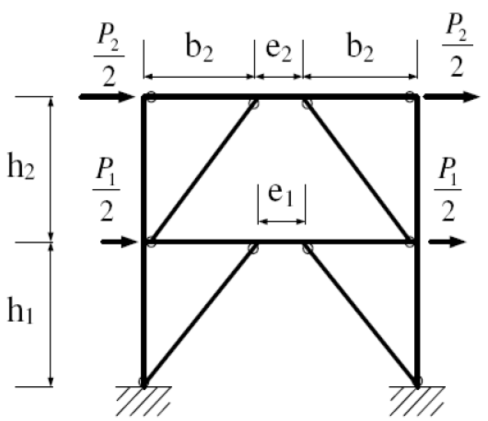


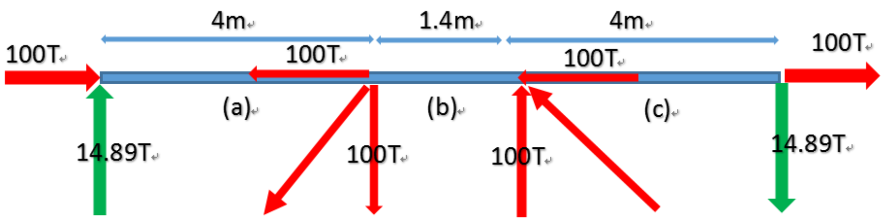
The link lengths, e1=e2=1.4m. The beam segment, b2=4.0m.
The story height, h1=h2=4.0m.
The upper beam is H500x300x12x26mm, and the lower beam H500x300xt_wxt_f mm. All member ends are pin connections. The LRFD seismic lateral forces, P2=200 tons, P1=100 tons. Assume all beams, columns are A572 GR50 (Fy=3.5 t/cm^2). Braces is A500B (Fy=3.5 t/cm^2). Assume beam segments outside the link are laterally supported at every 2 meters. Check the b/t ratios of the sections as you proceed.

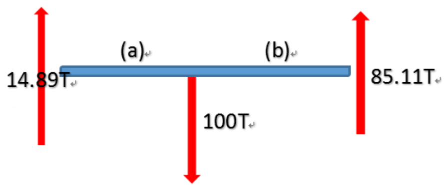
(1) Assume column base is pin connected, all lateral forces are resisted by the braces, check the shear force in the upper link by hand calculation to see if it exceeds the limit. Is the link in the upper floor beam a shear or flexural link?

答:

設左右兩端的接頭都為鉸接，且側向力皆由斜撐抵抗，則可視為一靜定結構，可得下圖。

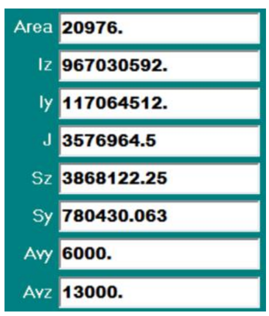
考慮力平衡如左圖所示得外力、反力及斜撐分力情形。





由簡易力平衡可得在 link(b)處的剪力內力為 85.11T。

由題意得頂層梁性質:H500X300X12X26mm 輸入 PISA3D 可以計算出斷面性質:



Area	20976.
Iz	967030592.
Iy	117064512.
J	3576964.5
Sz	3868122.25
Sy	780430.063
Avy	6000.
Avz	13000.

$$Z = 300\times500^2\times0.25-(300-12)\times(500-2\times26)^2\times0.25 = 4299312mm^3$$

$$M_p = ZF_y = 4299312\times0.001\times3.5 = 15047.6T-cm$$

另外

$$V_p = 0.6F_y(d-2t_f)t_w = 0.6\times3.5\times44.8\times1.2 = 112.9T$$

$$1.6\frac{M_p}{V_p} = 1.6\times\frac{15047.6}{112.9} = 213.25cm \geq e_2 = 140cm$$

因此為剪力型連桿

$\phi V_p = 0.9\times112.9 = 101.61T \geq 85.11T$ 故由此式得連桿沒有超過極限。

(2) Check the capacity design of the upper beam segment outside the link and report the demand-to-capacity ratio (DCR) of the beam segment outside the link.

$$DCR = \frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n}$$

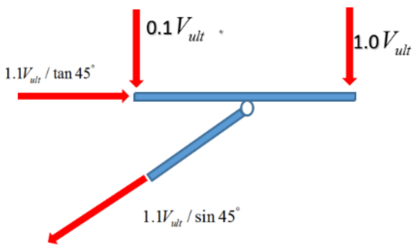
$$V_{ult} = 1.1 R_y V_p = 1.1 \times 1.1 \times 112.9 = 136.61T$$

$$M_a = M_b = \frac{V_{ult} \times e_2}{2} = 136.61 \times 140 / 2 = 9562.7T - cm = M_{ult}^{beam}$$

.

$$P_{ult}^{brace} = 1.1V_{ult} / \sin 45^{\circ} = 1.1 \times 136.61 / \sin 45^{\circ} = 212.515T$$

$$P_{ult}^{beam} = 1.1V_{ult} / \tan 45^{\circ} = 1.1 \times 136.61 / \tan 45^{\circ} = 150.271T$$



.軸力強度:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{117064512}{20976}} = 74.71mm = 7.471cm$$

$$\lambda = \frac{kl}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{1 \times 200}{7.471}\sqrt{\frac{3.5 \times 100 \times 100}{\pi^2 \times 2.04 \times 10^7}} = 0.353 \le 1.5$$

$$\phi P_n = \phi A_g F_{cr} = \phi A_g (0.658^{\lambda^2}) F_y = 0.85 \times 209.76 \times 0.658^{0.125} \times 3.5 \text{(壓)}$$

$$= 592.23T$$

$$\Rightarrow \phi P_n R_y = 592.23 \times 1.1 = 651.45T$$

$$R_y \phi P_n = R_y \phi A_g F_y = 0.9 \times 1.1 \times 209.76 \times 3.5 = 726.82T \qquad \text{(拉)}$$

抗彎強度:

Flange: $b / t = 150 / 26 = 5.55 \le \lambda_p = 16 / \sqrt{F_y} = 8.55$

Web : $b / t = 448 / 12 = 37.33 \le \lambda_p = 170 / \sqrt{F_y} = 90.87$

$$\Rightarrow M_n = M_p = ZF_y = 15047.6T - cm$$

$$R_y \phi M_n = 1.1 \times 0.9 \times 15047.6 = 14897.124T - cm$$

因此連桿外梁(a)段(左段) 受軸壓:

$$\frac{P_u}{\phi P_n} = \frac{150.271}{651.54} = 0.23 \ge 0.2$$

$$\rightarrow DCR = \frac{P_u}{\phi P_n} + \frac{8M_u}{9\phi M_n} = 0.23 + \frac{8 \times 9562.7}{9 \times 14897.124} = 0.8 \le 1$$

連桿外(b)段(右段) 受軸拉:

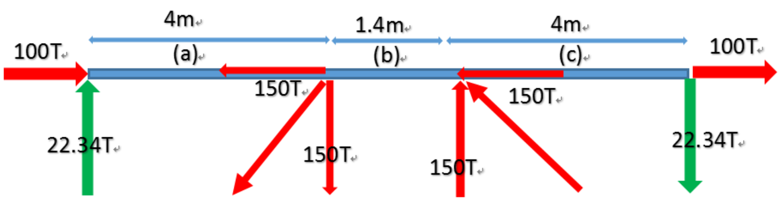
$$\frac{P_u}{\phi P_n}=\frac{150.271}{726.82}=0.2067\geq 0.2$$

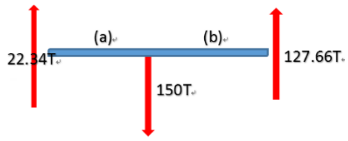
$$\rightarrow DCR=\frac{P_u}{\phi P_n}+\frac{8M_u}{9\phi M_n}=0.2067+\frac{8\times 9562.7}{9\times 14897.124}=0.777\leq 1$$

同第一題的假設求得反力及連桿中剪力

要設計一樓的梁符合相對長度小於 1.2

(3) Design the t_f and t_w for the lower beam so that it meets the requirements for a shear link and with a length no greater than $1.2M_p/V_p$. Check the DCR of the lower beam segment outside the link.





$$\frac{M_p}{V_p}=\frac{Z}{0.6(500-2t_f)t_w}=\frac{300\times 250^2-(300-t_w)(250-t_f)^2}{0.6(500-2t_f)t_w}\geq\frac{140}{1.2}=116.67$$

若令 $t_w=16mm$ 則 $t_f\geq 29.48$ m 且 $\phi V_p\geq 127.66T$ 所以選 *H500X300X16X30*

$$M_p=ZF_y=(300\times 250^2-(300-16)(250-30)^2)/1000\times 3.5=17515T-cm$$

$$V_p=0.6\times(500-2\times 30)\times 16\times 3.5/100=147.84T$$

$$1.2\frac{M_p}{V_p}=1.2\times\frac{17515}{147.84}=142.167cm\geq e_1=140cm$$

所以選此斷面後為剪力連桿

$$\phi V_p=0.9\times 147.84=133.056T\geq 127.66T$$

沒有超過極限。

Area	25040.
Iz	1108978688.
Iy	135150192.
J	5646786.5
Sz	4435914.5
Sy	901001.25
Avy	8000.
Avz	15000.

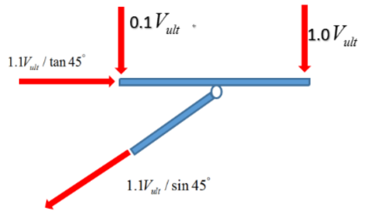
$$DCR = \frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n}$$

$$V_{ult} = 1.1 R_y V_p = 1.1 \times 1.1 \times 147.84 = 178.89T$$

$$M_a = M_b = \frac{V_{ult} \times e_2}{2} = 178.89 \times 140 / 2 = 12522.3T - cm = M_{ult}^{beam}$$

$$P_{ult}^{brace} = 1.1 V_{ult} / \sin 45^\circ = 1.1 \times 178.89 / \sin 45^\circ = 278.288T$$

$$P_{ult}^{beam} = 1.1 V_{ult} / \tan 45^\circ = 1.1 \times 178.89 / \tan 45^\circ = 196.799T$$



.軸力強度:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{135150192}{25040}} = 73.467mm = 7.3467cm$$

$$\lambda = \frac{kl}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{1 \times 200}{7.335}\sqrt{\frac{3.5 \times 100 \times 100}{\pi^2 \times 2.04 \times 10^7}} = 0.36 \le 1.5$$

$$\phi P_n = \phi A_g F_{cr} = \phi A_g (0.658^{\lambda^2}) F_y = 0.85 \times 250.4 \times 0.658^{0.1296} \times 3.5 \quad (壓)$$

$$= 705.6T$$

$$\Rightarrow \phi P_n R_y = 705.6 \times 1.1 = 776.17T$$

$$R_y \phi P_n = R_y \phi A_g F_y = 0.9 \times 1.1 \times 250.4 \times 3.5 = 867.636T \qquad (拉)$$

抗彎強度:

Flange: $b/t = 150/30 = 5 \le \lambda_p = 16/\sqrt{F_y} = 8.55$

Web: $b/t = 440/16 = 27.5 \le \lambda_p = 170/\sqrt{F_y} = 90.87$

$$\Rightarrow M_n = M_p = ZF_y = 17515T - cm$$

$$R_y \phi M_n = 1.1 \times 0.9 \times 17515 = 17339.85T - cm$$

因此連桿外梁(a)段(左段) 受軸壓:

$$\frac{P_u}{\phi P_n} = \frac{196.799}{776.17} = 0.2536 \ge 0.2$$

$$\rightarrow DCR = \frac{P_u}{\phi P_n} + \frac{8M_u}{9\phi M_n} = 0.2536 + \frac{8 \times 12522.3}{9 \times 17339.85} = 0.8955 \le 1$$

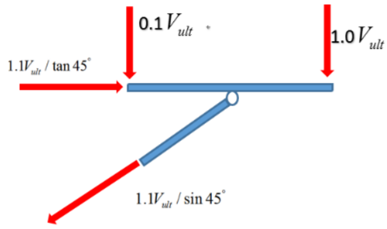
連桿外(b)段(右段) 受軸拉:

$$\frac{P_u}{\phi P_n} = \frac{196.799}{867.636} = 0.2268 \ge 0.2$$

$$\rightarrow DCR = \frac{P_u}{\phi P_n} + \frac{8M_u}{9\phi M_n} = 0.2268 + \frac{8 \times 12522.3}{9 \times 17339.85} = 0.8687 \le 1$$

(4) Select two different square hollow structural sections (HSS) for the braces in the upper and lower stories. Show DCRs and all the calculations to meet the capacity design requirements.

在上層中



$$V_p = 0.6F_y(d-2t_f)t_w = 0.6\times3.5\times44.8\times1.2 = 112.9T$$

$$V_{ult} = 1.25R_yV_p = 1.25\times1.1\times112.9 = 155.2375T$$

$$P_{ult}^{brace} = 1.1V_{ult}/\sin 45^{\circ} = 1.1\times155.2375/\sin 45^{\circ} = 241.5T$$

太難反推了，所以不斷的嘗試之後 選擇 *HSS280X280X10*

斜撐長為 $400\sqrt{2} = 565.69cm$

Area	10800.
Iz	131400000.
Iy	131400000.
J	196830000.
Sz	938571.438
Sy	938571.438
Avy	5600.
Avz	5600.

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{131400000}{10800}} = 110.3mm = 11.03cm$$

$$\lambda = \frac{kl}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{1\times565.69}{11.03}\sqrt{\frac{3.5\times100\times100}{\pi^2\times2.04\times10^7}} = 0.676 \le 1.5 \quad (壓)$$

$$\phi P_n = \phi A_g F_{cr} = \phi A_g(0.658^{\lambda^2})F_y = 0.85\times108\times0.658^{0.457}\times3.5$$
$$= 265.36T$$

$$DCR = \frac{P_u}{\phi P_n} = \frac{241.5}{265.36} = 0.91 \le 1 \ (壓)$$

$$\phi P_n = \phi A_g F_y = 0.9\times108\times3.5 = 340.2T \ (拉)$$

$$DCR = \frac{P_u}{\phi P_n} = \frac{241.5}{340.2} = 0.71 \le 1 \ (拉)$$

在低樓層中，因為連桿強度較高，因此選擇斷面要較上樓層大一些，故嘗試之後選 *HSS320X320X10*

Area	12400.
Iz	198813328.
Iy	198813328.
J	297910016.
Sz	1242583.375
Sy	1242583.375
Avy	6400.
Avz	6400.

$$V_p = V_p = 0.6\times(500-2\times30)\times16\times3.5/100 = 147.84T$$

$$V_{ult} = 1.25R_yV_p = 1.25\times1.1\times147.84 = 203.28T$$

$$P_{ult}^{brace} = 1.1V_{ult}/\sin 45^{\circ} = 1.1\times203.28/\sin 45^{\circ} = 316.23T$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{198813328}{12400}} = 126.62mm = 12.662cm$$

$$\lambda = \frac{kl}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{1\times565.69}{12.662}\sqrt{\frac{3.5\times100\times100}{\pi^2\times2.04\times10^7}} = 0.5889 \le 1.5$$

$$\phi P_n = \phi A_g F_{cr} = \phi A_g(0.658^{\lambda^2})F_y = 0.85\times124\times0.658^{0.3468}\times3.5$$
$$= 317.25T$$

(壓)

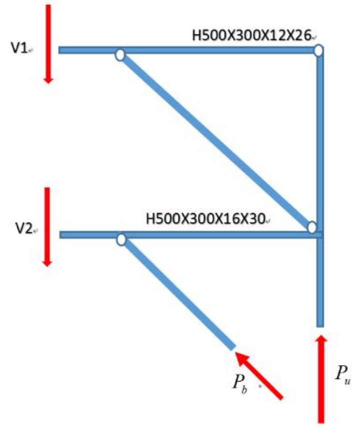
$$DCR = \frac{P_u}{\phi P_n} = \frac{316.23}{317.25} = 0.9967 \le 1 \ \ (壓)$$

$$\phi P_n = \phi A_g F_y = 0.9 \times 124 \times 3.5 = 390.6T \ \ (拉)$$

$$DCR = \frac{P_u}{\phi P_n} = \frac{316.23}{390.6} = 0.81 \le 1 \ \ (拉)$$

(5) If the column size is H350x350x15x32mm. Assume all the columns are laterally supported at the floor level, apply strain hardening factor of 1.1 for link and check the DCR in the column.

右圖是針對垂直向分力的自由體圖



$$V_1 = 1.1 R_y V_{p1} = 1.1 \times 1.1 \times 112.9 = 136.61T$$
$$V_2 = 1.1 R_y V_{p2} = 1.1 \times 1.1 \times 147.84 = 178.89T$$
$$P_b = 1.1 V_2 / \sin 45^\circ = 1.1 \times 178.89 / \sin 45^\circ = 278.29T$$
$$P_u = V_1 + V_2 - P_b \sin 45^\circ = 136.61 + 178.89 - 1.1 \times 178.89 = 118.721T$$

由題意知 柱的斷面為 H350X350X15X32

輸入 PISA3D 會幫我們計算好斷面性質如右圖

Area	26690.
Iz	597447936.
Iy	228747104.
J	7516586.
Sz	3413988.25
Sy	1307126.25
Avy	5250.
Avz	18666.666

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{228747104}{26690}} = 92.577mm = 9.2577cm$$

$$\lambda = \frac{kl}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{1 \times 400}{9.2577}\sqrt{\frac{3.5 \times 100 \times 100}{\pi^2 \times 2.04 \times 10^7}} = 0.57 \le 1.5 \qquad (壓)$$

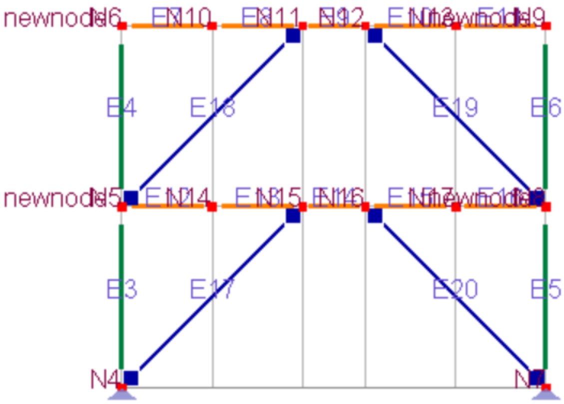
$$\phi P_n = \phi A_g F_{cr} = \phi A_g (0.658^{\lambda^2}) F_y = 0.85 \times 266.9 \times 0.658^{0.3249} \times 3.5$$
$$= 693.1T$$

$$DCR = \frac{P_u}{\phi P_n} = \frac{118.721}{693.1} = 0.171 \le 1 (壓)$$

$$\phi P_n = \phi A_g F_y = 0.9 \times 266.9 \times 3.5 = 840.735T \ \ (拉)$$

$$DCR = \frac{P_u}{\phi P_n} = \frac{118.721}{840.735} = 0.141 \le 1 \ \ (拉)$$

(6) Construct the PISA3D model, apply the stated lateral forces and compute the elastic inter-story drift. Apply the C_d factor (prescribed in AISC 2005), estimate the inelastic link deformational demands and size the link web stiffeners for the upper and lower links.



upper beam	H500X300X12X26
lower beam	H500X300X16X30
upper brace	HSS280X280X10
lower brace	HSS320X320X10
upper link	H500X300X12X26
lower link	H500X300X16X30
column	H350X350X15X32

Section properties :

Upper beam:

Area	20976.
Iz	967030592.
Iy	117064512.
J	3576964.5
Sz	3868122.25
Sy	780430.063
Avy	6000.
Avz	13000.

lower beam:

Area	25040.
Iz	1108978688.
Iy	135150192.
J	5646786.5
Sz	4435914.5
Sy	901001.25
Avy	8000.
Avz	15000.

Upper Brace :

Area	10800.
Iz	131400000.
Iy	131400000.
J	196830000.
Sz	938571.438
Sy	938571.438
Avy	5600.
Avz	5600.

Lower brace:

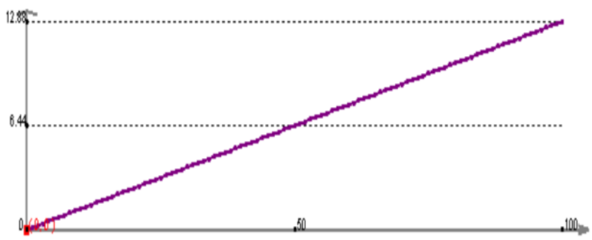
Area	12400.
Iz	198813328.
Iy	198813328.
J	297910016.
Sz	1242583.375
Sy	1242583.375
Avy	6400.
Avz	6400.

Column:

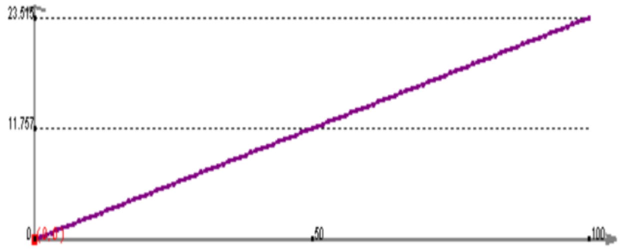
Area	26690.
Iz	597447936.
Iy	228747104.
J	7516586.
Sz	3413988.25
Sy	1307126.25
Avy	5250.
Avz	18666.666

定義 LOAD CASE 並基底設為 pinned 分析之後 利用 plot 求得一樓頂及二樓頂層位移結果:

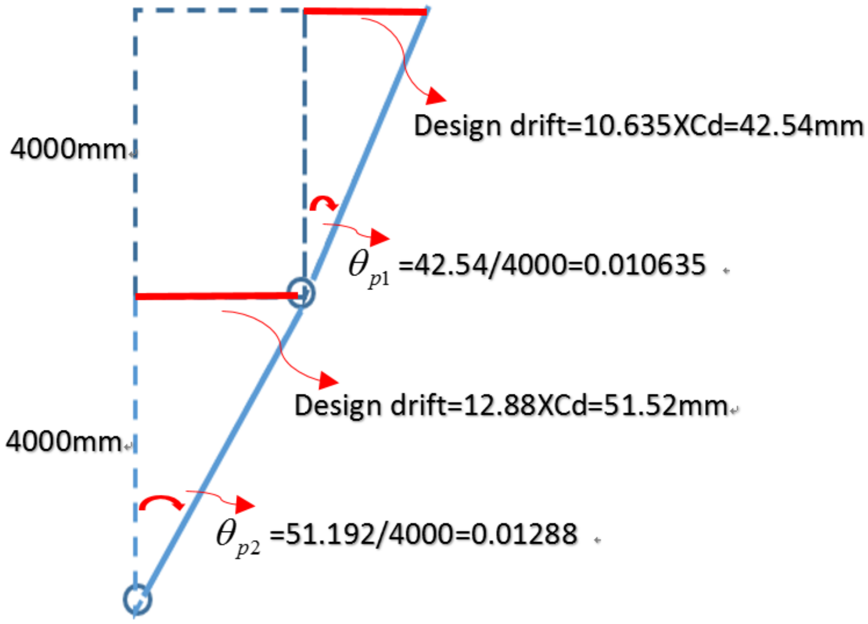
1stDrift:



2ndDrift:



設 $C_d = 4$ 且 L=9400mm, e=1400mm



$$\gamma_{p1} = \frac{L}{e}\theta_{p1} = \frac{9400}{1400} \times 0.010635 = 0.0714 rad$$

$$\gamma_{p2} = \frac{L}{e}\theta_{p2} = \frac{9400}{1400} \times 0.01288 = 0.08648 rad$$

若相對長度小於 1.6(剪力連桿) 則 stiffener :

寬不得小於 $b_f - 2t_w$ 且

厚不得小於 $0.75t_w$ 或 3/8-inch 且

若 γ_p 大於 0.08 間隔(spaceing)不得大於 $30t_w - \frac{d}{5}$

γ_p 小於 0.02 間隔(spaceing)不得大於 $52t_w - \frac{d}{5}$ ，

介於之間則內插。

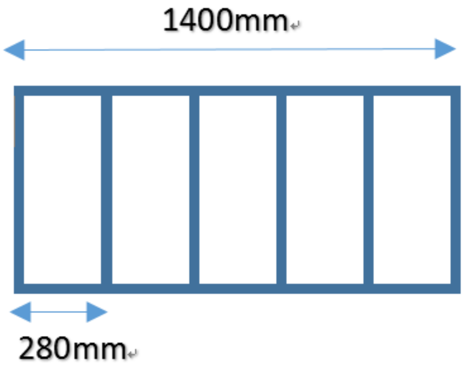
則在頂層連桿中，使用的斷面為 H500X300X12X26,且 $\gamma_{p1}=0.0714rad$

寬 $\geq b-2t_w=300-2\times12=276\,\text{mm}$; 厚 $\geq 0.75t_w=9mm$ 或 9.5mm

$$間隔 \leq \frac{0.08-0.0714}{0.08-0.02}(52\times12-\frac{500}{5})+\frac{0.0714-0.02}{0.08-0.02}(30\times12-\frac{500}{5})=297.84mm$$

因此取:

間隔=280mm 寬=280mm 厚=9.5mm



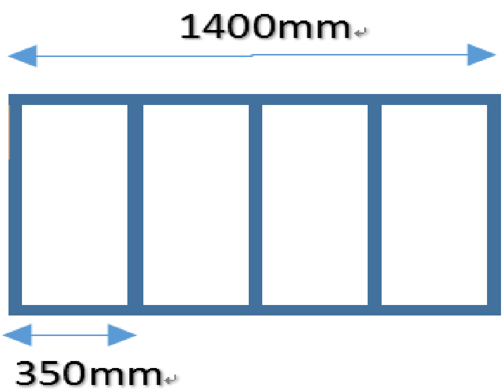
則在低層連桿中，使用的斷面為 H500X300X16X30,且 $\gamma_{p1}=0.08648rad$

寬 $\geq b-2t_w=300-2\times16=268\,\text{mm}$; 厚 $\geq 0.75t_w=12mm$ 或 9.5mm

$$間隔 \leq 30\times16-\frac{500}{5}=380\,\text{mm}$$

因此取:

間隔=350mm 寬=270mm 厚=12mm



(7) Apply proper material model for the link so that the link can yield in shear and bending accordingly. Keep the stated lateral load pattern unchanged but apply cyclic roof drift for ±0.005, ±0.01, ±0.015, ±0.020 and ±0.025 radians, two cycles each. Properly specify the strain hardening control parameters so that at the ±0.025 radians story deformation, the strain hardened shear strength is between 1.3~1.4V_p. Plot the cyclic lateral shear force versus lateral drift, link shear versus link rotation relationships for both floors. Mark the LRFD lateral shear force and the link shear yield strength in the plots.