$$\Rightarrow \phi R_n = 0.75 \cdot 774 = 581\,kips > 320\,kips$$

(4) Buckling capacity of the $\phi = 3"$ extra-strong pipe welded to beam web:

$$P_{cr} = \frac{\pi^2 E \cdot I}{(kL)^2} = \frac{\pi^2 (29000ksi) \cdot 3.89in^4}{(1.0 \cdot 26")^2} = 1647\,kips > 320\,kips$$

(5) Buckling of stiffeners: (four stiffeners around each hole)

$$I = \frac{1}{12} \cdot (6\frac{1}{16}") \cdot 1^3 = 0.5\,in^4; L = 26"$$

$$P_{cr} = 4 \cdot \frac{\pi^2 E \cdot I}{(kL)^2} = 4 \cdot \frac{\pi^2 (29000ksi) \cdot 0.5in^4}{(1.0 \cdot 26")^2} = 847\,kips > 320\,kips$$

(6) Yielding of compression strut: (pipe plus four stiffeners around each hole)

$$A_{pipe} = 3.02\,in^2; A_{stiffeners} = 4 \cdot (6\frac{1}{16}" \cdot 1") = 24.25\,in^2$$

$$R_n = 50ksi \cdot (3.02 + 24.25) = 1364\,kips > 320\,kips$$

A.3.4. Check the lateral supporting frame

The SAP2000 model for the lateral supporting frame is shown in Fig. A.35. It is assumed that 5% of maximum actuator forces (for each floor level) acting on six supporting points.

$$F_1 = 300 \cdot 0.05 = 15\,kips$$

$$F_2 = 600 \cdot 0.05 = 30\,kips$$

Check the stress ratios directly in the SAP2000 model: $SR_{\max} = 0.72 < 1.0$

From static analysis, check the stress at the root of cantilever beam:

$$V = 123\,kips;\ \ M = 21900\,kip-in;\ \ T = 43.8\,kip-in$$

$$f_n = 26.2\,ksi\,(bending) + 3.34\,ksi\,(wraping) = 29.54\,ksi < 0.9 \cdot 36 = 32.4\,ksi$$

$$f_{v-web} = 4.94\,ksi\,(shear) + 0.15\,ksi\,(wraping) = 5.09\,ksi < 0.9 \cdot 0.6 \cdot 36 = 19.4\,ksi$$

$$f_{v-flange} = 1.1\,ksi\,(shear) + 0.09\,ksi\,(wraping) = 1.19\,ksi < 0.9 \cdot 0.6 \cdot 36 = 19.4\,ksi$$

Check the maximum lateral displacement:

$$\delta_{\max} = 1.36'' \Rightarrow \frac{1.36''}{(312 + 2 - 16 - 3.25)} = \frac{4.6}{1000} < \frac{5}{1000}$$

Providing a HSS 8 x 8 x 0.5 brace at the middle of cantilever beam, the maximum deflection will be much lower than this value. The stresses in the Tee sections, guider plate assemblies (saddles) and kickers are also checked briefly by hand.

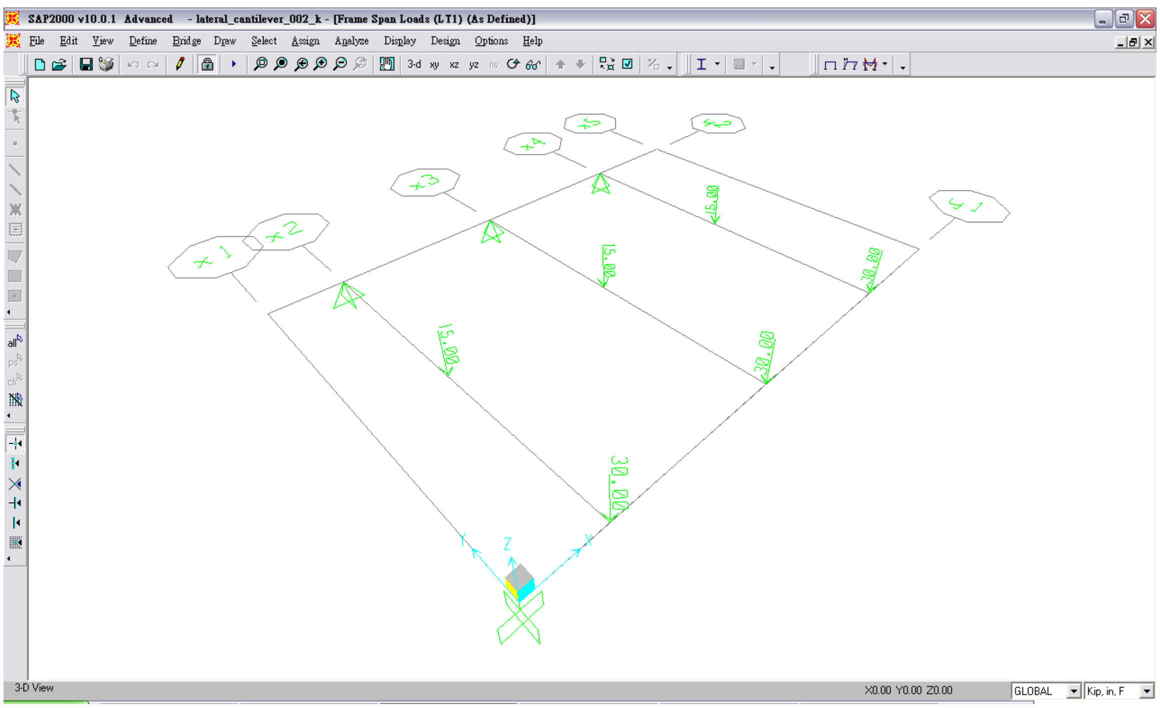


Figure A.35 The SAP2000 model for lateral supporting frame

A.3.5. Check the weight of 1.5 M-lb actuators

The total weight of a 1.5 M-lb actuator should be less than the capacity of bridge crane in NEES lab for setup installation. The overhead bridge crane capacity in the lab is about 26270 lb (117 kN, 12 US-ton). From Table A.12 and Fig. A.36, the total weight of 1.5 M-lb actuator is less than the crane capacity. Two W8 x 40 cantilever beams with 48 inches in length are provided to temporarily support the actuators during the specimen fabrication stage.

Table A.12 Detail lists of the 1.5 M-lb actuator parts

Actuator parts	Qty.	Weight (lb) (from drawings)	Weight (lb) (Approx. calculation)	Weight (lb) (from photos)
① Assembly	1	N.A.	< 12,000	14600 (①+③+⑥)
② Cap end mounting bracket	1	4,045	4,136	4100 (②)
③ Rod eye	1	1,100	1,153	-
④ Rod end mounting bracket	1	4,860	4,970	4860 (④)
⑤ Pin	2	672	673	1450 (⑤+⑦)
⑥ Load cell	1	175	207	-
⑦ Pin retainer plate	2	N.A.	6.8	-
Sum	**< 23,825 lb**			**25,010 lb**

Figure A.36 The 1.5 M-lb actuator assemblies in Richmond Field Station structural lab

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